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ENGINEERING INSTRUCTION

NEW YORK STATE DEPARTMENT OF TRANSPORTATION

SUBJECT: REVISION TO STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES

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The attached pages are revisions to Standard Specifications for Highway Bridges. These pages should be inserted immediately in the Manual.

Pages I to XX - These pages replace, in their entirety, the old CONTENTS AND INDEX sections.

Page 5 - No change
Page 6 - Section 1.1.9 - Revised
Page 7 - Section 1.1.9 - Continuation of revision on page 6
Section 1.1.10 - No change
Page 8 - Rewritten - No change
Page 8A - Figure 1.1.9 - Revised
Page 8B - Added - New information
Page 11 - Section 1.2.4 - Revised
Page 12 - No change
Page 12A - Footnote added
Page 12B - No change
Page 12C - Footnote added
Page 13 - No change
Page 14 - No change
Page 15 - Section 1.2.11(C)(1.) - Revised
Page 16 - Section 1.2.11(C)(1.) - Revision continued
Page 16A - Rewritten due to previous revision -
Includes top three lines from old page 17
Page 17 - No change except deletion of top three lines which are now on page 16A. Back side of page 17 is blank.
Page 27 - Section 1.3.1 - Footnote added
Page 28 - Item on "Prestressed Concrete Spread Box Beams" added
Page 37 - No change
Page 38 - Section 1.3.6 - Footnote added
Page 57 - Revised 1.4.8(F) - Expansion and Contraction Joints *****
Page 58 - No change
Page 59 - No change - Back of page 59 is blank
Pages 103, 104, 104A - Section 1.6.24(A) New, Subparagraphs (B), (C), (D), (E) and (F) same as old paragraphs (A), (B), (C), (D) and (E).
Back of page 104A is blank

Manual	Code	Date	Page
Subject:			

Page 109B - Subitem (J) deleted
Page 109C - No change
Page 111 - No change
Page 112 - Rewritten - Last two paragraphs added from old page 113
Page 113 - Rewritten to include new information added after last formula
Pages 114 to 162 inclusive - All section numbers revised including any referenced sections. (NOTE: Pages 135A (blank on back), 157 and 158 are not included.)
Page 118 - Section 1.7.13 - Revised
Page 120 - Section 1.7.19 - Last paragraph revised
Page 135 - Section 1.7.69 - Added reference to Hybrid Girders
Page 139 - Section 1.7.73 - Added reference to Hybrid Girders
Pages 159 and 160 - Section 1.7.110 - Revised
Page 160 - Section 1.7.111 - Revised
Page 160A - Continuation of Section 1.7.111. Back of page 160A is blank
Page 160B - Footnote added
Page 161 - Section 1.7.115 - First formula corrected
Page 162 - No change. Back of page 162 is blank
Pages 162-1 to 162-23 - All section numbers to and including Section 1.7.138, revised including any referenced sections - No revisions in these pages
Pages 162-23 to 162-26 - Sections 1.7.139 to 1.7.148 (Pertaining to Orthotropic - Deck - Bridges) New

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CONTENTS

DESIGN & (DETAILING INDEX)

DESIGN

DESIGN & (DETAILING) INDEX

References to items in the Standard Details for Highway Bridges Manual are shown in parentheses ( ).

THE WIDTH OF THE SIDEWALK SHALL BE THE CLEAR WIDTH, MEASURED AT RIGHT ANGLES TO THE LONGITUDINAL CENTER LINE OF THE BRIDGE, FROM THE EXTREME INSIDE PORTION OF THE HANDRAIL TO THE BOTTOM OF THE CURB OR GUARD-TIMBER, EXCEPT THAT IF THERE IS A TRUSS, GIRDER, OR PARAPET WALL ADJACENT TO THE ROADWAY CURB, THE WIDTH SHALL BE MEASURED TO ITS EXTREME WALK SIDE PORTION.

1.1.7-CLEARANCES

A. NAVIGATIONAL

PERMITS FOR THE CONSTRUCTION OF CROSSINGS OVER NAVIGABLE STREAMS, EXCEPT THOSE STREAMS THAT HAVE BEEN PLACED IN THE "ADVANCE APPROVAL" CATEGORY BY THE COMMANDANT, U.S. COAST GAURD, MUST BE OBTAINED FROM THE U.S. COAST GAURD AND OTHER APPROPRIATE AGENCIES. REQUESTS FOR SUCH PERMITS FROM THE U.S. COAST GUARD SHOULD BE ADDRESSED TO THE APPROPRIATE DISTRICT COMMANDER.

B. VEHICULAR *****

FOR REQUIRED CLEARANCES, REFER TO THE POLICY ON GEOMETRICS OF STRUCTURES DATED JULY 1968 WITH CURRENT ADDENDUMS AND MODIFICATIONS

C. OTHER

THE CHANNEL OPENINGS AND CLEARANCE SHALL BE CLEARED WITH OTHER AGENCIES HAVING JURISDICTION OVER SUCH MATTERS. CHANNEL OPENINGS AND CLEARANCES IN GENERAL SHALL CONFORM IN WIDTH, HEIGHT, AND LOCATION TO ALL FEDERAL, STATE AND LOCAL REQUIREMENTS.

1.1.8-CURBS AND SIDEWALKS *****

THE FACE OF THE CURB IS DEFINED AS THE VERTICAL OR SLOPING SURFACE ON THE ROADWAY SIDE OF THE CURB. HORIZONTAL MEASUREMENTS OF ROADWAY AND CURB WIDTH ARE GIVEN FROM THE BOTTOM OF THE FACE, OR, IN THE CASE OF STEPPED BACK CURBS, FROM THE BOTTOM OF THE LOWER FACE FOR ROADWAY WIDTH. WIDTH OF BRUSH CURBS, IF USED, SHALL BE 6 INCHES.

WHERE CURB AND GUTTER SECTIONS ARE USED ON THE ROADWAY APPROACH, AT EITHER OR BOTH ENDS OF THE BRIDGE, THE CURB HEIGHT ON THE BRIDGE SHALL MATCH THE CURB HEIGHT ON THE ROADWAY APPROACH. WHERE NO CURBS ARE USED ON THE ROADWAY APPROACHES, THE HEIGHT OF THE BRIDGE CURB ABOVE THE ROADWAY SHALL BE 6 INCHES.

CURBS SHALL NOT BE USED ON STRUCTURES OVER FEATURES OTHER THAN HIGHWAYS OR RAILROADS, UNLESS THE APPROACH HIGHWAY IS CURBED, THE STRUCTURE IS A HIGH-LEVEL BRIDGE, THE SHOULDERS ARE LESS THAN FIVE FEET IN WIDTH, OR THEY ARE REQUIRED FOR DRAINAGE.

1.1.9-RAILINGS ****

Railing shall be provided at the edge of structures for the protection of traffic and for the protection of pedestrians if pedestrian walkways are provided.

Where pedestrian walkways are provided adjacent to roadways, a traffic railing or barrier may be provided between the two with a pedestrian railing outside.

A. TRAFFIC RAILING

While the primary purpose of traffic railing is to contain the average vehicle using the structure, consideration should also be given to protection of the occupants of a vehicle in collision with the railing, to protection of other vehicles near the collision, to vehicles or pedestrians on roadways being over-crossed, and to appearance and freedom of view from passing vehicles.

Materials for traffic railing shall be concrete, metal, timber or a combination, metal materials with less than 10 percent tested elongation shall not be used.

Traffic railings should provide a smooth, continuous face of rail on the traffic side with the posts set back from the face of rail. Structural continuity in the rail members, including anchorage of ends, is essential. The railing system shall be able to resist the applied loads at all locations.

Protrusions or depressions at rail joints shall be acceptable provided their thickness or depth is no greater than the wall thickness of the rail member or 3/8" whichever is less.

The height of traffic railing shall be no less than 2'-3" measured from the top of the roadway or curb to the top of the upper rail member, except that parapets designed with sloping traffic faces intended to allow vehicles to ride up them under low angle contacts shall be at least 2'-8" in heights. This sloping face parapet height may be reduced to 2'-3" provided a traffic railing is mounted on top of the parapet at a height not exceeding 3'-3". The lower element of a traffic or combination railing should consist of a parapet at least 18 inches high or a rail centered between 15 and 20 inches above the roadway surface or surface of curb or sidewalk extending more than 6 inches in front of the traffic face of the railing. The maximum clear vertical opening below the lower rail or between succeeding rails shall not exceed 15 inches. (See Figure 1.1.9) Railings other than those shown in Figure 1.1.9 are permissible provided the total applied loading is not less than 10 kips.

Careful attention should be given to the treatment of railing at the bridge ends, exposed rail ends, posts and sharp changes in the geometry of the railing should be avoided.

A smooth transition by means of a continuation of the bridge barrier, guide rail anchored to the bridge ends, or other effective means shall be provided to protect traffic from direct collision with bridge rail ends.

B. PEDESTRIAN RAILING *****

Railing components shall be proportioned commensurate with the type and volume of anticipated pedestrian traffic, taking account of appearance, safety and freedom of view from passing vehicles.

Materials for pedestrian railing may be concrete, metal, timber or a combination.

The minimum height of pedestrian railing shall be 3'-0" (a preferred height is 3'-6") measured from the top of the walkway to the top of the upper rail member.

Rail heights shown shall be measured from top of curb if the curb width exceeds six inches, but from the top of roadway if the curb width is six inches or less. (See Figure 1.1.9).

1.1.10-ROADWAY DRAINAGE *****

The transverse drainage of the roadway should be accomplished by providing a suitable crown in the roadway surface and longitudinal drainage should be accomplished by camber or gradiant, water flowing downgrade in a gutter section should be intercepted and not permitted to run onto the bridge. Short, continuous span bridges, particularly over-passes, may be built without inlets and the water from the bridge roadway carried downslope by open or closed chutes near the end of the bridge structure, longitudinal drainage on long bridges is accomplished by means of scuppers or inlets which should be of sufficient size and number to drain the gutters adequately, downspouts, where required, should be of rigid corrosion-resistant material not less than 4 inches in least dimension and should be provided with cleanouts. The details of deck drains should be such as to prevent the discharge of drainage water against any portion of the structure and to prevent erosion at the outlet of the downspout.

Deck drainage shall be designed in accordance with Bureau of Public Roads circular memorandum No. 2 on surface drainage, the design being based on the rainfall intensity of the most severe storm of five minute duration likely to occur in a ten year period.

Sufficient scuppers shall be provided that the puddle will not encroach more than 4 feet into the travelled way.

1.1.11-SUPER ELEVATION *****

The super elevation of the floor surface of a bridge on a horizontal curve shall be provided in accordance with the A.A.S.H.O. policy on geometric design of urban or rural highways using a maximum of .08 feet per foot of roadway width.

1.1.12-FLOOR SURFACES

All bridge floors shall have skid-resistant characterciistics.

1.1.13-BLAST PROTECTION (DELETED)

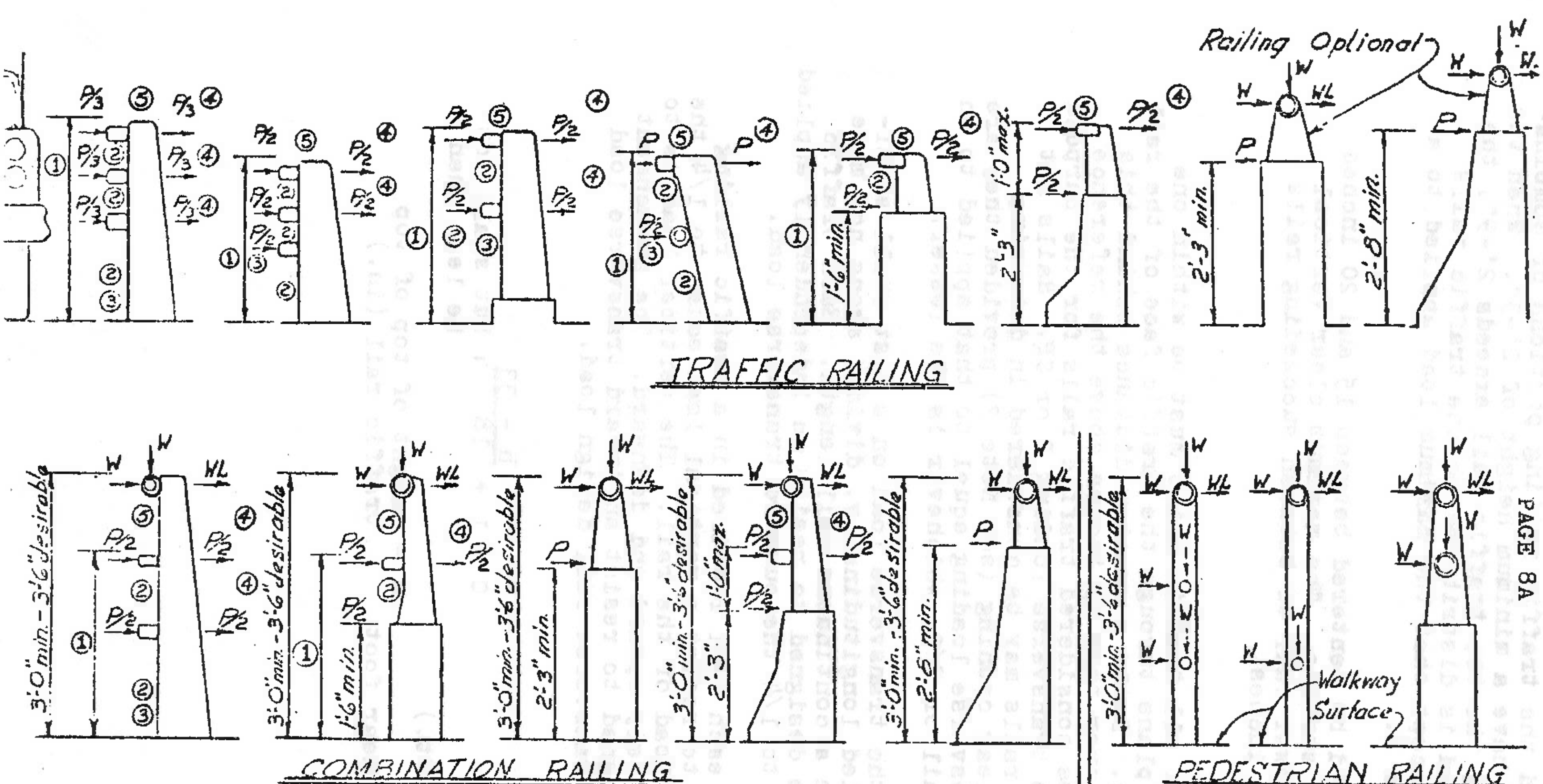
1.1.14-UTILITIES

Where required, provisions shall be made for trolley wire supports and poles, pillars for lights, electric conduits, telephone conduits, water pipes, gas pipes and sanitary sewers.

1.1.15-ROADWAY WIDTHS, CURBS AND CLEARANCES FOR TUNNELS (DELETED)

1.1.16 + 1.1.17-ROADWAY WIDTH, CURBS AND CLEARANCES FOR DEPRESSED ROADWAYS AND UNDERPASSES

For required clearances refer to policy on Geometrics of Structures dated July 1968 with Current Addendums and Modifications.



LOADINGS ON LEFT OF RAILING ARE RAIL LOADINGS.
LOADINGS ON RIGHT OF RAILING ARE POST LOADINGS.

THE SHAPES OF RAIL MEMBERS ARE ILLUSTRATIVE ONLY. ANY MATERIAL OR COMBINATION OF MATERIALS LISTED IN ARTICLE 1.1.9 MAY BE USE IN ANY CONFIGURATION

RAIL HEIGHTS SHALL BE MEASURED FROM THE TOP OF CURB IF THE CURB WIDTH EXCEEDS SIX INCHES, BUT FROM THE TO OF ROADWAY IF THE CURB WIDTH IS SIX INCHES OR LESS. (THE SURFACE FROM WHICH THE RAIL HEIGHT IS MEASURED IS THE REFERENCE SURFACE.)

FIGURE 1.1.9

SEE NOTES ON THE FOLLOWING PAGE.

8-B

Notes for Fig. 1.1.9

1. Traffic railings and the traffic railing portions of combination railings shall have a minimum height of 2'-3". When the height of the top of the top traffic rail exceeds 2'-9", the total transverse load is distributed to the traffic rails shall be equal CP except that the maximum load applied to a rail need not exceed P.

2. The lower rail should be centered between 15 and 20 inches above the reference surface. The maximum clear vertical opening below the lower rail or between succeeding rails shall not exceed 15 inches.

3. The traffic faces of all traffic rails must be within one inch of a vertical plane through the traffic face of the rail closest the traffic. Rails a greater distance behind this plane or centered lower than 15 inches above the reference surface shall not be considered traffic rails for the purpose of distributing the transverse loading P or CP. Rails not considered traffic rails may be considered in determing maximum vertical clear opening (see Note 2) provided they are designed for a transverse loading equal to that applied to an adjacent traffic rail or P/2, whichever is the lesser.

4. A load equal 1/2 the transverse load on a post shall simultaneously be applied longitudinally, divided among not more than four posts in a continuous rail length. Each traffic post shall also be designed to resist an independently applied inward load equal to 1/4 the outward transverse load.

5. The attachment of each rail required in a traffic railing shall be designed to resist a vertical load equal to 1/4 the transverse design load of the rail. The vertical load is to be applied alternately upward and downward. The attachment shall also be designed to resist an inward transverse load equal to 1/4 the transverse rail design load.

Nomenclature:

$P = 10{,}000$ lbs.

L = post spacing (ft.)

$w = 50$ lbs. per linear foot

$C = 1 + \frac{h - 33}{18}$; but shall not be less than 1

h = height of top of top traffic rail (in.)

H=Depth in feet of fill over culvert.
W=Effective weight, per cu. ft., of fill material
(May be taken as 70% or 83% in accordance with above provision)
e=2.7182818 = Base of natural logarithms, abstract number

B. SHEAR IN TOP SLABS

The maximum shear in the top slabs of culvert under embankments shall be assumed to occur at a distance, "D", out from the wall or abutment; "D" being equal to the depth from the compression face of the slab to the centroid of the tension reinforcement.

C. SHEAR IN BOTTOM SLABS

The shear in bottom slabs shall be computed as specified for footings in Article 1.4.6.

1.2.3-LIVE LOAD

The live load shall consist of the weight of the applied moving load of vehicles, cars and pedestrians.

1.2.4-OVERLOAD PROVISION *

The following provision for overload shall apply to all loadings except the H 20 and HS 20 loadings:

Provision for infrequent heavy loads shall be made by applying in any single lane an H or HS truck as specified, increased 100 per cent, and without concurrent loading of any other lanes, combined dead, live and impact stresses resulting from such loading shall not be greater than 150 per cent of the allowable stresses prescribed herein. The overload shall apply to all parts of the structure affected, except the deck.

* For orthotropic-deck bridges, the deck consists of the deck plate and stiffening ribs.

1.2.5-HIGHWAY LOADINGS

A. GENERAL

The highway live loadings on the roadways of bridges or incidental structures shall consist of standard trucks or of lane loads which are equivalent to truck trains. Two systems of loading are provided. The H loadings and the HS loadings. The corresponding HS loadings being heavier than the H loadings.

B. H LOADINGS

THE H LOADINGS ARE ILLUSTRATED IN FIGURES 1.2.5A AND 1.2.5B. THEY CONSIST OF A TWO-AXLE TRUCK OR THE CORRESPONDING LANE LOADING. THE H LOADINGS ARE DESIGNATED H FOLLOWED BY A NUMBER INDICATING THE GROSS WEIGHT IN TONS OF THE STANDARD TRUCK.

C. HS LOADINGS

THE HS LOADINGS ARE ILLUSTRATED IN FIGURES 1.2.5B AND 1.2.5C. THEY CONSIST OF A TRACTOR TRUCK WITH SEMI-TRAILER OR OF THE CORRESPONDING LANE LOADING. THE HS LOADINGS ARE DESIGNATED BY THE LETTERS HS FOLLOWED BY A NUMBER INDICATING THE GROSS WEIGHT IN TONS OF THE TRACTOR TRUCK. THE VARIABLE AXLE SPACING HAS BEEN INTRODUCED IN ORDER THAT THE SPACING OF AXLES MAY APPROXIMATE MORE CLOSELY THE TRACTOR TRAILERS NOW IN USE. THE VARIABLE SPACING ALSO PROVIDES A MORE SATISFACTORY LOADING FOR CONTINUOUS SPANS, IN THAT HEAVY AXLE LOADS MAY BE SO PLACED ON ADJOINING SPANS AS TO PRODUCE MAXIMUM NEGATIVE MOMENT.

D. CLASSES OF LOADING

HIGHWAY LOADINGS SHALL BE OF FIVE CLASSES: H 20, H 15, H 10, HS 20 AND HS 15. LOADINGS H 15 AND H 10 ARE 75 PER CENT AND 50 PER CENT, RESPECTIVELY, OF LOADING H 20. LOADING HS 15 IS 75 PER CENT OF LOADING HS 20. IF LOADINGS OF WEIGHTS OTHER THAN THOSE DESIGNATED ARE DESIRED, THEY SHALL BE OBTAINED BY PROPORTIONATELY CHANGING THE WEIGHTS SHOWN FOR BOTH THE STANDARD TRUCK AND THE CORRESPONDING LANE LOADS.

E. DESIGNATION OF LOADINGS

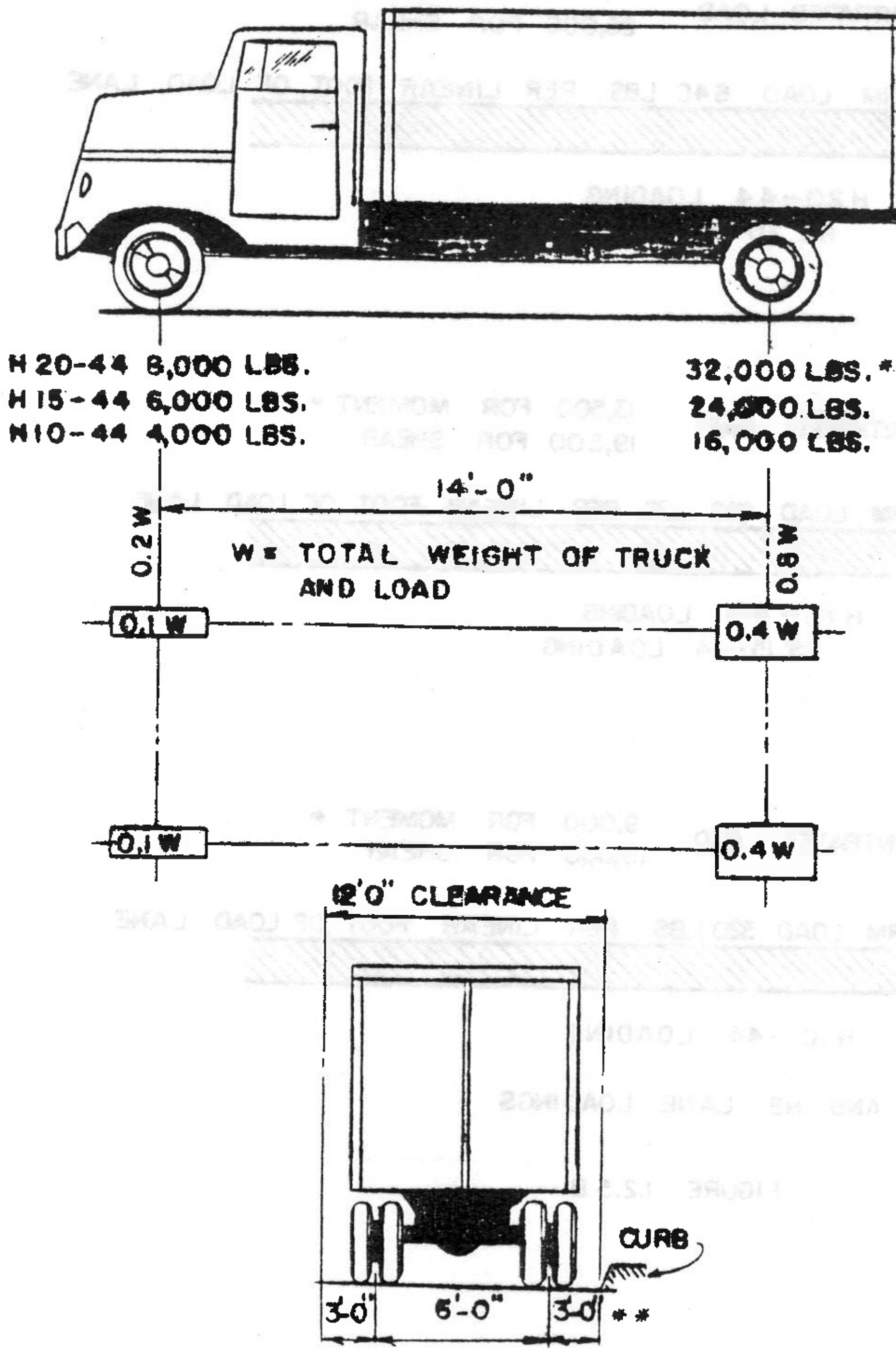
THE POLICY OF AFFIXING THE YEAR TO LOADINGS TO IDENTIFY THEM WAS INSTITUTED WITH THE PUBLICATION OF THE 1944 EDITION IN THE FOLLOWING MANNER:

H10 LOADING, 1944 EDITION SHALL BE DESIGNATED	H10-44
H15 LOADING, 1944 EDITION SHALL BE DESIGNATED	H15-44
H20 LOADING, 1944 EDITION SHALL BE DESIGNATED	H20-44
H15-S12 LOADING, 1944 EDITION SHALL BE DESIGNATED	HS15-44
H20-S16 LOADING, 1944 EDITION SHALL BE DESIGNATED	HS20-44

THE AFFIX REMAINS UNCHANGED UNTIL SUCH TIME AS THE LOADING SPECIFICATION IS REVISED. THE SAME POLICY FOR IDENTIFICATION SHALL BE APPLIED, FOR FUTURE REFERENCE, TO LOADINGS PREVIOUSLY ADOPTED BY THE AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS.

F. MINIMUM LOADING *****

IN GENERAL HS20 LIVE LOAD SHALL BE USED FOR THE DESIGN OF STRUCTURES ON ALL HIGHWAYS.

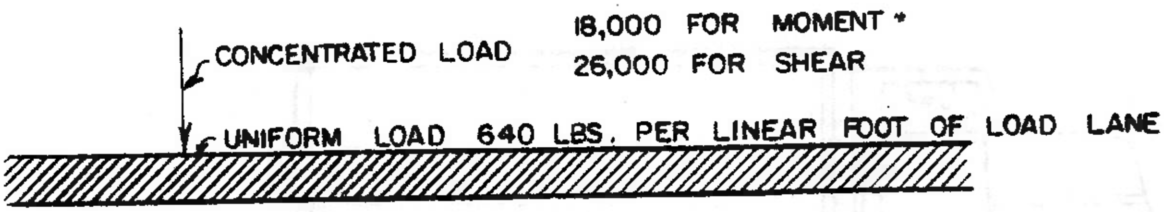


STANDARD H TRUCKS

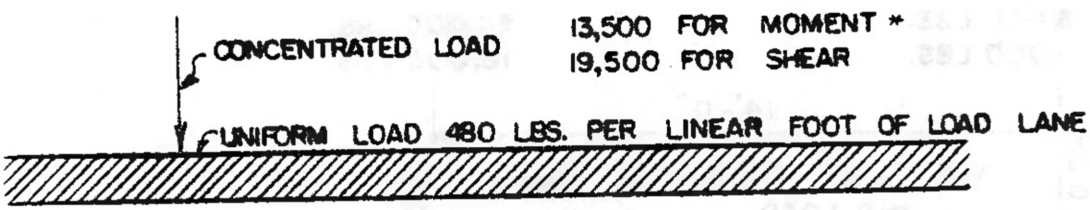
FIGURE 1.2.5 A

*In the design of timber floors and orthotropic steel decks (excluding transverse beams) for H20 loading, one axle load of 24,000 pounds or two axle loads of 16,000 pounds each, spaced 4 feet apart may be used, whichever produces the greater stress, instead of the 32,000 pound axle shown.

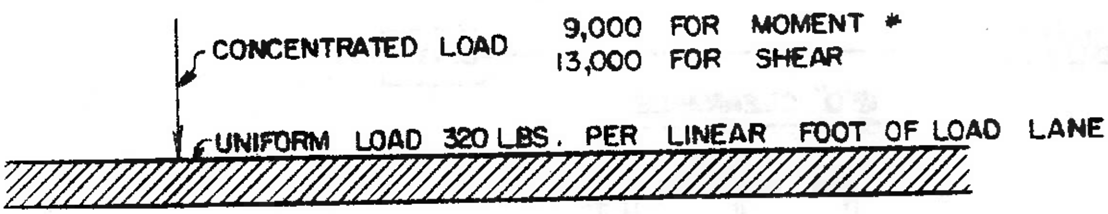
**For slab design, the center line of wheels shall be assumed to be 1 foot from face of curb. (See Art. 1.3.2(B))



H 20-44 LOADING
HS 20-44 LOADING



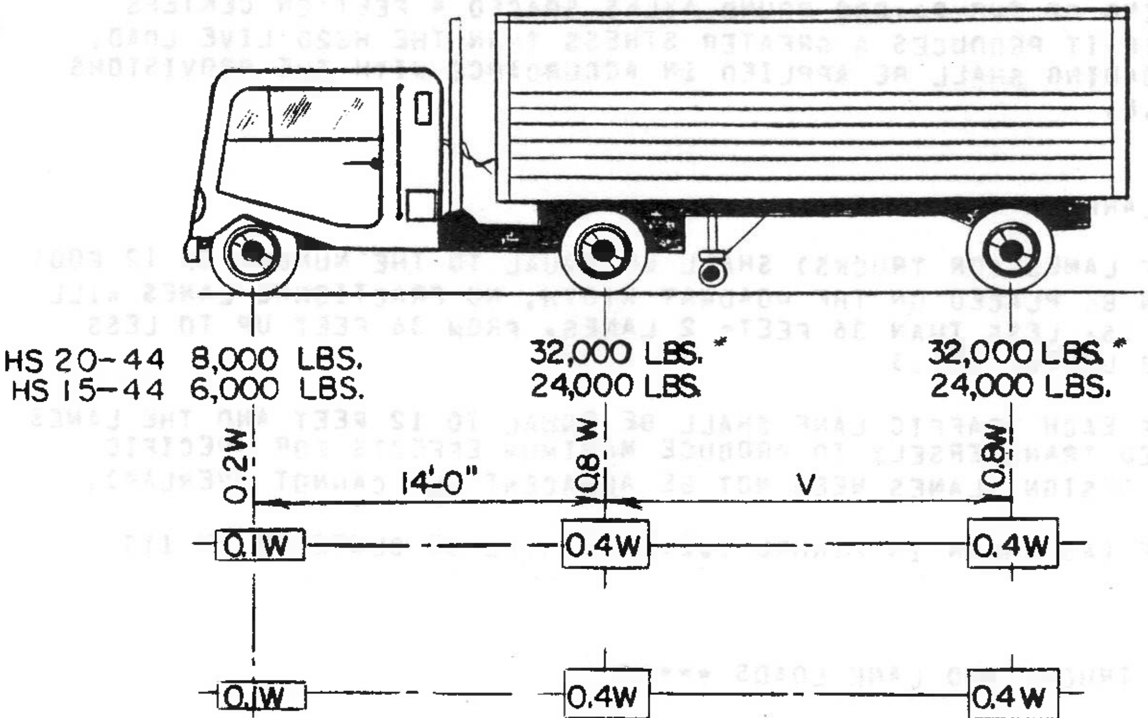
H 15-44 LOADING
HS 15-44 LOADING



H 10-44 LOADING

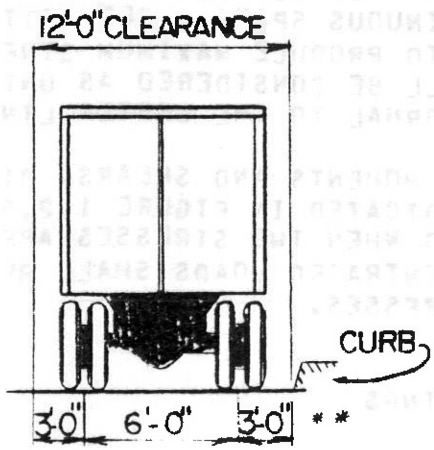
H LANE AND HS LANE LOADINGS

FIGURE 1.2.5 B



W= COMBINED WEIGHT ON THE FIRST TWO AXLES WHICH IS THE SAME AS FOR THE CORRESPONDING H TRUCK.

V= VARIABLE SPACING - 14 FEET TO 30 FEET INCLUSIVE. SPACING TO BE USED IS THAT WHICH PRODUCES MAXIMUM STRESSES



STANDARD HS TRUCKS

FIGURE 1.2.5 C

*In the design of timber floors and orthotropic steel decks (excluding transverse beams) for HS20 loading, one axle load of 24,000 pounds or two axle loads of 16,000 pounds each, spaced 4 feet apart may be used, whichever produces the greater stress, instead of the 32,000 pound axle shown.

**For slab design, the center line of wheels shall be assumed to be 1 foot from face of curb. (See Art. 1.3.2(B))

IN ADDITION FOR STRUCTURES ON THE MAIN LINE OF TRUNK HIGHWAYS, THE NATIONAL SYSTEM OF INTERSTATE HIGHWAYS AND OTHER DESIGNATED EXPRESSWAYS A SPECIAL LOADING OF TWO 24,000 POUND AXLES SPACED 4 FEET ON CENTERS SHALL BE USED IF IT PRODUCES A GREATER STRESS THAN THE HS20 LIVE LOAD. THIS SPECIAL LOADING SHALL BE APPLIED IN ACCORDANCE WITH THE PROVISIONS OF ARTICLE 1.2.8.

1.2.6-TRAFFIC LANES *****

THE NUMBER OF LANES (OR TRUCKS) SHALL BE EQUAL TO THE NUMBER OF 12 FOOT LANES WHICH CAN BE PLACED ON THE ROADWAY WIDTH. NO FRACTIONAL LANES WILL BE USED. (THAT IS, LESS THAN 36 FEET- 2 LANES, FROM 36 FEET UP TO LESS THAN 48 FEET- 3 LANES, ETC.)

THE WIDTH OF EACH TRAFFIC LANE SHALL BE EQUAL TO 12 FEET AND THE LANES SHALL BE SHIFTED TRANSVERSELY TO PRODUCE MAXIMUM EFFECTS FOR SPECIFIC ELEMENTS UNDER DESIGN (LANES NEED NOT BE ADJACENT BUT CANNOT OVERLAP).

THE VECHICLE (AS SHOWN IN FIGURE 1.2.5 C) WILL BE CENTERED IN ITS DESIGN LANE.

1.2.7-STANDARD TRUCKS AND LANE LOADS *****

THE WHEEL SPACING, WEIGHT DISTRIBUTION, AND CLEARANCE OF THE STANDARD H AND HS TRUCKS SHALL BE AS SHOWN IN FIGURES 1.2.5A AND 1.2.5C AND CORRESPONDING LANE LOADS SHALL BE AS SHOWN IN FIGURE 1.2.5B.

EACH LANE LOADING SHALL CONSIST OF A UNIFORM LOAD PER LINEAR FOOT OF TRAFFIC LANE COMBINED WITH A SINGLE CONCENTRATED LOAD(OR TWO CONCENTRATED LOADS IN THE CASE OF CONTINUOUS SPANS - SEE ARTICLE 1.2.8(C)) SO PLACED ON THE SPAN AS TO PRODUCE MAXIMUM STRESS. THE CONCENTRATED LOAD AND UNIFORM LOAD SHALL BE CONSIDERED AS UNIFORMLY DISTRIBUTED OVER A 12 FEET WIDTH ON A LINE NORMAL TO THE CENTER LINE OF THE LANE.

FOR THE COMPUTATION OF MOMENTS AND SHEARS, DIFFERENT CONCENTRATED LOADS SHALL BE USED AS INDICATED IN FIGURE 1.2.5B. THE LIGHTER CONCENTRATED LOADS SHALL BE USED WHEN THE STRESSES ARE PRIMARILY BENDING STRESSES AND HEAVIER CONCENTRATED LOADS SHALL BE USED WHEN THE STRESSES ARE PRIMARILY SHEARING STRESSES.

1.2.8-APPLICATION OF LOADINGS

A. TRAFFIC LANE UNITS *****

IN COMPUTING STRESSES, EACH 12 FOOT LANE LOADING OR SINGLE STANDARD. TRUCK SHALL BE CONSIDERED AS A UNIT, AND FRACTIONAL LOAD LANE WIDTHS OR FRACTIONAL TRUCKS SHALL NOT BE USED.

B. NUMBER AND POSITION, TRAFFIC LANE UNITS

THE NUMBER AND POSITION OF THE LANE LOADINGS OR TRUCK LOADINGS SHALL BE AS SPECIFIED IN ARTICLE 1.2.6 AND, WHETHER LANE LOADING OR TRUCK LOADING, SHALL BE SUCH AS TO PRODUCE MAXIMUM STRESS, SUBJECT TO THE REDUCTION SPECIFIED IN ARTICLE 1.2.9.

C. LANE LOADINGS-CONTINUOUS SPANS

THE LANE LOADINGS SHOWN IN FIGURE 1.2.5B SHALL BE MODIFIED AS FOLLOWS FOR THE DESIGN OF CONTINUOUS SPANS: THE LANE LOADINGS SHALL CONSIST OF THE LOADS SHOWN IN FIGURE 1.2.5B AND IN ADDITION THERETO ANOTHER CONCENTRATED LOAD OF EQUAL WEIGHT SHALL BE PLACED IN ONE OTHER SPAN IN THE SERIES IN SUCH POSITION AS TO PRODUCE MAXIMUM NEGATIVE MOMENT. FOR MAXIMUM POSITIVE MOMENT, ONLY ONE CONCENTRATED LOAD SHALL BE USED PER LANE, COMBINED WITH AS MANY SPANS LOADED UNIFORMLY AS REQUIRED TO PRODUCE MAXIMUM MOMENT.

D. LOADING FOR MAXIMUM STRESS

THE TYPE OF LOADING, WHETHER LANE LOADING OR TRUCK LOADING, TO BE USED, AND WHETHER THE SPANS BE SIMPLE OR CONTINUOUS, SHALL BE THE LOADING WHICH PRODUCES THE MAXIMUM STRESS. THE MOMENT AND SHEAR TABLES GIVEN IN APPENDIX A SHOW WHICH LOADING CONTROLS FOR SIMPLE SPANS. THE AXLE SPACING FOR HS TRUCKS SHALL BE VARIED BETWEEN THE SPECIFIED LIMITS TO PRODUCE MAXIMUM STRESSES.

FOR CONTINUOUS SPANS, THE LANE LOADING SHALL BE CONTINUOUS OR DIS-CONTINUOUS, AS MAY BE NECESSARY TO PRODUCE MAXIMUM STRESSES, AND THE CONCENTRATED LOAD OR LOADS AS SPECIFIED IN PARAGRAPH (C) SHALL BE PLACED IN SUCH POSITIONS AS TO PRODUCE MAXIMUM STRESSES.

FOR CONTINUOUS SPANS, ONLY ONE STANDARD H OR HS TRUCK PER LANE SHALL BE CONSIDERED ON THE STRUCTURE AND PLACED SO AS TO PRODUCE MAXI-MUM POSITIVE AND NEGATIVE MOMENTS.

1.2.9-REDUCTION IN LOAD INTENSITY

WHERE MAXIMUM STRESSES ARE PRODUCED IN ANY MEMBER BY LOADING ANY NUMBER OF TRAFFIC LANES SIMULTANEOUSLY, THE FOLLOWING PERCENTAGES OF THE RESULTANT LIVE LOAD STRESSES SHALL BE USED IN VIEW OF IMPROBABLE COINCIDENT MAXIMUM LOADING:

	PER CENT
ONE OR TWO LANES	100
THREE LANES	90
FOUR LANES OR MORE	75

THE REDUCTION IN INTENSITY OF FLOOR BEAM LOADS SHALL BE DETERMINED AS IN THE CASE OF MAIN TRUSSES OR GIRDERS, USING THE WIDTH OF ROADWAY WHICH must be loaded to produce maximum stresses in the floor beam.

1.2.10-ELECTRIC RAILWAY LOADING (DELETED)

1.2.11-SIDEWALK, CURB, SAFETY CURB AND RAILING LOADING

A. SIDEWALK LOADING

Sidewalk floor, stringers and their immediate supports, shall be designed for a live load of 85 pounds per square foot of sidewalk area, girders, trusses, arches and other members shall be designed for the following sidewalk live loads per square foot of sidewalk area:

Spans 0 to 25 ft. in length	85 Lbs.
Spans 26 to 100 ft. in length	60 Lbs.

Spans over 100 ft. in length according to the formula

$$p = \left(30 + \frac{3000}{L}\right)\left(\frac{55 - w}{50}\right)$$ in which

p = live load per square foot (maximum, 60 lbs. per sq. ft.).
L = loaded length of sidewalk in feet.
w = width of sidewalk in feet.

Pedestrian bridges shall be designed for a live load of 85 pounds per sq. ft. of walkway area.

In calculating stresses in structures which support cantilevered sidewalks, the sidewalk shall be considered as fully loaded on only one side of the structure if this condition produces maximum stress.

B. CURB LOADING

Curbs shall be designed to resist a lateral force of not less than 500 pounds per linear foot of curb, applied at the top of the curb, or at an elevation 10 inches above the floor if the curb is higher than 10 inches.

Where sidewalk, curb and traffic rail form an integral system, the traffic railing loading shall apply and stresses in curbs computed accordingly.

C. RAILING LOADING

1. TRAFFIC RAILING

Rail members and parapets shall be designed for a transverse load (P) of 10,000 lbs. or C times P divided between the various members that are centered 15 inches, or more, above the bridge floor (or top of curb wider than 6 inches) as shown in Figure 1.1.9.

All members that have this transverse load distributed to them shall have their roadside faces within 1" of a common vertical plane. Rail members offset more than 1" back of this plane or centered less than 15 inches above the bridge floor (or top of curb wider than 6 inches) and required because of the spacing requirements of Article 1.1.9(A) shall be designed for a transverse load equal to that applied to adjacent traffic rails, except that this loading need not exceed 5,000 lbs.

The handrail members of combination railings shall be designed for a moment at center of panel and at posts of $0.1wL^2$. L is the post spacing.

Each attachment of a rail to a post shall be designed for vertical loads applied upward and downward, but not simultaneously, equal to P'/4 applied at the center line of the rail. Each rail attachment shall also be designed to resist an inward transverse load equal to 1/4 the transverse rail design load.

Posts shall be designed for the transverse loading indicated in Figure 1.1.9, plus simultaneous longitudinal loading of 1/2 this amount. * When the tensile strength of the rail members is maintained through a series of post spaces, the longitudinal loading may be divided among as many as four posts in this continuous length.

Each traffic post shall also be designed to resist an independently applied roadward load equal to 1/4 the outward transverse load.

The transverse force on concrete parapet walls shall be spread over a longitudinal length of 5 feet.

Railing loads shall be applied to the supporting slab in accordance with Article 1.3.2 (H) (2). Railing and wheel loads are not to be applied simultaneously.

* The designer is alerted to the possibility of heavy loads being applied at higher lever arms than normally encountered. Posts which have rails above the minimum traffic rail height of 2'-3" may have to be investigated for possible impact by vehicles with high centers of gravity such as tractor-trailers.

2. PEDESTRIAN RAILING

The minimum design loading for pedestrian railing shall be w=50 lbs. per lin. ft. acting simultaneously transversely and vertically on each longitudinal member. Rail members located more than 5 ft. 0 in. above the walkway are excluded from these requirements.

Posts shall be designed for a transverse load of W1 acting at the center of gravity of the upper rail, or for high rails, at 5 ft. 0 in. maximum above the walkway.

d. DESIGN

Railing shall be designed by the elastic method to the allowable stresses for the appropriate material. For aluminum alloys 5154-H38, 6061-T6, 6063-T6, 6005-T5, and 6351-T5. The design stresses given in Tables 3,3,7, 8, 9 and 10 of the April 1969, "Specifications For Aluminum Bridge and Other Highway Structures" published by the Aluminum Association shall be used. For Alloy A344-T4, 35% of the values listed in Table 3,3,8, and for Alloys A3560T61 and 356-T6, T7, 45% of the values listed in Table 3,3,8 shall be used for design. Aluminum railings shall be fabricated and built in accordance with the provisions of Section 6 of the above publication for riveted and bolted fabrication, and in accordance with Section 10 of the 1968 "Specifications For The Design And Construction of Structural Supports For Highway Signs" for welded fabrication.

The allowable unit stresses for steel shall be as given by the AASHO "Standard Specifications For Highway Bridges" except as modified by "Section 6, Unit Stresses" of the AASHO "Specifications For The Design and Construction of Structural Supports For Highway Signs."

1.2.12-IMPACT

LIVE LOAD STRESSES PRODUCED BY H OR HS LOADINGS SHALL BE INCREASED FOR ITEMS IN GROUP A BY ALLOWANCE AS STATED HEREIN FOR DYNAMIC, VIBRATORY AND IMPACT EFFECTS. IMPACT SHALL NOT BE APPLIED TO ITEMS IN GROUP B.

A. GROUP A *****

(1) SUPERSTRUCTURE, INCLUDING STEEL OR CONCRETE SUPPORTING COLUMNS, STEEL TOWERS, LEGS OF RIGID FRAMES AND GENERALLY THOSE PORTIONS OF THE STRUCTURE WHICH EXTEND DOWN TO THE MAIN FOUNDATION.

(2) THE PORTION ABOVE THE GROUND LINE OF CONCRETE OR STEEL PILES WHICH ARE RIGIDLY CONNECTED TO THE SUPERSTRUCTURE AS IN RIGID FRAME OR CONTINUOUS DESIGNS.

(3) COLUMNS AND CAP BEAMS OF PIERS. ("L" SHALL BE TAKEN AS THE LENGTH OF BOTH SPANS SUPPORTED BY THE PIER.)

B. GROUP B

(1) ABUTMENTS, RETAINING WALLS, SOLID PIERS AND PILES.
(2) FOUNDATION PRESSURES AND FOOTINGS.
(3) TIMBER STRUCTURES.
(4) SIDEWALK LOADS.
(5) CULVERTS AND STRUCTURES HAVING COVER OF 3 FEET OR MORE.

C. IMPACT FORMULA

THE AMOUNT OF THIS ALLOWANCE OR INCREMENT IS EXPRESSED AS A FRACTION OF LIVE LOAD STRESS, AND SHALL BE DETERMINED BY THE FORMULA:

$$I = \frac{50}{L + 125} \quad \text{IN WHICH}$$

I = IMPACT FRACTION (MAXIMUM 30 PER CENT)
L = LENGTH IN FEET OF THE PORTION OF THE SPAN WHICH IS LOADED TO PRODUCE THE MAXIMUM STRESS IN THE MEMBER.

FOR UNIFORMITY OF APPLICATION THE LOADED LENGTH "L" SHALL BE ESPECIALLY CONSIDERED AS FOLLOWS:

FOR ROADWAY FLOORS, USE THE DESIGN SPAN LENGTH.

FOR TRANSVERSE MEMBERS, SUCH AS FLOOR BEAMS, USE THE SPAN LENGTH OF MEMBER CENTER TO CENTER OF SUPPORTS.

SECTION 3 DISTRIBUTION OF LOADS

1.3.1-DISTRIBUTION OF WHEEL LOADS TO STRINGERS, LONGITUDINAL BEAMS AND FLOOR BEAMS *

A. POSITION OF LOADS FOR SHEAR

IN CALCULATING END SHEARS AND END REACTIONS IN TRANSVERSE FLOOR BEAMS AND LONGITUDINAL BEAMS AND STRINGERS, NO LONGITUDINAL DISTRIBUTION OF THE WHEEL LOAD SHALL BE ASSUMED FOR THE WHEEL OR AXLE LOAD ADJACENT TO THE END AT WHICH THE STRESS IS BEING DETERMINED.

LATERAL DISTRIBUTION OF THE WHEEL LOAD SHALL BE THAT PRODUCED BY ASSUMING THE FLOORING TO ACT AS A SIMPLE SPAN BETWEEN STRINGERS OR BEAMS. FOR LOADS IN OTHER POSITIONS ON THE SPAN, THE DISTRIBUTION FOR SHEAR SHALL BE DETERMINED BY THE METHOD PRESCRIBED FOR MOMENT, EXCEPT THAT THE CALCULATION OF HORIZONTAL SHEAR IN RECTANGULAR TIMBER BEAMS SHALL BE IN ACCORDANCE WITH ARTICLE 1.10.2.

B. BENDING MOMENT IN STRINGERS AND LONGITUDINAL BEAMS

IN CALCULATING BENDING MOMENTS IN LONGITUDINAL BEAMS OR STRINGERS, NO LONGITUDINAL DISTRIBUTION OF THE WHEEL LOADS SHALL BE ASSUMED. THE LATERAL DISTRIBUTION SHALL BE DETERMINED AS FOLLOWS:

1. INTERIOR STRINGERS AND BEAMS *****

THE LIVE LOAD BENDING MOMENT FOR EACH INTERIOR STRINGER SHALL BE DETERMINED BY APPLYING TO THE STRINGER THE FRACTION OF A WHEEL LOAD (BOTH FRONT AND REAR) DETERMINED BY THE FOLLOWING TABLE:

* <u>Provisions in this article shall not apply to orthotropic-deck bridges.</u>

KIND OF FLOOR	BRIDGE DESIGNED FOR ONE TRAFFIC LANE	BRIDGE DESIGNED FOR TWO OR MORE TRAFFIC LANES
TIMBER		
PLANK	S/4.0	S/3.75
STRIP 4IN. THICK OR MULTIPLE LAYER FLOORS OVER 5IN. THICK	S/4.5	S/4.0
STRIP 6IN. OR MORE THICK	S/5.0 IF S EXCEEDS 5 FT. FOLLOW NOTE 2 BELOW	S/4.25 IF S EXCEEDS 6.5 FT. FOLLOW NOTE 2 BELOW
CONCRETE:		
ON STEEL OR PRECAST CONCRETE STRINGERS	S/7.0 IF S EXCEEDS 10 FT. FOLLOW NOTE 2 BELOW	S/5.5 IF S EXCEEDS 14 FT. FOLLOW NOTE 2 BELOW
ON CONCRETE T-BEAMS	S/6.5 IF S EXCEEDS 6 FT. FOLLOW NOTE 2 BELOW	S/6.0 IF S EXCEEDS 10 FT. FOLLOW NOTE 2 BELOW
ON TIMBER STRINGERS	S/6.0 IF S EXCEEDS 6 FT. FOLLOW NOTE 2 BELOW	S/5.0 IF S EXCEEDS 10 FT. FOLLOW NOTE 2 BELOW
ON STEEL BOX GIRDERS - (SEE ART. 1.7.104)		
On Prestressed Concrete Spread Box Beams. . . .(See Article 1.6.24(A).		
CONCRETE BOX GIRDERS (SEE NOTE 1 - BELOW)	S/8.0 FOLLOW NOTE 2 BELOW	S/7.0 FOLLOW NOTE 2 BELOW
STEEL GRID:		
(LESS THAN 4 IN. THICK)	S/4.5	S/4.0
(4 IN. OR MORE)	S/6.0 IF S EXCEEDS 6.0 FT. FOLLOW NOTE 2 BELOW	S/5.0 IF S EXCEEDS 10.5 FT. FOLLOW NOTE 2 BELOW

S=AVERAGE STRINGER SPACING IN FEET.

NOTE 1 THE SIDEWALK LIVE LOAD (SEE ARTICLE 1.2.11) SHALL BE OMITTED FOR INTERIOR AND EXTERIOR BOX GIRDERS DESIGNED IN ACCORDANCE WITH THE WHEEL LOAD DISTRIBUTION INDICATED HEREIN.

NOTE 2 IN THIS CASE THE LOAD ON EACH STRINGER SHALL BE THE REACTION OF THE WHEEL LOADS, ASSUMING THE FLOORING BETWEEN THE STRINGERS TO ACT

1.3.5-DISTRIBUTION OF LOADS AND DESIGN OF COMPOSITE WOOD-CONCRETE MEMBERS

A. DISTRIBUTION OF CONCENTRATED LOADS FOR BENDING MOMENT AND SHEAR

FOR FREELY SUPPORTED OR CONTINUOUS SLAB SPANS OF COMPOSITE WOOD-CONCRETE CONSTRUCTION THE WHEEL LOADS SHALL BE DISTRIBUTED OVER A TRANSVERSE WIDTH OF 5 FEET FOR BENDING MOMENT AND A WIDTH OF 4 FEET FOR SHEAR.

FOR COMPOSITE T-BEAMS OF WOOD AND CONCRETE THE EFFECTIVE FLANGE WIDTH SHALL NOT EXCEED THAT GIVEN IN ARTICLE 1.7.99. SHEAR CONNECTORS SHALL BE CAPABLE OF RESISTING BOTH VERTICAL AND HORIZONTAL MOVEMENT.

B. DISTRIBUTION OF BENDING MOMENTS IN CONTINUOUS SPANS

BOTH POSITIVE AND NEGATIVE MOMENTS SHALL BE DISTRIBUTED IN ACCORDANCE WITH THE FOLLOWING TABLE:

MAXIMUM BENDING MOMENTS-PER CENT OF SIMPLE SPAN MOMENT

SPAN	MAXIMUM UNIFORM DEAD LOAD MOMENTS				MAXIMUM LIVE LOAD MOMENTS			
	WOOD SUBDECK		COMPOSITE SLAB		CONCENTRATED LOAD		UNIFORM LOAD	
	POS.	NEG.	POS.	NEG.	POS.	NEG.	POS.	NEG.
INTERIOR	50	50	55	45	75	25	75	55
END	70	60	70	60	85	30	85	65
2-SPAN*	65	70	60	75	85	30	80	75

*CONTINUOUS BEAM OF 2 EQUAL SPANS.

IMPACT SHOULD BE CONSIDERED IN COMPUTING STRESSES FOR CONCRETE AND STEEL, BUT NEGLECTED FOR WOOD.

C. DESIGN

THE COMBINATION IN A STRUCTURAL MEMBER OF TWO ELEMENTS HAVING DIFFERENT MECHANICAL PROPERTIES REQUIRES THE FORMULATION OF A DESIGN PREMISE. SUCH A FORMULATION AS FOLLOWS IS BASED ON THE ELASTIC PROPERTIES OF THE MATERIALS:

EC/EW= 1 FOR SLAB IN WHICH THE NET CONCRETE THICKNESS IS LESS THAN HALF THE OVERALL DEPTH OF THE COMPOSITE SECTION

EC/EW= 2 FOR SLAB IN WHICH THE NET CONCRETE THICKNESS IS A LEAST HALF THE OVERALL DEPTH OF THE COMPOSITE SECTION

ES/EW= 18.75 (FOR DOUGLAS FIR AND SOUTHERN PINE)
IN WHICH

EC= MODULUS OF ELASTICITY OF CONCRETE

EW= MODULUS OF ELASTICITY OF WOOD

ES= MODULUS OF ELASTICITY OF STEEL

1.3.6-DISTRIBUTION OF WHEEL LOADS ON STEEL GRID FLOORS *****

A. GENERAL *

THE GRID FLOOR SHALL BE DESIGNED AS CONTINUOUS, SIMPLE SPAN MOMENTS MAY BE USED AND REDUCED AS PROVIDED IN ARTICLE 1.3.2.

THE FORMULAS FOR DISTRIBUTION OF LOADS PROVIDED HEREIN ARE BASED UPON THERE BEING ADEQUATE TRANSFER OF THE LOAD NORMAL TO THE MAIN ELEMENTS, REINFORCEMENT FOR THIS PURPOSE SHALL CONSIST OF TRANSVERSE BARS OR SHAPES WELDED TO THE MAIN STEEL. THE STRENGTH AND DETAILS OF THE TRANSVERSE REINFORCEMENT SHALL MEET WITH THE APPROVAL OF THE ENGINEER.

A WHEEL LOAD SHALL BE DISTRIBUTED, NORMAL TO THE MAIN BARS, OVER A WIDTH EQUAL TO 1-1/4 INCHES PER TON OF AXLE LOAD PLUS TWICE THE DISTANCE CENTER TO CENTER OF MAIN BARS. THE PORTION OF THE LOAD ASSIGNED TO EACH MAIN BAR SHALL BE APPLIED TO THE BAR UNIFORMLY OVER A LENGTH EQUAL TO THE REAR TIRE WIDTH (20 INCHES FOR H20, 15 INCHES FOR H15).

THE STRENGTH OF THE SECTION SHALL BE DETERMINED BY THE MOMENT OF INERTIA METHOD. THE ALLOWABLE STRESSES SHALL BE AS SET FORTH IN ARTICLE 1.7.1

* Provisions in this article shall not apply to orthotropic-deck bridges.

The vertical or face walls of counterforted and buttressed walls shall be designed as fixed or continuous beams. The face walls shall be securely anchored to the supporting counterforts or buttresses by means of adequate reinforcement.

D. COUNTERFORTS AND BUTTRESSES

Counterforts shall be designed as T-Beams, Buttresses shall be designed as rectangular beams. In connection with the main tension reinforcement of counterforts there shall be a system of horizontal and vertical bars or stirrups to effectively anchor the face walls and base slab. These stirrups shall be anchored as near the outside faces of the face walls, and as near the bottom of the base slab as practicable.

E. REINFORCEMENT FOR TEMPERATURE *****

Exposed faces of abutments and walls, not otherwise reinforced, shall be reinforced with a minimum of number 5 bars at 1 foot horizontally and number 5 bars at 2 feet vertically, to resist the formation of temperature and shrinkage cracks.

F. EXPANSION AND CONTRACTION JOINTS *****

Vertical contraction joints will be required at thirty-foot intervals in all retaining walls, and wingwalls more than sixty feet long. These contraction joints shall not extend through the footing.

Expansion joints will be required at ninety-foot (90') intervals in all retaining walls and wingwalls more than one hundred and eighty feet (180') long. These expansion joints shall extend through the footing.

G. DRAINAGE *****

The filling material behind all retaining walls shall be effectively drained and weepholes shall be placed at a maximum spacing of 30 feet. In counterforted walls there shall be at least one weephole for each pocket formed by the counterforts.

1.4.9-PIERS

A. GENERAL *****

Piers shall be designed to withstand the dead and live loads superimposed thereon; wind pressures acting on the pier and superstructure; the forces due to stream current, floating ice and drift; and longitudinal forces. Piers shall be made solid to a point two feet above ordinary high water.

B. PIER NOSE

In streams carrying ice or drift, the pier nose shall be designed as an ice breaker. When a steel angle or other metal nosing is used it shall be effectively secured to the masonry by means of suitable anchors.

C. REINFORCEMENT FOR TEMPERATURE-SOLID PIERS *****

Exposed faces of abutments and walls, not otherwise reinforced, shall be reinforced with a minimum of number 5 bars at 1 foot horizontally and number 5 bars at 2 feet vertically, to resist the formation of temperature and shrinkage cracks.

1.4.10-TUBULAR STEEL PIERS

A. USE

Preferably, tubular steel piers shall not be used and they shall never be used in locations where they will be subjected to lateral earth pressure. In special cases their use may be permitted, in which cases the following requirements shall apply:

B. DEPTH

The general requirements governing the depths of foundations as above set forth shall govern in the case of tubular steel piers except that steel tubes resting upon gravel foundation without piling shall in no case be carried to a depth less than 8 feet below the permanent bed of the stream and to such additional depth as may be necessary to eliminate all danger of undermining.

C. PILING

Piles used in connection with tubular piers shall extend into the concrete filling a sufficient distance to thoroughly brace the tubes. In general, these piles shall extend not less than 6 to 8 feet above the bottom of the concrete.

D. DIMENSIONS OF SHELL *****

The minimum thickness of the metal in the shells of tubular piers shall be 5/16 inch. This thickness shall be increased where necessary to secure strength and rigidity for placing the shell. In all cases the pier shall be designed for safe pile or soil bearing values as specified herein, but when the diameter required by these values is greater than that required for the superstructure bearing, the diameter may be reduced at any splice point. The minimum diameter of steel cylinders used for piers shall be 30 inches.

E. SPLICES AND JOINTS

All horizontal joints shall be butt joints. Vertical joints may be lapped if the corners of the plates are properly scarfed. When field splicing is necessary the lower section of the tube shall extend at least 2 feet above the water line when in position.

F. BRACING

Adequate bracing connecting the tubes of cylinder piers shall be provided. In general, this bracing shall consist of a steel or concrete girder diaphragm effectively secured to the tubes. The depth of this diaphragm shall be as great as conditions will permit.

1.6.24--BOX GIRDERS

(A) Lateral Distribution of Loads for Bending Moment

(1) Interior Beams

The live load bending moment for each interior beam in a spread box beam superstructure shall be determined by applying to the beam the fraction (D.F.) of the wheel (both front and rear) determined by the following equation:

$$D.F. = \frac{2N_L}{N_B} + k\frac{S}{L}$$

where N_L = number of design traffic lanes (as defined as N in Art. 1.2.6)

N_B = number of beams ($4 \leq N_B \leq 10$)

S = beam spacing, in feet ($6.75 \leq S \leq 11.00$)

L = span length, in feet

$k = 0.07W - N_L(0.10N_L - 0.26) - 0.20N_B - 0.12$

w = roadway width between curbs, in feet (defined as W_c in Art. 1.2.6) ($32 \leq W \leq 66$)

(2) Exterior Beams

The live load bending moment in the exterior beams shall be determined by applying to the beams the reaction of the wheel loads obtained by assuming the flooring to act as a simple span (of length S) between beams, but shall not be less than $2N_L/N_B$.

(B) EFFECTIVE COMPRESSION FLANGE WIDTH

IN GIRDER AND FLANGE CONSTRUCTION, CONSISTING OF A STEM WITH TOP AND BOTTOM SLAB, EFFECTIVE AND ADEQUATE BOND AND SHEAR RESISTANCE SHALL BE PROVIDED AT THE JUNCTURE OF THE GIRDER AND THE SLAB. THE SLAB MAY THEN BE CONSIDERED AN INTERGRAL PART OF THE GIRDER, BUT ITS EFFECTIVE WIDTH AS A GIRDER FLANGE SHALL NOT EXCEED THE FOLLOWING:

(1) ONE FOURTH OF THE SPAN LENGTH OF THE GIRDER
(2) THE DISTANCE CENTER-TO-CENTER OF GIRDERS
(3) TWELVE TIMES THE LEAST THICKNESS OF THE SLAB PLUS THE WIDTH OF THE GIRDER WEB

FOR GIRDERS HAVING A FLANGE ON ONE SIDE ONLY, THE EFFECTIVE OVERHANGING WIDTH SHALL NOT EXCEED THE FOLLOWING:

(1) ONE TWELFTH OF THE SPAN LENGTH OF THE GIRDER
(2) ONE HALF OF THE CLEAR DISTANCE TO THE NEXT GIRDER
(3) SIX TIMES THE LEAST THICKNESS OF THE SLAB

(C) FLANGE THICKNESS

(1) TOP FLANGE

THE MINIMUM FLANGE THICKNESS SHALL BE 1/16 OF THE CLEAR DISTANCE BETWEEN GIRDERS, OR 6 IN. WHICHEVER IS GREATER, EXCEPT THE MINIMUM THICKNESS MAY BE REDUCED FOR FACTORY PRODUCED PRECAST ELEMENTS TO 5 1/2 IN.

(2) BOTTOM FLANGE

THE MAXIMUM THICKNESS OF THE BOTTOM FLANGE SHALL BE DETERMINED BY MAXIMUM ALLOWABLE UNIT STRESSES AS SPECIFIED IN 1.6.6 BUT IN NO CASE SHALL BE LESS THAN 1/16 OF THE CLEAR SPAN BETWEEN GIRDERS OR 5 1/2 IN., WHICHEVER IS THE GREATER, EXCEPT THE MINIMUM THICKNESS MAY BE REDUCED FOR FACTORY PRODUCED PRECAST ELEMENTS TO 5 IN. ADEQUATE FILLETS SHALL BE PROVIDED AT THE INTERSECT9ONS OF ALL SURFACES WITHIN THE CELL OF A BOX GIRDER EXCEPT AT THE JUNCTION OF WEB AND BOTTOM FLANGE WHERE NONE ARE REQUIRED.

(D) MINIMUM BAR REINFORCEMENT FOR CAST-IN-PLACE POST-TENSIONED BOX GIRDERS

(1) TOP FLANGE

The minimum top flange reinforcement shall be the same as for reinforced concrete box girders.

(2) Bottom Flange

The minimum bottom flange reinforcement shall be the same as for reinforced concrete box girders except the minimum reinforcement shall be 0.3 percent of the flange section.

(E) SHEAR

The horizontal shearing unit stress at the junction of the flange and the monolithic fillet joining it to the girder web shall not exceed $0.015f'c$.
Changes in girder stem thickness shall be tapered for a minimum distance of 12 times the difference in web thickness.

(F) DIAPHRAGMS *****

Diaphragms or spreaders within the precast box beams shall be placed at midspan for spans up to 50 ft.; at quarter points for spans over 50 ft.

Diaphragms or spreaders shall be placed between the girders at intervals not to exceed 80 ft. Diaphragm spacing for curved girders shall be given special consideration.

Table 1.7.3B

	Category, Type and Location of Material	Type of Maximum Stress	100,000 Cycles			500,000 Cycles			2,000,000 Cycles		
			fro	α	k2	fro	α	k2	fro	α	k2
A	Base Metal	(4) Tension or Reversal	60,000	0	1.00	36,000	0	1.00	24,000	0	1.00
B	Base Metal adjacent to friction type fastener	(4) Tension	20,500	1.06	0.55	20,500	0.78	0.55	20,500	0.54	0.55
		(1) Compression	13,300	1.06	—	13,300	0.78	—	13,300	0.54	—
C	Base Metal adjacent to bearing type fastener	(4) Tension or Compression	20,500	0	0.55	17,200	0	0.62	15,000	0	0.67
D	Weld Metal or Base Metal (3) adjacent to Butt Weld	(4) Tension	20,500	0.65	0.55	17,200	0.23	0.62	15,000	0	0.67
		(1) Compression	13,300	0.65	—	10,600	0.23	—	9,000	0	—
E	Flanges with Stud shear connectors	Tension	20,500	1.06	0.55	16,500	0	0.65	11,500	0	0.75
F	Base Metal adjacent to or connected by fillet (2) or plug welds	(5) Tension or Compression	18,000	0	1.00	12,000	0	1.00	9,000	0	1.00
G	Weld Metal	(4) Shear	12,000	0.78	0.50	10,800	0.36	0.55	9,000	0	0.62
H	Friction type fastener	Shear	Fv	0	0	Fv	0	0	Fv	0	0
I	Bearing type fastener	Shear	Fv	0	0.50	13,500	0	0.50	11,200	0	0.50

Notes to Table 1.7.3B-

(1) Use the formula:

$$Fr = \frac{0.55\ Fy}{1 - \left(\frac{0.55\ Fy}{k_1 f_{ro}} - 1\right) R}$$

(2) The usual continuous fillet welded flange-web connections and similar connections shall be governed by Category J.

(3) Base Metal adjacent to longitudinal butt welds and the welds and the weld metal in longitudinal butt welds shall be governed by Category J.

(4) See Graphs on Figure 1.7.3A.

(5) See Graphs on Figure 1.7.3B.

(6) The Category G, "Weld Metal" in Table 1.7.3B does not apply in this case. Where the shear stress in the welds exceeds 15 ksi, $F_r^{\,2} \geq F_b^{\,2} + 3F_v^{\,2}$ in which F_b and F_v are the maximum bending and shear stresses in the weld and F_r is the allowable fatigue stress for Category J, "Base Metal adjacent to continuous flange-web fillet welds". Intermittent fillet welds shall not be permitted.

ASTM DESIGNATION WITH GRADE OR CLASS		A-235 CLASS F	A-235 CLASS G	A-237 CLASS A
SIZE LIMITATIONS			12 IN. DIA. OR LESS	OVER 12 IN. DIA.
MINIMUM YIELD POINT, PSI	F_y	37,500	50,000	50,000
STRESS IN EXTREME FIBER	0.80 F_y	30,000	40,000	40,000
SHEAR	0.40 F_y	15,000	20,000	20,000
BEARING ON PINS NOT SUBJECT TO ROTATION	0.80 F_y	30,000	40,000	40,000
BEARING ON PINS SUBJECT TO ROTATION	0.40 F_y	15,000	20,000	20,000

1.7.5-BOLTS

IN PROPORTIONING BOLTS, THE NOMINAL DIAMETER SHALL BE USED, EXCEPT AS OTHERWISE NOTED.

THE EFFECTIVE BEARING AREA OF A BOLT SHALL BE ITS DIAMETER MULTIPLIED BY THE THICKNESS OF THE METAL ON WHICH IT BEARS. IN METAL LESS THAN 3/8 INCH THICK, COUNTERSUNK BOLTS, TURNED BOLTS, OR RIBBED BOLTS SHALL NOT BE ASSUMED TO CARRY STRESS. IN METAL 3/8 INCH THICK AND OVER, ONE-HALF THE DEPTH OF COUNTERSINK SHALL BE OMITTED IN CALCULATING THE BEARING AREA.

ALLOWABLE UNIT STRESSES IN POUNDS PER SQUARE INCH FOR BOLTS SHALL BE AS LISTED IN THE TABLE BELOW:

TYPE OF BOLT	TENSION	BEARING	SHEAR: FRICTION TYPE CONNECTION	SHEAR: BEARING TYPE CONNECTION
(A) LOW CARBON STEEL BOLTS				
TURNED BOLTS (ASTM A-307) AND RIBBED BOLTS	13,500*	20,000		11,000
(B) HIGH STRENGTH BOLTS				
HIGH STRENGTH STEEL BOLTS (ASTM A-325)	36,000	40,000**	13,500	20,000***

* BASED ON AREA AT THE ROOT OF THREAD.

** DOES NOT APPLY TO FRICTION TYPE CONNECTIONS.

*** THE ALLOWABLE SHEAR VALUE OF BOLTS FOR BEARING TYPE CONNECTIONS IN STEEL WITH A YIELD POINT LESS THAN 42,000 PSI SHALL BE REDUCED BY 20% WHEN THE END OF THE SPLICE MATERIAL IS MORE THAN 24 INCHES FROM THE END OF THE CONNECTED MEMBER, AS MEASURED ALONG THE GAGE LINE OF THE BOLTS.

ALL BOLTS EXCEPT HIGH STRENGTH BOLTS, SHALL HAVE SINGLE SELF-LOCKING NUTS OR DOUBLE NUTS.

JOINTS REQUIRED TO RESIST SHEAR BETWEEN THEIR CONNECTED PARTS ARE DESIGNATED AS EITHER FRICTION TYPE OR BEARING TYPE CONNECTIONS. SHEAR CONNECTIONS SUBJECTED TO STRESS REVERSAL SHALL BE FRICTION TYPE EXCEPT FOR SECONDARY MEMBERS.

Bolts in girder field splices shall be friction type.

ASTM A-307 Bolts shall not be used in structural connections.

Bolted bearing type connections using high strength bolts shall be used for connections of secondary members.

In bearing type connections, pull-out shear in a plate should be investigated between the end of the plate and the end row of fasteners.

For combined shear and tension in friction type joints where applied forces reduce the total clamping force on the friction plane, the allowable unit shearing stress, f_v, in (ASTM A325) high strength bolts shall not exceed the values obtained from the following equation:

$$f_v = 13{,}500 - .22f_t$$

Where f_t = tensile stress due to applied loads

When bearing type connections are subject to both shear and tension, the combined stress shall not exceed values obtained from the following equation:

$$s^2 + (0.555T)^2 = S^2$$

Whre s = the computed unit stress in shear
T = the computed unit stress in tension
S = the allowable unit stress in shear

For secondary members, such as cross frames and lateral bearing enlarged or slotted holes may be used with high strength bolts proportioned to meet the allowable unit stresses given above except as hereinafter restricted:

1. Holes 3/16 inch larger than bolts 7/8 inch and less in diameter, 1/4 inch larger than bolts 1 inch in diameter, and 5/16 inch larger than bolts 1-1/8 inch and greater in diameter may be used in uncoated friction type shear connections provided a hardened washer is inserted under both the head and nut.

2. Slotted holes 1/16 inch wider than the bolt diameter and of a length more than allowed in subparagraph 1 but not more than 2-1/2 times the bolt diameter may be used without regard to direction of loading, in enclosed parts of friction-type shear connections if one-third more bolts are provided than needed to satisfy the design requirements.

3. When enlarged or slotted holes are used, the distances between edges of holes and edges of holes and edges of members shall not be less than that permitted with conventional size holes.

1.7.6 -CAST STEEL, DUCTILE IRON CASTINGS, MALLEABLE CASTINGS AND CAST IRON

A. CAST STEEL AND DUCTILE IRON

FOR CAST STEEL CONFORMING TO SPECIFICATIONS FOR STEEL CASTINGS FOR HIGHWAY BRIDGES, ASTM A 486, MILD-TO-MEDIUM-STRENGTH CARBON-STEEL CASTINGS FOR GENERAL APPLICATION, ASTM A 27, AND CORROSION-RESISTANT IRON-CHROMIUM-NICKEL ALLOY CASTINGS FOR GENERAL APPLICATION, ASTM A 296 AND FOR DUCTILE IRON CASTINGS, ASTM A 536 THE FOLLOWING ALLOWABLE STRESSES IN POUNDS PER SQUARE INCH SHALL BE USED:

ASTM DESIGNATION	A 27 A 486	A 486		A 296	A 536
CLASS OR GRADE	70-36 70	90	120	CA-15	60-40-18
YIELD POINT, MINIMUM, F_y	36,000	60,000	95,000	65,000	40,000
AXIAL TENSION	14,500	22,500	34,000	24,000	16,000
TENSION IN EXTREME FIBER	14,500	22,500	34,000	24,000	16,000
AXIAL COMPRESSION, SHORT COLUMNS	20,000	30,000	45,000	32,000	22,000
COMPRESSION IN EXTREME FIBER	20,000	30,000	45,000	32,000	22,000
SHEAR	9,000	13,500	21,000	14,000	10,000
BEARING, STEEL PARTS IN CONTACT	30,000	45,000	68,000	48,000	33,000
BEARING ON PINS NOT SUBJECT TO ROTATION	26,000	40,000	60,000	43,000	28,000
BEARING ON PINS SUBJECT TO ROTATION (SUCH AS USED IN ROCKERS AND HINGES)	13,000	20,000	30,000	21,500	14,000

WHEN IN CONTACT WITH CASTINGS OR STEEL OR A DIFFERENT YIELD POINT, THE ALLOWABLE UNIT BEARING STRESS OF THE MATERIAL WITH THE LOWER YIELD POINT

SHALL GOVERN. FOR BOLTED CONNECTIONS THE FASTENER SPECIFICATIONS SHALL GOVERN

B. MALLEABLE CASTINGS

FOR MALLEABLE CASTINGS CONFORMING TO SPECIFICATIONS FOR MALLEABLE IRON CASTINGS, ASTM A47, THE FOLLOWING ALLOWABLE STRESSES IN POUNDS PER SQUARE INCH, SHALL BE USED:

TENSION	18,000
BENDING IN EXTREME FIBER	18,000
MODULUS OF ELASTICITY	25,000,000

C. CAST IRON

FOR CAST IRON CASTINGS CONFORMING TO SPECIFICATIONS FOR GRAY IRON CASTINGS, ASTM A48, THE FOLLOWING ALLOWABLE STRESSES IN POUNDS PER SQUARE INCH, SHALL BE USED:

BENDING IN EXTREME FIBER	3,000
SHEAR	3,000
DIRECT COMPRESSION, SHORT COLUMNS	12,000

1.7.7 -BRONZE OR COPPER-ALLOY

BRONZE CASTINGS, ASTM B 22, ALLOYS A OR B OR COPPER-ALLOY PLATES, ASTM B 100, ALLOY NO. 1 SHALL BE SPECIFIED.

THE ALLOWABLE UNIT BEARING STRESS IN POUNDS PER SQUARE INCH ON BRONZE CASTINGS OR COPPER-ALLOY PLATES SHALL BE 2,000.

1.7.8 -BEARING ON MASONRY

THE ALLOWABLE UNIT BEARING STRESS IN POUNDS PER SQUARE INCH, ON THE FOLLOWING TYPES OF MASONRY, SHALL BE:

GRANITE	800
SANDSTONE AND LIMESTONE	400

CONCRETE:

BRIDGE SEATS, UNDER HINGED ROCKERS AND BOLSTERS (NOT SUBJECTED TO HIGH EDGE LOADING BY DEFLECTING BEAM, GIRDER, OR TRUSS) ..	1,000

BRIDGE SEATS, UNDER BEARING PLATES OR NON-HINGED SHOES (SUBJECTED TO HIGH EDGE LOADING BY THE DIRECT BEARING, UPON THE PLATE OR SHOE, OF A DEFLECTING BEAM OR GIRDER),

AVERAGE	700

THE ABOVE BRIDGE SEAT UNIT STRESSES WILL APPLY ONLY WHERE THE EDGE OF THE BRIDGE SEAT PROJECTS AT LEAST 3 INCHES (AVERAGE) BEYOND EDGE OF SHOE OR PLATE. OTHERWISE, THE UNIT STRESSES PERMITTED WILL BE 75 PERCENT OF THE ABOVE AMOUNTS.

DETAILS OF DESIGN

1.7.9 -EFFECTIVE LENGTH OF SPAN

FOR THE CALCULATION OF STRESSES, SPAN LENGTHS SHALL BE ASSUMED AS THE DISTANCE BETWEEN CENTERS OF BEARINGS OR OTHER POINTS OF SUPPORT.

1.7.10 -DEPTH RATIOS

FOR BEAMS OR GIRDERS THE RATIO OF DEPTH TO LENGTH OF SPAN, PREFERABLY SHALL NOT BE LESS THAN 1/25.

FOR COMPOSITE GIRDERS THE RATIO OF THE OVER-ALL DEPTH OF GIRDER (CONCRETE SLAB, PLUS STEEL GIRDER) TO THE LENGTH OF SPAN PREFERABLY SHALL NOT BE LESS THAN 1/25, AND THE RATIO OF DEPTH OF STEEL GIRDER ALONE TO LENGTH OF SPAN SHALL NOT BE LESS THAN 1/30.

FOR TRUSSES THE RATIO OF DEPTH TO LENGTH OF SPAN SHALL NOT BE LESS THAN 1/10.

FOR CONTINUOUS SPAN DEPTH RATIO, THE SPAN LENGTH SHALL BE CONSIDERED AS THE DISTANCE BETWEEN THE DEAD LOAD POINTS OF CONTRAFLECTURE.

1.7.11 -LIMITING LENGTHS OF MEMBERS

FOR COMPRESSION MEMBERS, THE RATIO OF UNSUPPORTED LENGTH TO RADIUS OF GYRATION SHALL NOT EXCEED 120 FOR MAIN MEMBERS, OR THOSE IN WHICH THE MAJOR STRESSES RESULT FROM DEAD OR LIVE LOAD, OR BOTH; AND SHALL NOT EXCEED 140 FOR SECONDARY MEMBERS, OR THOSE WHOSE PRIMARY PURPOSE IS TO BRACE THE STRUCTURE AGAINST LATERAL OR LONGITUDINAL FORCES, OR TO BRACE OR REDUCE THE UNSUPPORTED LENGTH OF OTHER MEMBERS, MAIN OR SECONDARY.

IN DETERMINING THE RADIUS OF GYRATION FOR THE PURPOSE OF APPLYING THE LIMITATIONS OF THE PRECEDING PARAGRAPH, THE AREA OF ANY PORTION OF A MEMBER MAY BE NEGLECTED PROVIDED THAT THE STRENGTH OF THE MEMBER AS CALCULATED WITHOUT USING THE AREA THUS NEGLECTED AND THE STRENGTH OF THE MEMBER AS COMPUTED FOR THE ENTIRE SECTION WITH THE l/r RATIO APPLICABLE THERETO BOTH EQUAL OR EXCEED THE COMPUTED TOTAL STRESS THAT THE MEMBER

MUST SUSTAIN.

THE RADIUS OF GYRATION AND THE EFFECTIVE AREA FOR CARRYING STRESS OF A MEMBER CONTAINING PERFORATED COVER PLATES SHALL BE COMPUTED FOR A TRANSVERSE SECTION THROUGH THE MAXIMUM WIDTH OF PERFORATION. WHEN PERFORATIONS ARE STAGGERED IN OPPOSITE COVER PLATES THE CROSS-SECTIONAL AREA OF THE MEMBER SHALL BE CONSIDERED THE SAME AS FOR A SECTION HAVING PERFORATIONS IN THE SAME TRANSVERSE PLANE.

UNSUPPORTED LENGTH SHALL BE ASSUMED AS FOLLOWS:

FOR THE TOP CHORDS OF HALF-THROUGH TRUSSES, THE LENGTH BETWEEN PANEL POINTS LATERALLY SUPPORTED AS INDICATED UNDER ARTICLE 1.7.86; FOR OTHER MAIN MEMBERS, THE LENGTH BETWEEN PANEL POINT INTERSECTIONS OR CENTERS OF BRACED POINTS OR CENTERS OF END CONNECTIONS; FOR SECONDARY MEMBERS, THE LENGTH BETWEEN THE CENTERS OF THE END CONNECTIONS OF SUCH MEMBER OR CENTERS OF BRACED POINTS.

FOR TENSION MEMBERS, EXCEPT RODS, EYEBARS, CABLES AND PLATES, THE RATIO OF UNSUPPORTED LENGTH TO RADIUS OF GYRATION SHALL NOT EXCEED 200 FOR MAIN MEMBERS, 240 FOR BRACING MEMBERS, AND 140 FOR MAIN MEMBERS SUBJECTED TO REVERSAL OF STRESS.

CROSS FRAME MEMBERS FOR CURVED STRINGERS SHALL BE CONSIDERED MAIN MEMBERS AND SHALL HAVE A MAXIMUM l/r OF 120.

1.7.12 -DEFLECTION *****

THE TERM "DEFLECTION" AS USED HEREIN SHALL BE THE DEFLECTION COMPUTED IN ACCORDANCE WITH THE ASSUMPTION MADE FOR LOADING WHEN COMPUTING THE STRESS IN THE MEMBER.

MEMBERS HAVING SIMPLE OR CONTINUOUS SPANS SHALL BE DESIGNED SO THAT THE DEFLECTION DUE TO LIVE LOAD PLUS IMPACT SHALL NOT EXCEED 1/800 OF THE SPAN, EXCEPT ON BRIDGES IN URBAN AREAS USED IN PART BY PEDESTRIANS WHEREON THE RATIO PREFERABLY SHALL BE 1/1000.

THE DEFLECTION OF CANTILEVER ARMS DUE TO LIVE LOAD PLUS IMPACT SHALL BE LIMITED TO 1/300 OF THE CANTILEVER ARM EXCEPT FOR THE CASE INCLUDING PEDESTRIAN USE, WHERE THE RATIO PREFERABLY SHALL BE 1/375.

WHEN SPANS HAVE CROSS-BRACING OR DIAPHRAGMS SUFFICIENT IN DEPTH OR STRENGTH TO INSURE LATERAL DISTRIBUTION OF LOADS, THE DEFLECTION MAY BE COMPUTED FOR THE STANDARD H OR HS LOADING, CONSIDERING ALL BEAMS OR STRINGERS AS ACTING TOGETHER AND HAVING EQUAL DEFLECTION.

THE MOMENT OF INERTIA OF THE GROSS CROSS-SECTION AREA SHALL BE USED FOR COMPUTING THE DEFLECTIONS OF BEAMS AND GIRDERS. WHEN THE BEAM OR GIRDER IS A PART OF A COMPOSITE MEMBER, THE LIVE LOAD MAY BE CONSIDERED AS ACTING UPON THE COMPOSITE SECTION.

The gross area of each truss member shall be used in computing deflections of trusses. If perforated plates are used, the effective area shall be the net volume divided by the length from center to center of perforations.

The contract plans shall show design deflections for steel, concrete slab and dead load applied after slab is cured, and required camber for dead load and vertical curvature. The effect of pouring sequence on deflections shall be recognized and where necessary, the pouring sequence shall be shown in the plans.

1.7.13 - MINIMUM THICKNESS OF METAL

Structural steel (including bracing, cross frames and all types of gusset plates), except for webs of certain rolled shapes, closed ribs in orthotropic decks, fillers and in railings, shall be not less than 5/16 in. in thickness. The web thickness of rolled beams, channels, or structural tees shall not be less than 0.23 in. The thickness of closed ribs in orthotropic decks shall not be less than 3/16 in.

Where the metal will be exposed to marked corrosive influences, it shall be increased in thickness or specially protected against corrosion.

It should be noted that there are other provisions in this section pertaining to thickness for fillers, segments of compression members, gusset plates, etc. As stated above fillers need not be 5/16 in. min.

For stiffners and outstanding legs of angles, etc., refer to article 1.7.15

For stiffners and other plates refer to "Plate Girders."

For compression members refer to "Trusses."

1.7.14 - EFFECTIVE AREA OF ANGLES AND TEE SECTIONS IN TENSION

The effective area of a single angle tension member, a tee section tension member, or each angle of a double angle tension member in which the shapes are connected back to back on the same side of a gusset plate, shall be assumed as the net area of the connected leg or flange plus one half of the area of the outstanding leg.

If a double angle or tee section tension member is connected with the angles or flanges back to back on opposite sides of a gusset plate, the full net area of the shapes shall be considered as effective.

1.7.15 - OUTSTANDING LEGS OF ANGLES

The widths of outstanding legs of angles in compression (except

WHERE REINFORCED BY PLATES) SHALL NOT EXCEED THE FOLLOWING:

IN MAIN MEMBERS CARRYING AXIAL STRESS, 12 TIMES THE THICKNESS.
IN BRACING AND OTHER SECONDARY MEMBERS, 16 TIMES THE THICKNESS.

FOR OTHER LIMITATIONS SEE ARTICLE 1.7.88.

1.7.16 -EXPANSION AND CONTRACTION

THE DESIGN SHALL BE SUCH AS TO ALLOW FOR TOTAL THERMAL MOVEMENT AT THE RATE OF 1 1/4 IN. IN 100 FEET. PROVISIONS SHALL BE MADE FOR CHANGES IN LENGTH OF SPAN RESULTING FROM LIVE LOAD STRESSES. IN SPANS MORE THAN 300 FEET LONG, ALLOWANCE SHALL BE MADE FOR EXPANSION AND CONTRACTION IN THE FLOOR. THE EXPANSION END SHALL BE SECURED AGAINST LATERAL MOVEMENT.

1.7.17 -COMBINED STRESSES

ALL MEMBERS SUBJECT TO COMBINED BENDING AND DIRECT STRESSES SHALL BE PROPORTIONED FOR THE MAXIMUM UNIT STRESS SPECIFIED IN APPENDIX C. WHEN BENDING STRESSES ARE INDUCED BY THE COMPONENT OF EXTERNALLY APPLIED LOADS ACTING PERPENDICULAR TO THE AXIS OF THE MEMBER,"a" SHALL BE ASSUMED EQUAL TO +1.

1.7.18 -ECCENTRIC CONNECTIONS

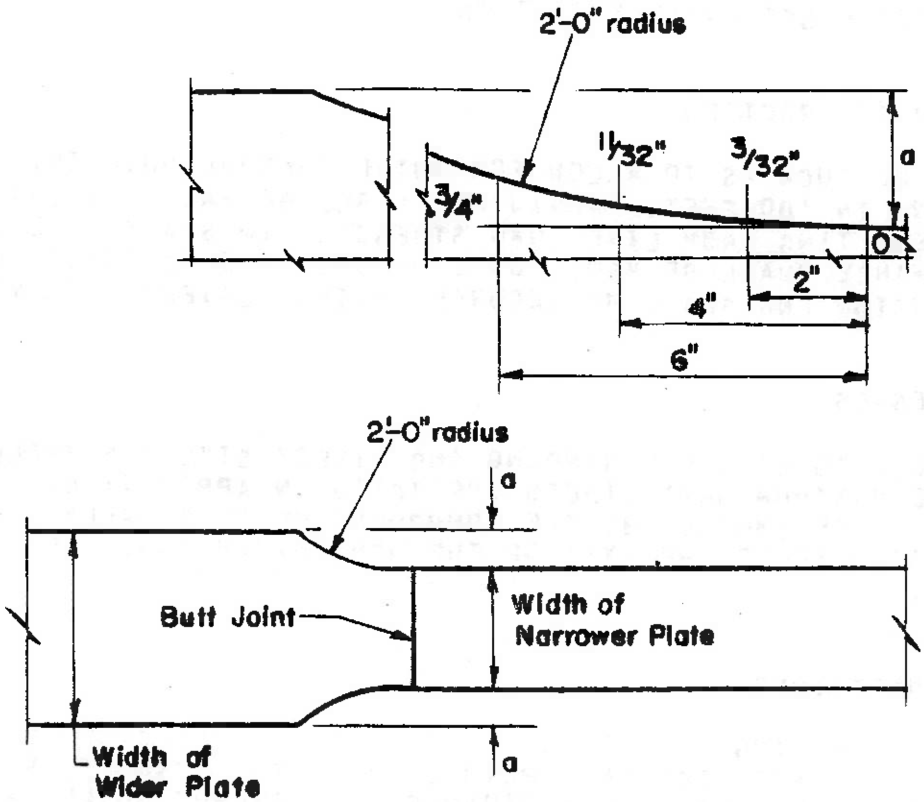
MEMBERS, INCLUDING BRACING, PREFERABLY SHALL BE SO CONNECTED THAT THEIR GRAVITY AXES WILL INTERSECT IN A POINT. ECCENTRIC CONNECTIONS SHALL BE AVOIDED, IF PRACTICABLE, BUT IF UNAVOIDABLE THE MEMBERS SHALL BE SO PROPORTIONED THAT THE COMBINED FIBER STRESSES WILL NOT EXCEED THE ALLOWED AXIAL STRESS.

1.7.19 -FIELD SPLICES AND CONNECTIONS *****

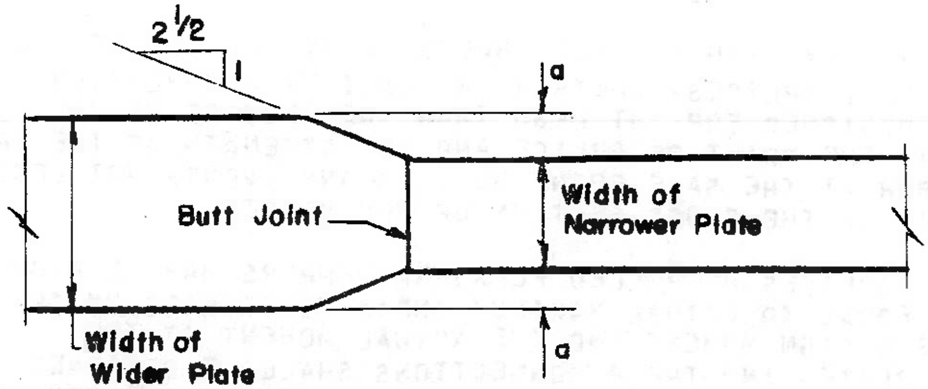
SPLICES MAY BE MADE BY HIGH STRENGTH BOLTS OR BY THE USE OF FULL PENETRATION BUTT WELDS. SPLICES, WHETHER IN TENSION, COMPRESSION, BENDING OR SHEAR, SHALL BE DESIGNED FOR NOT LESS THAN THE AVERAGE OF THE CALCULATED STRESS AT THE POINT OF SPLICE AND THE STRENGTH OF THE GROSS SECTION OF THE MEMBER AT THE SAME POINT BUT, IN ANY EVENT, NOT LESS THAN 75 % OF THE STRENGTH OF THE GROSS SECTION OF THE MEMBER.

AS AN ALTERNATE, SPLICES OF ROLLED FLEXURAL MEMBERS MAY BE PROPORTIONED FOR A SHEAR EQUAL TO ACTUAL MAXIMUM SHEAR MULTIPLIED BY TH RATIO OF THE SPLICE DESIGN MOMENT AND THE ACTUAL MOMENT AT THE SPLICE. WEB SPLICE PLATES AND THEIR CONNECTIONS SHALL BE DESIGNED FOR THE PORTION OF THE DESIGN MOMENT RESISTED BY THE WEB AND FOR THE MOMENT DUE TO THE ECCENTRICITY OF SHEAR INTRODUCED BY THE SPLICE CONNECTION. FLANGE SPLICE PLATES NEED BE DESIGNED ONLY FOR THE PORTION OF THE DESIGN MOMENT NOT RESISTED BY THE WEB.

FIGURE 1.7.19



(a) 2'–0" Radius Transition



(b) Straight Tapered Transition

SPLICE DETAILS

IN THE DESIGN OF SPLICES, DUE CONSIDERATION SHALL BE GIVEN TO FATIGUE.

WEB PLATES SHALL BE SPLICED SYMMETRICALLY BY PLATES ON EACH SIDE. THE SPLICE PLATES FOR SHEAR SHALL EXTEND THE FULL DEPTH OF THE GIRDER BETWEEN FLANGES. IN THE SPLICE THERE SHALL BE NOT LESS THAN 2 ROWS OF BOLTS ON EACH SIDE OF THE JOINT.

SPLICES IN TRUSS CHORDS AND COLUMNS SHALL BE LOCATED AS NEAR TO THE PANEL POINTS AS PRACTICABLE AND USUALLY ON THAT SIDE WHERE THE SMALLER STRESS OCCURS. THE ARRANGEMENT OF SPLICE ELEMENTS SHALL BE SUCH AS TO MAKE PROPER PROVISION FOR THE STRESSES, BOTH AXIAL AND BENDING, IN THE COMPONENT PARTS OF THE MEMBERS SPLICED.

IN CONTINUOUS SPANS SPLICES PREFERABLY SHALL BE MADE AT OR NEAR POINTS OF CONTRAFLEXURE.

THE NET SECTION OF A BOLTED TENSION SPLICE IS THE SUM OF THE NET SECTIONS OF ITS COMPONENT PARTS. THE NET SECTION OF A PART IS THE PRODUCT OF THE THICKNESS OF THE PART MULTIPLIED BY ITS LEAST NET WIDTH.

THE NET WIDTH FOR ANY CHAIN OF HOLES EXTENDING PROGRESSIVELY ACROSS THE PART SHALL BE OBTAINED BY DEDUCTING FROM THE GROSS WIDTH THE SUM OF THE DIAMETERS OF ALL THE HOLES IN THE CHAIN AND ADDING, FOR EACH GAGE SPACE IN THE CHAIN, THE QUANTITY:

$$S^2 / 4g$$

WHERE S = PITCH OF ANY TWO SUCCESSIVE HOLES IN THE CHAIN.

g = GAGE OF THE SAME HOLES.

THE NET SECTION OF THE PART IS OBTAINED FROM THE CHAIN WHICH GIVES THE LEAST NET WIDTH.

AT A SPLICE THE TOTAL STRESS IN THE MEMBER BEING SPLICED IS TRANSFERRED BY FASTENERS TO THE SPLICE MATERIAL.

WHEN DETERMINING THE UNIT STRESS ON ANY LEAST NET WIDTH OF EITHER SPLICE MATERIAL OR MEMBER BEING SPLICED, THE AMOUNT OF THE STRESS PREVIOUSLY TRANSFERRED BY FASTENERS ADJACENT TO THE SECTION BEING INVESTIGATED SHALL BE CONSIDERED IN DETERMINING THE UNIT STRESS ON THE NET SECTION.

The diameter of the hole shall be taken as 1/8 inch greater than the nominal diameter of the rivet or high strength bolt unless larger holes are permitted in accordance with Article 1.7.5.

1.7.20 STRENGTH OF CONNECTIONS

EXCEPT AS OTHERWISE PROVIDED HEREIN, CONNECTIONS SHALL BE DESIGNED

FOR THE AVERAGE OF THE CALCULATED STRESS AND THE STRENGTH OF THE GROSS SECTION OF THE MEMBER, BUT THEY SHALL BE DESIGNED FOR NOT LESS THAN 75 PERCENT OF THE STRENGTH OF THE GROSS SECTION OF THE MEMBER.

CONNECTIONS SHALL BE MADE SYMMETRICAL ABOUT THE AXIS OF THE MEMBERS INSOFAR AS PRACTICABLE. CONNECTIONS, EXCEPT FOR HANDRAILS, SHALL CONTAIN NOT LESS THAN TWO FASTENERS OR EQUIVALENT WELD.

1.7.21!-DIAPHRAGMS, CROSS FRAMES AND LATERAL BRACING

ROLLED BEAM AND PLATE GIRDER SPANS SHALL BE PROVIDED WITH CROSS FRAMES OR DIAPHRAGMS AT EACH END AND INTERMEDIATE CROSS FRAMES OR DIAPHRAGMS SPACED AT INTERVALS NOT TO EXCEED 25 FEET. CROSS FRAMES SHALL BE AS DEEP AS PRACTICABLE. DIAPHRAGMS SHALL BE AT LEAST 1/3 AND PREFERABLY 1/2 THE GIRDER DEPTH. END CROSS FRAMES OR DIAPHRAGMS SHALL BE PROPORTIONED TO ADEQUATELY TRANSMIT ALL THE LATERAL FORCES TO THE BEARINGS. SPECIAL CONSIDERATION SHALL BE GIVEN TO THE DESIGN OF CROSS FRAMES USED ON HORIZONTALLY CURVED STEEL GIRDER BRIDGES. THESE CROSS FRAMES SHALL BE DESIGNED AS MAIN MEMBERS WITH ADEQUATE PROVISIONS FOR TRANSFER OF LATERAL FORCES FROM THE GIRDER FLANGES.

ON SPANS 125 FEET OR LONGER OR ON SPANS WITH CURVED SUPPORTING MEMBERS, HAVING A CONCRETE FLOOR OR OTHER FLOOR OF EQUAL RIGIDITY, WHICH IS ADEQUATELY ATTACHED TO THE TOP FLANGES, ONE PLANE OR SYSTEM OF LATERAL BRACING SHALL BE PROVIDED NEAR THE BOTTOM FLANGE. SPANS WITH TIMBER OR OTHER NON-RIGID FLOORING SHALL HAVE A BOTTOM LATERAL SYSTEM FOR SPANS LONGER THAN 40 FEET AND TOP AND BOTTOM LATERAL SYSTEMS FOR SPANS OF 125 FEET OR LONGER. THE LATERAL BRACING SHALL BE PLACED IN ALTERNATE BAYS OF SPANS WITH CURVED SUPPORTING MEMBERS AND IN OUTSIDE BAYS OF SPANS WITH STRAIGHT SUPPORTING MEMBERS. CROSS FRAMES OR DIAPHRAGMS SHALL BE PLACED IN ALL BAYS. FOR CONTINUOUS STRUCTURES WHERE THE LENGTH BETWEEN DEAD LOAD INFLECTION POINTS IS 125 FEET OR LONGER, A BOTTOM LATERAL SYSTEM SHALL BE PROVIDED FOR THE ENTIRE STRUCTURE.

WHERE BEAMS OR GIRDERS COMPRISE THE MAIN MEMBERS OF THROUGH SPANS, SUCH MEMBERS SHALL BE STIFFENED AGAINST LATERAL DEFORMATION BY MEANS OF GUSSET PLATES OR KNEE BRACES WITH SOLID WEBS WHICH SHALL BE CONNECTED TO THE STIFFENERS ON THE MAIN MEMBERS AND THE FLOOR BEAMS. IF THE UNSUPPORTED LENGTH OF THE EDGE OF THE GUSSET PLATE (OR SOLID WEB) EXCEEDS 60 TIMES ITS THICKNESS, THE PLATE OR WEB SHALL HAVE A STIFFENING PLATE OR ANGLES CONNECTED ALONG ITS UNSUPPORTED EDGE.

THROUGH TRUSS SPANS, DECK TRUSS SPANS AND SPANDREL BRACED ARCHES SHALL HAVE TOP AND BOTTOM LATERAL BRACING.

BRACING SHALL BE COMPOSED OF ANGLES, OTHER SHAPES OR WELDED SECTIONS.

IF A DOUBLE SYSTEM OF BRACING IS USED, BOTH SYSTEMS MAY BE CONSIDERED EFFECTIVE SIMULTANEOUSLY IF THE MEMBERS MEET THE REQUIREMENTS BOTH AS TENSION AND COMPRESSION MEMBERS. THE MEMBERS SHALL BE CONNECTED AT THEIR INTERSECTIONS.

THE LATERAL BRACING OF COMPRESSION CHORDS, PREFERABLY SHALL BE AS

DEEP AS THE CHORDS AND EFFECTIVELY CONNECTED TO BOTH FLANGES.

THE SMALLEST ANGLE USED IN BRACING SHALL BE 3 BY 2 1/2 INCHES. THERE SHALL BE NOT LESS THAN 2 FASTENERS OR EQUIVALENT WELD IN EACH END CONNECTION OF THE ANGLES.

FOR BRIDGES WITH SKEWS UP TO AND INCLUDING 30 DEGREES, DIAPHRAGMS OR CROSS FRAMES SHALL BE PLACED IN A CONTINUOUS LINE PARALLEL TO THE SKEW. FOR SKEWS OVER 30 DEGREES, THEY SHALL BE PLACED IN A CONTINUOUS LINE ACROSS THE BRIDGE AT RIGHT ANGLES TO THE STRINGERS.

FOR SECONDARY MEMBERS, THE EDGE OF THE GUSSET OR CONNECTION PLATE SHALL BE STIFFENED IF THE OUTSTANDING WIDTH OF THAT PORTION OF THE PLATE OUTSIDE THE MAIN MEMBER IS EQUAL TO OR GREATER THAN THE FOLLOWING NUMBER OF TIMES ITS THICKNESS.

26 FOR STEEL WITH 36000 P.S.I. Y.P. MIN.
24 FOR STEEL WITH 42000 P.S.I. Y.P. MIN.
23 FOR STEEL WITH 46000 P.S.I. Y.P. MIN.
22 FOR STEEL WITH 50000 P.S.I. Y.P. MIN.

1.7.22 -NUMBER OF MAIN MEMBERS ON THROUGH SPANS

WHERE BEAMS, GIRDERS OR TRUSSES ARE USED FOR THROUGH SPANS, THE SPANS PREFERABLY SHALL HAVE ONLY TWO MAIN MEMBERS. SUCH MEMBERS SHALL BE SPACED A SUFFICIENT DISTANCE APART (CENTER TO CENTER) TO BE SECURE AGAINST OVERTURNING BY THE ASSUMED LATERAL FORCES.

1.7.23 -ACCESSIBILITY OF PARTS

THE ACCESSIBILITY OF ALL PARTS OF A STRUCTURE FOR INSPECTION, CLEANING AND PAINTING SHALL BE SECURED BY THE PROPER PROPORTIONING OF MEMBERS AND THE DESIGN OF THEIR DETAILS.

1.7.24 -CLOSED SECTIONS AND POCKETS

CLOSED SECTIONS, AND POCKETS OR DEPRESSIONS WHICH WILL RETAIN WATER, SHALL BE AVOIDED WHERE PRACTICABLE. POCKETS SHALL BE PROVIDED WITH EFFECTIVE DRAIN HOLES OR BE FILLED WITH WATERPROOFING MATERIAL.

DETAILS SHALL BE SO ARRANGED THAT THE DESTRUCTIVE EFFECTS OF BIRD LIFE, THE RETENTION OF DIRT, LEAVES, AND OTHER FOREIGN MATTER WILL BE REDUCED TO A MINIMUM. WHERE ANGLES ARE USED, EITHER SINGLY OR IN PAIRS, THEY PREFERABLY SHALL BE PLACED WITH THE VERTICAL LEGS EXTENDING DOWNWARD.

1.7.25 -WELDING, GENERAL *****

WELDING SYMBOLS AND FABRICATION SHALL CONFORM TO THE CURRENT SPECIFICATIONS FOR WELDED HIGHWAY AND RAILWAY BRIDGES OF THE AMERICAN

WELDING SOCIETY.

MATERIAL FOR STRUCTURAL MEMBERS WHICH IS DESIGNED AND SPECIFIED TO BE WELDED SHALL CONFORM TO ASTM A36, ASTM A441, OR ASTM A588.

NO INTERMITTENT WELDING WILL BE PERMITTED.

1.7.26 -MINIMUM SIZE OF FILLET WELDS

THE MINIMUM FILLET WELD SIZE SHALL BE AS SHOWN IN THE FOLLOWING TABLE EXCEPT THE MINIMUM SIZE FILLET WELD CONNECTING PARTS CARRYING PRIMARY STRESS SHALL BE 5/16 IN. IN LIEU OF 1/4 IN. SHOWN IN THE TABLE. WELD SIZE IS DETERMINED BY THE THICKER OF THE TWO PARTS JOINED UNLESS A LARGER SIZE IS REQUIRED BY CALCULATED STRESS. THE WELD SIZE NEED NOT EXCEED THE THICKNESS OF THE THINNER PART JOINED. BEARINGS, BRACING CONNECTION STIFFENERS, DIAPHRAGMS AND LATERAL BRACING SHALL HAVE MINIMUM 5/16 IN. FILLET WELDS.

MATERIAL THICKNESS OF THICKER PART JOINED (INCHES)	MINIMUM SIZE OF FILLET WELD (INCHES)
TO 3/4 INCLUSIVE	1/4
OVER 3/4 TO 1 1/2	5/16
OVER 1 1/2 TO 2 1/4	3/8
OVER 2 1/4 TO 6	1/2
OVER 6	5/8

THE MINIMUM SIZE SEAL WELD SHALL BE 1/4 IN. FILLET WELD.

1.7.27 -MAXIMUM EFFECTIVE SIZE OF FILLET WELDS

THE MAXIMUM SIZE OF A FILLET WELD THAT MAY BE ASSUMED IN THE DESIGN OF A CONNECTION SHALL BE SUCH THAT THE STRESSES IN THE ADJACENT BASE MATERIAL DO NOT EXCEED THE VALUES ALLOWED IN ARTICLE 1.7.1. THE MAXIMUM SIZE THAT MAY BE USED ALONG EDGES OF CONNECTED PARTS SHALL BE:

(1) ALONG EDGES OF MATERIAL LESS THAN 1/4 INCH THICK, THE MAXIMUM SIZE MAY BE EQUAL TO THE THICKNESS OF THE MATERIAL.

(2) ALONG EDGES OF MATERIAL 1/4 INCH OR MORE IN THICKNESS, THE MAXIMUM SIZE SHALL BE 1/16 INCH LESS THAN THE THICKNESS OF THE MATERIAL, UNLESS THE WELD IS ESPECIALLY DESIGNATED ON THE DRAWINGS TO BE BUILT OUT TO OBTAIN FULL THROAT THICKNESS.

1.7.28 -EFFECTIVE WELD AREAS

A. BUTT WELDS

THE EFFECTIVE AREA SHALL BE THE EFFECTIVE WELD LENGTH MULTIPLIED BY THE EFFECTIVE THROAT THICKNESS.

(1) THE EFFECTIVE WELD LENGTH FOR ANY BUTT WELD, SQUARE OR SKEWED, SHALL BE THE WIDTH OF THE PART JOINED, PERPENDICULAR TO THE DIRECTION OF STRESS.

(2) THE EFFECTIVE THROAT THICKNESS SHALL BE THE THICKNESS OF THE THINNER PIECE OF BASE METAL JOINED. (NO INCREASE IS PERMITTED FOR WELD REINFORCEMENT.)

B. FILLET WELDS

THE EFFECTIVE AREA SHALL BE THE EFFECTIVE WELD LENGTH MULTIPLIED BY THE EFFECTIVE THROAT THICKNESS. (STRESS IN A FILLET WELD SHALL BE CONSIDERED AS APPLIED TO THIS EFFECTIVE AREA, FOR ANY DIRECTION OF APPLIED LOAD.)

(1) THE EFFECTIVE LENGTH OF STRAIGHT FILLET WELD SHALL BE THE OVERALL LENGTH OF THE FULL-SIZE FILLET INCLUDING END RETURNS.

(2) THE EFFECTIVE LENGTH OF A CURVED FILLET WELD SHALL BE THE LENGTH OF THE LINE GENERATED BY THE CENTERPOINT OF THE EFFECTIVE THROAT THICKNESS.

(3) THE EFFECTIVE THROAT THICKNESS SHALL BE THE SHORTEST DISTANCE FROM THE ROOT OF THE DIAGRAMMATIC WELD TO THE FACE.

1.7.29 -MINIMUM EFFECTIVE LENGTH OF FILLET WELDS

THE MINIMUM EFFECTIVE LENGTH OF A FILLET WELD SHALL BE FOUR TIMES ITS SIZE AND IN NO CASE LESS THAN 1 1/2 INCHES.

1.7.30 -FILLET WELD END RETURNS

FILLET WELDS WHICH SUPPORT A TENSILE FORCE THAT IS NOT PARALLEL TO THE AXIS OF THE WELD, OR WHICH ARE PROPORTIONED TO WITHSTAND REPEATED STRESS SHALL NOT TERMINATE AT CORNERS OF PARTS OR MEMBERS BUT SHALL BE RETURNED CONTINUOUSLY, FULL SIZE, AROUND THE CORNER FOR A LENGTH EQUAL TO TWICE THE WELD SIZE WHERE SUCH RETURN CAN BE MADE IN THE SAME PLANE. END RETURNS SHALL BE INDICATED ON DESIGN AND DETAIL DRAWINGS.

1.7.31 -LAP JOINTS

THE MINIMUM WIDTH OF LAPS ON LAP JOINTS SHALL BE 5 TIMES THE THICK-

NESS OF THE THINNER PART JOINED AND NOT LESS THAN 1 INCH. LAP JOINTS

JOINING PLATES OR BARS SUBJECTED TO AXIAL STRESS SHALL BE FILLET WELDED ALONG THE EDGE OF BOTH LAPPED PARTS EXCEPT WHERE THE DEFLECTION OF THE LAPPED PARTS IS SUFFICIENTLY RESTRAINED TO PREVENT OPENING OF THE JOINT UNDER MAXIMUM LOADING.

1.7.32 -SEAL WELDS

SEAL WELDING SHALL PREFERABLY BE ACCOMPLISHED BY A CONTINUOUS WELD COMBINING THE FUNCTIONS OF SEALING AND STRENGTH, CHANGING SECTION ONLY AS THE REQUIRED STRENGTH OR THE REQUIREMENTS OF MINIMUM SIZE FILLET WELD, BASED ON MATERIAL THICKNESS, MAY NECESSITATE.

1.7.33 FILLET WELDS IN SKEWED TEE JOINTS

WHEN JOINING MATERIAL IN SKEWED TEE JOINTS, FILLET WELDS SHALL NOT BE USED FOR JOINTS THAT HAVE AN INCLUDED ANGLE OF LESS THAN 60 DEGREES.

1.7.34 -FILLET WELDS IN HOLES AND SLOTS

FILLET WELDS IN HOLES OR SLOTS MAY BE USED TO TRANSMIT SHEAR IN LAP JOINTS OR TO PREVENT THE BUCKLING OR SEPARATION OF LAPPED PARTS, AND TO JOIN COMPONENTS OF BUILT-UP MEMBERS. SUCH FILLET WELDS MAY OVERLAP, SUBJECT TO THE PROVISIONS OF ARTICLE 1.7.28 . FILLET WELDS IN HOLES OR SLOTS ARE NOT TO BE CONSIDERED PLUG OR SLOT WELDS.

1.7.35 -SIZE OF FASTENERS (HIGH STRENGTH BOLTS)

FASTENERS SHALL BE OF THE SIZE SHOWN ON THE DRAWINGS, BUT GENERALLY SHALL BE 3/4 INCH OR 7/8 INCH IN DIAMETER. FASTENERS 5/8 INCH IN DIAMETER SHALL NOT BE USED IN MEMBERS CARRYING CALCULATED STRESS EXCEPT IN 2 1/2 INCH LEGS OF ANGLES AND IN FLANGES OF SECTIONS REQUIRING 5/8 INCH FASTENERS.

THE DIAMETER OF FASTENERS IN ANGLES CARRYING CALCULATED STRESS SHALL NOT EXCEED ONE-FOURTH THE WIDTH OF THE LEG IN WHICH THEY ARE PLACED.

IN ANGLES WHOSE SIZE IS NOT DETERMINED BY CALCULATED STRESS, 5/8 INCH FASTENERS MAY BE USED IN 2 INCH LEGS, 3/4 INCH FASTENERS IN 2 1/2 INCH LEGS, 7/8 INCH FASTENERS IN 3 INCH LEGS, AND 1 INCH FASTENERS IN 3 1/2 INCH LEGS.

STRUCTURAL SHAPES WHICH DO NOT ADMIT THE USE OF 5/8 INCH DIAMETER FASTENERS SHALL NOT BE USED EXCEPT IN HANDRAILS.

1.7.36 -SPACING OF FASTENERS

THE PITCH OF FASTENERS IS THE DISTANCE ALONG THE LINE OF PRINCIPAL STRESS, IN INCHES, BETWEEN CENTERS OF ADJACENT FASTENERS, MEASURED ALONG ONE OR MORE FASTENER LINES. THE GAGE OF FASTENERS IS THE DISTANCE IN INCHES BETWEEN ADJACENT LINES OF FASTENERS OR THE DISTANCE FROM THE BACK OF ANGLE OR OTHER SHAPE TO THE FIRST LINE OF FASTENERS. THE PITCH OF FASTENERS SHALL BE GOVERNED BY THE REQUIREMENTS FOR SEALING OR STITCH, WHICHEVER IS THE MINIMUM.

THE MINIMUM DISTANCE BETWEEN CENTERS OF FASTENERS SHALL NOT BE LESS THAN THE FOLLOWING:

FOR 1 1/8 INCH FASTENERS, 4 INCHES.
FOR 1 INCH FASTENERS, 3 1/2 INCHES.
FOR 7/8 INCH FASTENERS, 3 INCHES.
FOR 3/4 INCH FASTENERS, 2 1/2 INCHES.
FOR 5/8 INCH FASTENERS, 2 1/4 INCHES.

1.7.37 -MAXIMUM SPACING OF FASTENERS ****

FOR SEALING, THE MAXIMUM SPACING OF FASTENERS ALONG THE FREE EDGE OF A PLATE SHALL BE 4 INCHES + FOUR TIMES THE THICKNESS OF THE THINNER PLATE, BUT NOT MORE THAN 7 INCHES.

1.7.38 -EDGE DISTANCE OF FASTENERS ****

A. GENERAL

THE MINIMUM DISTANCE FROM THE CENTER OF ANY FASTENER TO THE EDGE OF A PLATE SHALL BE:

FOR 1 1/8 INCH FASTENERS, 2 INCHES.
FOR 1 INCH FASTENERS, 1 3/4 INCHES.
FOR 7/8 INCH FASTENERS, 1 1/2 INCHES.
FOR 3/4 INCH FASTENERS, 1 1/4 INCHES.
FOR 5/8 INCH FASTENERS, 1 1/8 INCHES.

IN THE FLANGES OR LEGS OF ROLLED SECTIONS THE DISTANCE SHALL BE:

FOR 1 1/8 INCH FASTENERS, 1 3/8 INCHES.
FOR 1 INCH FASTENERS, 1 1/4 INCHES.
FOR 7/8 INCH FASTENERS, 1 1/8 INCHES.
FOR 3/4 INCH FASTENERS, 1 INCH.
FOR 5/8 INCH FASTENERS, 7/8 INCH.

THE MAXIMUM DISTANCE FROM ANY EDGE SHALL BE EIGHT TIMES THE THICKNESS OF THE THINNEST OUTSIDE PLATE, BUT SHALL NOT EXCEED 5 INCHES.

B. SPECIAL

IN CONNECTIONS DESIGNED BY BEARING ON THE PLATES AND HAVING NO MORE THAN TWO LINES OF FASTENERS PARALLEL TO THE DIRECTION OF THE STR THE DISTANCE BETWEEN THE CENTER OF THE NEAREST FASTENER AND THAT END THE CONNECTED MEMBER TOWARD WHICH THE PRESSURE FROM THE FASTENER IS DIRECTED, SHALL NOT BE LESS THAN THE NOMINAL SHEARING AREA OF THE FASTENER (SINGLE OR DOUBLE SHEAR, AS THE CASE MAY BE) DIVIDED BY TWO-THIRDS OF THE PLATE THICKNESS. THIS END DISTANCE MAY BE PROPORTIONATEL LESS WHERE THE STRESS PER FASTENER IS LESS THAN THAT OF THE MAXIMUM PER MITTED, BUT NOT LESS THAN 1-1/2 TIMES THE DIAMETER OF THE FASTENER.

IN BEARING TYPE CONNECTIONS HAVING NO MORE THAN TWO FASTENERS IN A LINE PARALLEL TO THE DIRECTION OF STRESS, THE DISTANCE BETWEEN THE CENTER OF THE NEAREST FASTENER AND THAT END OF THE CONNECTED MEMBER TOWARDS WHICH THE PRESSURE FROM THE FASTENER IS DIRECTED SHALL NOT BE LESS THAN AC/T FOR SINGLE SHEAR OR 2AC/T FOR DOUBLE SHEAR, WHERE A IS TH NOMINAL CROSS-SECTIONAL AREA OF THE FASTENER, T IS THE THICKNESS OF THE CONNECTED PART AND C IS THE RATIO OF SPECIFIED MINIMUM TENSILE STRENGTH OF THE FASTENER TO THE SPECIFIED MINIMUM TENSILE STRENGTH OF THE CON-NECTED PART. THIS END DISTANCE MAY BE PROPORTIONATELY LESS WHERE THE SHEAR STRESS PER FASTENER IS LESS THAN THAT PERMITTED IN ARTICLE 1.7. BUT NOT LESS THAN 1 1/2 TIMES THE FASTENER DIAMETER. IT NEED NOT EXCE 1 1/2 TIMES THE TRANSVERSE SPACING OF THE FASTENERS.

1.7.39 -LONG RIVETS (DELETED)

1.7.40 -LINKS AND HANGERS

IN PIN-CONNECTED TENSION MEMBERS OTHER THAN EYEBARS, THE NET SECTION ACROSS THE PIN HOLE SHALL BE NOT LESS THAN 140 PERCENT, AND THE NET SECTION BACK OF THE PIN HOLE NOT LESS THAN 100 PERCENT OF THE REQUIRED NET SECTION OF THE BODY OF THE MEMBER. THE RATIO OF THE NET WIDTH (THROUGH THE PIN HOLE TRANSVERSE TO THE AXIS OF THE MEMBER) TO THE THICKNESS OF THE SEGMENT SHALL NOT BE MORE THAN 8. FLANGES NOT BEARING ON THE PIN SHALL NOT BE CONSIDERED IN THE NET SECTION ACROSS THE PIN.

1.7.41 -LOCATION OF PINS

PINS SHALL BE SO LOCATED WITH RESPECT TO THE GRAVITY AXIS OF THE MEMBERS AS TO REDUCE TO A MINIMUM STRESSES DUE TO BENDING.

1.7.42 -SIZE OF PINS

PINS SHALL BE PROPORTIONED FOR THE MAXIMUM SHEARS AND BENDING MOMENTS PRODUCED BY THE STRESSES IN THE MEMBERS CONNECTED. IF THERE ARE EYEBARS AMONG THE PARTS CONNECTED, THE DIAMETER OF THE PIN SHALL BE AS SPECIFIED IN ARTICLE 1.7.47.

1.7.43 -WEB SECTION AT PIN HOLES

WHEN NECESSARY FOR THE REQUIRED SECTION OR BEARING AREA, THE SECTION AT THE PIN HOLES SHALL BE INCREASED BY SPLICING ON A SECTION OF WEB PLATE OF THE REQUIRED THICKNESS TO THE NORMAL WEB.

1.7.44 -PINS AND PIN NUTS

PINS SHALL BE OF SUFFICIENT LENGTH TO SECURE A FULL BEARING OF ALL PARTS CONNECTED UPON THE TURNED BODY OF THE PIN. THEY SHALL BE SECURED IN POSITION BY HEXAGONAL RECESSED NUTS OR BY HEXAGONAL SOLID NUTS WITH WASHERS. IF THE PINS ARE BORED, THROUGH RODS WITH CAP WASHERS MAY BE USED. PIN NUTS SHALL BE MALLEABLE CASTINGS OR STEEL. THEY SHALL BE SECURED BY COTTER PINS IN THE SCREW ENDS OR ELSE THE SCREW ENDS SHALL BE LONG ENOUGH TO PERMIT BURRING THE THREADS.

MEMBERS SHALL BE HELD AGAINST LATERAL MOVEMENT ON THE PINS.

1.7.45 -UPSET ENDS (DELETED)

1.7.46 -EYEBARS

EYEBARS SHALL BE OF A UNIFORM THICKNESS WITHOUT REINFORCEMENT AT THE PIN HOLES. THE THICKNESS OF EYEBARS SHALL BE NOT LESS THAN 1/8 OF THE WIDTH, NOR LESS THAN 1/2 INCH, AND NOT GREATER THAN 2 INCHES. THE SECTION OF THE HEAD THROUGH THE CENTER OF THE PIN HOLE SHALL EXCEED THAT OF THE BODY OF THE BAR BY AT LEAST 35 PERCENT. THE NET SECTION BACK OF THE PIN HOLE SHALL NOT BE LESS THAN 75 PERCENT OF THE REQUIRED NET SECTION OF THE BODY OF THE MEMBER. THE RADIUS OF TRANSITION BETWEEN THE HEAD AND BODY OF THE EYEBAR SHALL BE EQUAL TO OR GREATER THAN THE WIDTH OF THE HEAD THROUGH THE CENTERLINE OF THE PIN HOLE. THE DIAMETER OF THE PIN SHALL BE NOT LESS THAN $\left[\frac{3}{4} + \frac{1}{4}\,\frac{(\text{YIELD POINT OF STEEL})}{100000}\right]$ TIMES THE WIDTH OF THE BODY OF THE EYEBAR.

1.7.47 -PACKING OF EYEBARS (DELETED)

1.7.48 -FORKED ENDS (DELETED)

1.7.49 -FIXED BEARINGS

FIXED ENDS SHALL BE FIRMLY ANCHORED. BEARINGS FOR SPANS LESS THAN 50 FEET NEED HAVE NO PROVISION FOR DEFLECTION. SPANS OF 50 FEET OR GREATER SHALL BE PROVIDED WITH A TYPE OF BEARING EMPLOYING A HINGE,

CURVED BEARING PLATES, ELASTOMERIC PADS OR PIN ARRANGEMENT FOR DEFLECTION PURPOSES. GRAPHITE FILLED BRONZE INSERTS MAY BE USED IF NECESSARY.

1.7.50 -EXPANSION BEARINGS

SPANS OF LESS THAN 50 FEET MAY BE ARRANGED TO SLIDE UPON METAL PLATES WITH SMOOTH SURFACES AND NO PROVISIONS FOR DEFLECTION OF THE SPANS NEED BE MADE. SPANS OF 50 FEET AND GREATER SHALL BE PROVIDED WITH ROLLERS, ROCKERS OR SLIDING PLATES FOR EXPANSION PURPOSES AND SHALL ALSO BE PROVIDED WITH A TYPE OF BEARING EMPLOYING A HINGE, CURVED BEARING PLATES,OR PIN ARRANGEMENT WITH GRAPHITE FILLED BRONZE INSERTS IF NECESSARY FOR DEFLECTION PURPOSES.

IN LIEU OF THE ABOVE REQUIREMENTS ELASTOMERIC BEARING PADS MAY BE USED

1.7.51 -BRONZE OR COPPER ALLOY SLIDING EXPANSION BEARINGS

BRONZE OR COPPER-ALLOY SLIDING PLATES SHALL BE CHAMFERED AT THE ENDS. THEY SHALL BE HELD SECURELY IN POSITION. PROVISIONS SHALL BE MADE AGAINST ANY ACCUMULATION OF DIRT WHICH WILL OBSTRUCT FREE MOVEMENT OF THE SPAN

1.7.52 -ROLLERS (DELETED)

1.7.53-SOLE PLATES AND MASONRY PLATES

SOLE PLATES AND MASONRY PLATES SHALL HAVE A MINIMUM THICKNESS OF 3/4 INCH.

FOR SPANS ON INCLINED GRADES GREATER THAN 1 % WITHOUT HINGED BEARINGS THE SOLE PLATES SHALL BE BEVELED SO THAT THE BOTTOM OF THE SOLE PLATE IS LEVEL, UNLESS THE BOTTOM OF THE SOLE PLATE IS RADIALLY CURVED.

1.7.54 -MASONRY BEARINGS (DELETED)

1.7.55 -ANCHOR BOLTS *****

ANCHOR BOLTS SHALL BE SWEDGED OR ROUGHENED TO SECURE A SATISFACTORY GRIP UPON THE MATERIAL IN WHICH THEY ARE IMBEDDED.

THE FOLLOWING ARE THE MINIMUM REQUIREMENTS FOR EACH BEARING:

FOR MULTIPLE STRINGER TYPE SPANS THE BEARINGS SHALL BE ANCHORED AT EACH END WITH 2 BOLTS 1 IN. IN DIAMETER, SET 6 IN. INTO THE MASONRY.

FOR OTHER TYPES OF SUPERSTRUCTURE SYSTEMS THE BEARINGS SHALL BE ANCHORED WITH 4 BOLTS 1 IN. IN DIAMETER SET 6 INCHES INTO THE MASONRY, UNLESS OTHERWISE REQUIRED BY DESIGN.

ANCHOR BOLTS SUBJECT TO TENSION SHALL BE DESIGNED TO ENGAGE A MASS OF MASONRY WHICH WILL PROVIDE A RESISTANCE EQUAL TO 1 1/2 TIMES THE CALCULATED UPLIFT.

1.7.56 -PEDESTALS AND SHOES

PEDESTALS AND SHOES PREFERABLY SHALL BE MADE OF CAST STEEL OR STRUCTURAL STEEL. THE DIFFERENCE IN WIDTH BETWEEN THE TOP AND BOTTOM BEARING SURFACES SHALL NOT EXCEED TWICE THE DISTANCE BETWEEN THEM. FOR HINGED BEARINGS, THIS DISTANCE SHALL BE MEASURED FROM THE CENTER OF THE PIN. IN BUILT-UP PEDESTALS AND SHOES, THE WEB PLATES AND ANGLES CONNECTING THEM TO THE BASE PLATE SHALL BE NOT LESS THAN 5/8 IN. THICK. IF THE SIZE OF THE PEDESTAL PERMITS, THE WEBS SHALL BE RIGIDLY CONNECTED TRANSVERSELY. THE MINIMUM THICKNESS OF THE METAL IN CAST STEEL PEDESTALS SHALL BE 1 IN. PEDESTALS AND SHOES SHALL BE SO DESIGNED THAT THE LOAD WILL BE DISTRIBUTED UNIFORMLY OVER THE ENTIRE BEARING.

WEBS AND PIN HOLES IN THE WEBS SHALL BE ARRANGED TO KEEP ANY ECCENTRICITY TO A MINIMUM. THE NET SECTION THROUGH THE HOLE SHALL PROVIDE 140 % OF THE NET SECTION REQUIRED FOR THE ACTUAL STRESS TRANSMITTED THROUGH THE PEDESTAL OR SHOE. PINS SHALL BE OF SUFFICIENT LENGTH TO SECURE A FULL BEARING. PINS SHALL BE SECURED IN POSITION BY APPROPRIATE NUTS WITH WASHERS. ALL PORTIONS OF PEDESTALS AND SHOES SHALL BE HELD AGAINST LATERAL MOVEMENT ON THE PINS.

FLOOR SYSTEM

1.7.57 -STRINGERS

STRINGERS PREFERABLY SHALL BE FRAMED INTO FLOORBEAMS. STRINGERS SUPPORTED ON THE TOP FLANGES OF FLOORBEAMS PREFERABLY SHALL BE CONTINUOUS OVER TWO OR MORE PANELS.

1.7.58 -FLOORBEAMS *****

FLOORBEAMS PREFERABLY SHALL BE AT RIGHT ANGLES TO THE TRUSSES OR MAIN GIRDERS AND SHALL BE RIGIDLY CONNECTED THERETO. FLOORBEAM CONNECTIONS PREFERABLY SHALL BE LOCATED SO THE LATERAL BRACING SYSTEM WILL ENGAGE BOTH THE FLOORBEAM AND THE MAIN SUPPORTING MEMBER.

1.7.59 -CROSS FRAMES (DELETED)

1.7.60 -EXPANSION JOINTS

TO PROVIDE FOR EXPANSION AND CONTRACTION MOVEMENT, FLOOR EXPANSION JOINTS SHALL BE PROVIDED AT ALL EXPANSION ENDS OF SPANS AND AT OTHER POINTS WHERE THEY MAY BE NECESSARY.

1.7.61 -END CONNECTIONS OF FLOORBEAMS AND STRINGERS

THE END CONNECTION SHALL BE DESIGNED FOR THE LOADS SPECIFIED. THE END CONNECTION ANGLES OF FLOORBEAMS AND STRINGERS SHALL BE NOT LESS THAN 3/8 INCH IN FINISHED THICKNESS. EXCEPT IN CASES OF SPECIAL END FLOORBEAM DETAILS, EACH END CONNECTION FOR FLOORBEAMS AND STRINGERS SHALL BE MADE WITH TWO ANGLES. THE LENGTH OF THESE ANGLES SHALL BE AS GREAT AS THE FLANGES WILL PERMIT. BRACKET OR SHELF ANGLES WHICH MAY BE USED TO FURNISH SUPPORT DURING ERECTION SHALL NOT BE CONSIDERED IN DETERMINING THE NUMBER OF FASTENERS REQUIRED TO TRANSMIT END SHEAR.

END CONNECTION DETAILS SHALL BE DESIGNED WITH SPECIAL CARE TO PROVIDE CLEARANCE FOR MAKING THE FIELD CONNECTION.

END CONNECTIONS OF STRINGERS AND FLOORBEAMS PREFERABLY SHALL BE BOLTED WITH HIGH STRENGTH BOLTS, HOWEVER, THEY MAY BE WELDED. IN THE CASE OF WELDED END CONNECTIONS, THEY SHALL BE DESIGNED FOR THE VERTICAL LOADS AND THE END BENDING MOMENT RESULTING FROM THE DEFLECTION OF THE MEMBERS.

WHERE TIMBER STRINGERS FRAME INTO STEEL FLOORBEAMS, SHELF ANGLES WITH STIFFENERS SHALL BE PROVIDED TO CARRY THE WHOLE REACTION. SHELF ANGLES SHALL BE NOT LESS THAN 7/16 INCH THICK.

1.7.62 -END FLOORBEAMS

THERE SHALL BE END FLOORBEAMS IN ALL SQUARE-ENDED TRUSSES AND GIRDER SPANS AND PREFERABLY IN SKEW SPANS. END FLOORBEAMS FOR TRUSS SPANS PREFERABLY SHALL BE DESIGNED TO PERMIT THE USE OF JACKS FOR LIFTING THE SUPERSTRUCTURE. FOR THIS CASE THE ALLOWABLE STRESSES MAY BE INCREASED 50 PERCENT.

END FLOORBEAMS SHALL BE ARRANGED TO PERMIT PAINTING OF THE SIDE OF THE BEAM ADJACENT TO THE ABUTMENT BACKWALL.

1.7.63 END PANEL OF SKEWED BRIDGES

IN SKEW BRIDGES WITHOUT END FLOORBEAMS, THE END PANEL STRINGERS SHALL BE SECURED IN CORRECT POSITION BY END STRUTS CONNECTED TO THE STRINGERS AND TO THE MAIN TRUSSES OR GIRDER. THE END PANEL LATERAL BRACING SHALL BE ATTACHED TO THE MAIN TRUSSES OR GIRDERS AND ALSO TO THE

END STRUTS. ADEQUATE PROVISIONS SHALL BE MADE FOR THE EXPANSION MOVEMENT OF STRINGERS.

1.7.64 -SIDEWALK BRACKETS

SIDEWALK BRACKETS SHALL BE CONNECTED IN SUCH A WAY THAT THE BENDING STRESSES WILL BE TRANSFERRED DIRECTLY TO THE FLOORBEAMS.

ROLLED BEAMS

1.7.65 ROLLED BEAMS, GENERAL *****

ROLLED BEAMS, INCLUDING THOSE WITH WELDED COVER PLATES, SHALL BE DESIGNED BY THE MOMENT OF INERTIA METHOD.

THE COMPRESSION FLANGES OF ROLLED BEAMS SUPPORTING TIMBER FLOORS SHALL NOT BE CONSIDERED TO BE LATERALLY SUPPORTED BY THE FLOORING UNLESS THE FLOOR AND FASTENINGS ARE SPECIALLY DESIGNED TO PROVIDE ADEQUATE SUPPORT.

1.7.66 -BEARING STIFFENERS

SUITABLE STIFFENERS SHALL BE PROVIDED TO STIFFEN THE WEBS OF ROLLED BEAMS AT BEARINGS WHEN THE UNIT SHEAR IN THE WEB ADJACENT TO THE BEARING EXCEEDS 75% OF THE ALLOWABLE SHEAR FOR GIRDER WEBS. SEE THE RELATED PROVISIONS OF ARTICLE 1.7.73.

1.7.67 -COVER PLATES

THE LENGTH OF ANY COVER PLATE ADDED TO A ROLLED BEAM SHALL BE NOT LESS THAN (2D+3) FEET WHERE (D) IS THE DEPTH OF THE BEAM IN FEET.

WELDED COVER PLATES SHALL BE LIMITED TO ONE ON ANY ONE FLANGE. THE MAXIMUM THICKNESS OF THE COVER PLATE ON A FLANGE SHALL NOT BE GREATER THAN 2 TIMES THE THICKNESS OF THE FLANGE TO WHICH THE COVER PLATE IS ATTACHED. THE THICKNESS AND WIDTH OF A COVER PLATE MAY BE VARIED BY BUTT WELDING PARTS OF DIFFERENT THICKNESS OR WIDTH, WITH TRANSITIONS CONFORMING TO THE REQUIREMENTS OF STANDARD PRACTICES. SUCH PLATES SHALL BE ASSEMBLED AND WELDS GROUND SMOOTH BEFORE ATTACHING TO FLANGE. COVER PLATES MAY BE EITHER NARROWER OR WIDER THAN THE FLANGE TO WHICH THEY ARE ATTACHED.

ANY PARTIAL LENGTH WELDED COVER PLATE SHALL EXTEND BEYOND THE THEORETICAL END BY THE TERMINAL DISTANCE, OR IT SHALL EXTEND TO A SECTION WHERE THE STRESS IN THE BEAM FLANGE IS EQUAL TO THE ALLOWABLE FATIGUE STRESS FOR "BASE METAL ADJACENT TO OR CONNECTED BY FILLET WELDS," WHICHEVER IS GREATER. THE THEORETICAL END OF THE COVER PLATE IS THE SECTION AT WHICH THE STRESS IN THE FLANGE WITHOUT THE COVER PLATE EQUAL THE ALLOWABLE STRESS EXCLUSIVE OF FATIGUE CONSIDERATIONS. THE TERMINAL DISTANCE IS 2 TIMES THE NOMINAL COVER PLATE WIDTH FOR COVER PLATES NOT WELDED ACROSS THEIR ENDS, AND 1 1/2 TIMES FOR COVER PLATES WELDED ACROSS THEIR ENDS. THE WIDTH AT ENDS OF TAPERED COVER PLATES SHALL NOT BE LESS THAN 3 INCHES. THE WELD CONNECTING THE COVER PLATE TO THE FLANGE IN ITS TERMINAL DISTANCE SHALL BE CONTINUOUS AND OF SUFFICIENT SIZE TO DEVELOP

A TOTAL STRESS OF NOT LESS THAN THE COMPUTED STRESS IN THE COVER PLATE AT ITS THEORETICAL END. ALL WELDS CONNECTING COVER PLATES TO BEAM FLANGES SHALL BE CONTINUOUS AND SHALL NOT BE SMALLER THAN THE MINIMUM SIZE PERMITTED BY ARTICLE 1.7.26.

PLATE GIRDERS

1.7.68 -PLATE GIRDERS, GENERAL

GIRDERS SHALL BE PROPORTIONED BY THE MOMENT OF INERTIA METHOD.

THE COMPRESSION FLANGES OF PLATE GIRDERS SUPPORTING TIMBER FLOORS SHALL NOT BE CONSIDERED TO BE LATERALLY SUPPORTED BY THE FLOORING UNLESS THE FLOOR AND FASTENINGS ARE SPECIALLY DESIGNED TO PROVIDE SUPPORT.

1.7.69 -FLANGES *****

EACH FLANGE PREFERABLY SHALL CONSIST OF A SINGLE PLATE. THE SINGLE PLATE MAY COMPRISE A SERIES OF SHORTER PLATES JOINED END TO END BY FULL PENETRATION BUTT WELDS. WHERE THE THICKNESSES OR WIDTHS OF PLATES VARY, THE SPLICE SHALL BE MADE IN ACCORDANCE WITH THE DETAILS SHOWN IN THE STANDARD PRACTICES

THE RATIO OF COMPRESSION FLANGE PLATE WIDTH TO THICKNESS SHALL NOT EXCEED THE VALUE DETERMINED BY THE FORMULA:

$b/t = \frac{3250}{\sqrt{f_b}}$ BUT IN NO CASE SHALL b/t EXCEED 24

WHERE THE CALCULATED COMPRESSIVE BENDING STRESS EQUALS .55 F_y THE b/t RATIOS FOR THE VARIOUS GRADES OF STEEL SHALL NOT EXCEED THE FOLLOWING:

36,000 PSI, Y.P. MIN.	$b/t = 23$
42,000 PSI, Y.P. MIN.	$b/t = 21$
46,000 PSI, Y.P. MIN.	$b/t = 21$
50,000 PSI, Y.P. MIN.	$b/t = 20$

IN THE ABOVE b IS THE FLANGE PLATE WIDTH, t IS THE THICKNESS, AND f_b IS THE CALCULATED MAXIMUM COMPRESSIVE BENDING STRESS.

(See Art. 1.7.112 for Hybrid Girders)

FOR COMPOSITE GIRDERS A b/t RATIO OF 24 SHALL BE USED REGARDLESS OF TYPE OF STEEL. THE COMPRESSION FLANGE OF COMPOSITE PLATE GIRDERS SHALL BE CONSIDERED SUPPORTED AFTER THE CONCRETE SLAB HAS CURED. THE THEORETICAL CALCULATED MAXIMUM COMPRESSION STRESS IS THE STRESS CAUSED BY THE DEAD LOAD OF THE STEEL AND THE CONCRETE DECK

THE WIDTH OF A FLANGE PLATE SHALL BE NOT LESS THAN 9 INCHES.

THE MINIMUM THICKNESS OF ANY FLANGE SHALL BE 1/2 IN.

FLANGE PLATES MAY BE REDUCED IN WIDTH, AT A BUTT WELD OR TAPERED BETWEEN BUTT WELDS.

THE THICKNESS RATIO OF TWO FLANGE PLATES AT A JOINT SHALL BE 2 TO 1 OR LESS.

1.7.70 -THICKNESS OF WEB PLATES *****

A. GIRDERS NOT STIFFENED LONGITUDINALLY

THE WEB PLATE THICKNESS OF PLATE GIRDERS WITHOUT LONGITUDINAL STIFFENERS SHALL NOT BE LESS THAN THAT DETERMINED BY THE FORMULA:

$$t = \frac{D\sqrt{f_b}}{23{,}000}$$ (SEE FIGURE 1.7.70)

BUT IN NO CASE SHALL THE THICKNESS BE LESS THAN D/170 OR LESS THAN 3/8 INCH.

WHERE THE CALCULATED COMPRESSIVE BENDING STRESS EQUALS THE ALLOWABLE BENDING STRESS, THE THICKNESS OF THE WEB PLATE, (WITH THE WEB STIFFENED OR NOT STIFFENED DEPENDING UPON THE REQUIREMENTS FOR TRANSVERSE STIFFENERS), SHALL NOT BE LESS THAN (WHERE THE Y.P. IS FOR THE FLANGE MATERIAL):

36,000 PSI. Y.P. MIN.	D / 165
42,000 PSI. Y.P. MIN.	D / 150
46,000 PSI. Y.P. MIN.	D / 145
50,000 PSI. Y.P. MIN.	D / 140

B. GIRDERS STIFFENED LONGITUDINALLY

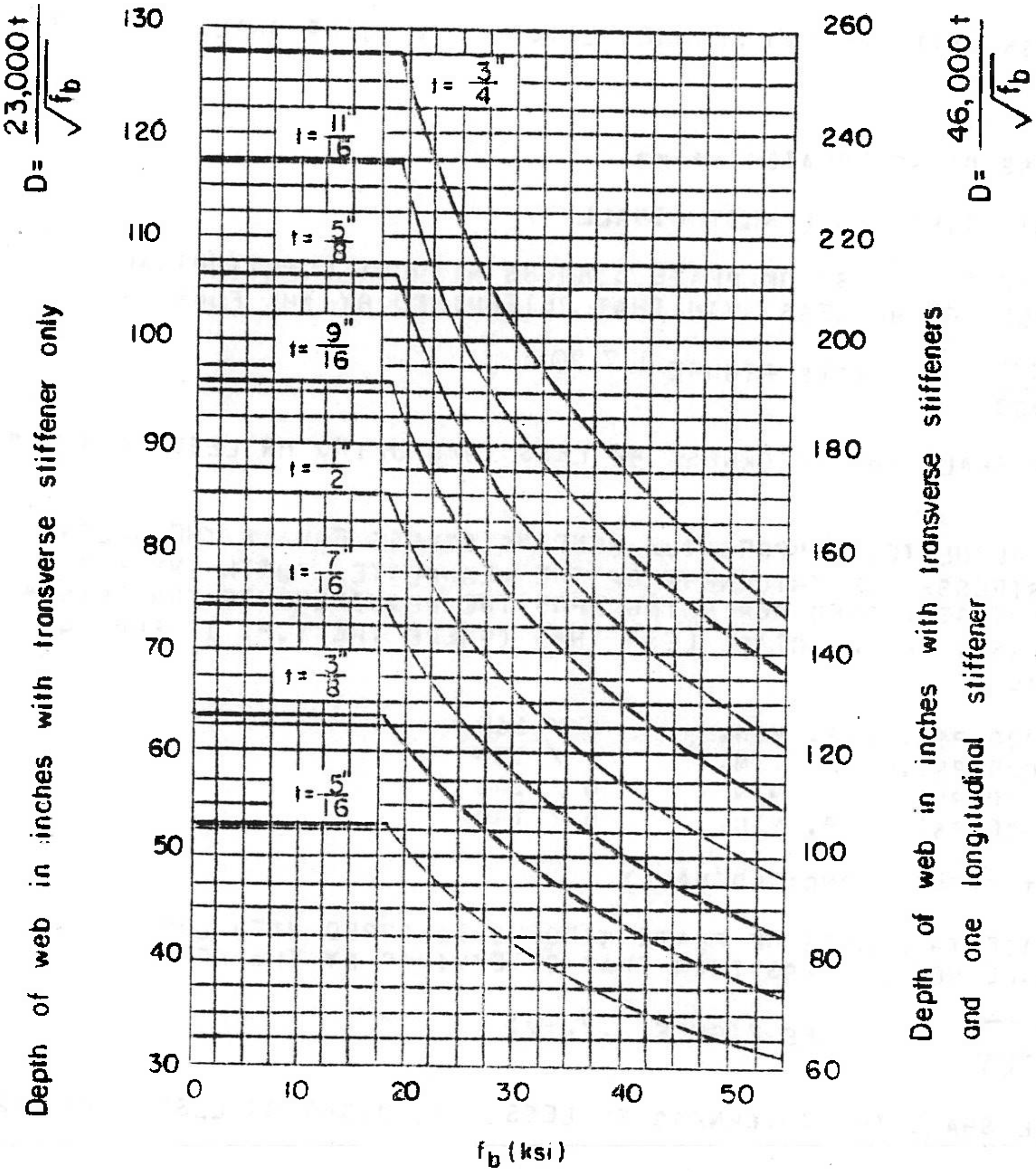
THE WEB PLATE THICKNESS OF PLATE GIRDERS EQUIPPED WITH LONGITUDINAL STIFFENERS SHALL NOT BE LESS THAN THAT DETERMINED BY THE FORMULA:

$$t = \frac{D\sqrt{f_b}}{46{,}000}$$ (SEE FIGURE 1.7.70)

BUT IN NO CASE SHALL THE THICKNESS BE LESS THAN D/340 OR LESS THAN 1/2 INCH.

WHERE THE CALCULATED BENDING STRESS EQUALS THE ALLOWABLE BENDING STRESS THE THICKNESS OF THE WEB PLATE STIFFENED WITH TRANSVERSE STIFFENERS IN COMBINATION WITH ONE LONGITUDINAL STIFFENER SHALL NOT BE LESS THAN (WHEN THE Y.P. IS FOR THE FLANGE MATERIAL):

36,000 PSI. Y.P. MIN.	D / 330
42,000 PSI. Y.P. MIN.	D / 300
46,000 PSI. Y.P. MIN.	D / 290



WEB THICKNESS AND GIRDER DEPTH

(a function of bending stress)

D = depth of web

t = thickness of web plate

f_b = calculated compressive bending stress in flange

FIGURE 1.7.70

50,000 PSI. Y.P. MIN. D / 280

IN THE ABOVE D (DEPTH OF WEB) IS THE CLEAR UNSUPPORTED DISTANCE, IN INCHES, BETWEEN FLANGE COMPONENTS, t IS THE WEB THICKNESS AND f_b IS THE CALCULATED FLANGE BENDING STRESS.

1.7.71 -TRANSVERSE INTERMEDIATE STIFFENERS

EXCEPT AS OTHERWISE PROVIDED BELOW, THE WEBS OF PLATE GIRDERS SHALL BE STIFFENED AT INTERVALS NOT GREATER THAN THE DISTANCE GIVEN BY THE FORMULA:

$$d = \frac{11,000t}{\sqrt{f_v}}$$ (SEE FIGURE 1.7.71A)

BUT NOT GREATER THAN THE CLEAR UNSUPPORTED DEPTH OF THE WEB PLATE BETWEEN FLANGES IN WHICH:

d = THE REQUIRED DISTANCE BETWEEN STIFFENERS, IN INCHES
t = THE THICKNESS OF THE WEB PLATE, IN INCHES
f_v = THE AVERAGE CALCULATED UNIT SHEARING STRESS IN THE GROSS SECTION OF THE WEB PLATE AT THE POINT CONSIDERED.

THE FIRST TWO STIFFENER SPACES AT THE SIMPLY SUPPORTED ENDS OF GIRDERS SHALL BE ONE-HALF THE VALUE SPECIFIED ABOVE.

TRANSVERSE INTERMEDIATE STIFFENERS MAY BE OMITTED IF THE WEB PLATE THICKNESS IS NOT LESS THAN THE THICKNESS DETERMINED BY THE FORMULA:

$$t = \frac{D\sqrt{f_v}}{7500}$$ (SEE FIGURE 1.7.71B))

IN NO CASE SHALL t BE LESS THAN D/150.

WHERE THE CALCULATED SHEAR STRESS EQUALS THE ALLOWABLE SHEAR STRESS, TRANSVERSE INTERMEDIATE STIFFENERS MAY BE OMITTED IF THE THICKNESS OF THE WEB IS NOT LESS THAN:

36,000 PSI. Y.P. MIN.	D / 68
42,000 PSI. Y.P. MIN.	D / 64
46,000 PSI. Y.P. MIN.	D / 60
50,000 PSI. Y.P. MIN.	D / 58

INTERMEDIATE STIFFENERS SHALL CONSIST OF A SINGLE PLATE WELDED TO ONE SIDE OF THE WEB AND THE COMPRESSION FLANGE. THEY SHALL BE PLACED ON ALTERNATE SIDES OF THE WEB EXCEPT WHERE THEY ARE USED IN CONJUNCTION WITH A LONGITUDINAL STIFFENER, IN WHICH CASE THEY SHALL ALL BE PLACED ON ONE SIDE WITH THE LONGITUDINAL STIFFENER ON THE OTHER SIDE.

THE MOMENT OF INERTIA OF ANY TYPE OF TRANSVERSE STIFFENER SHALL NOT

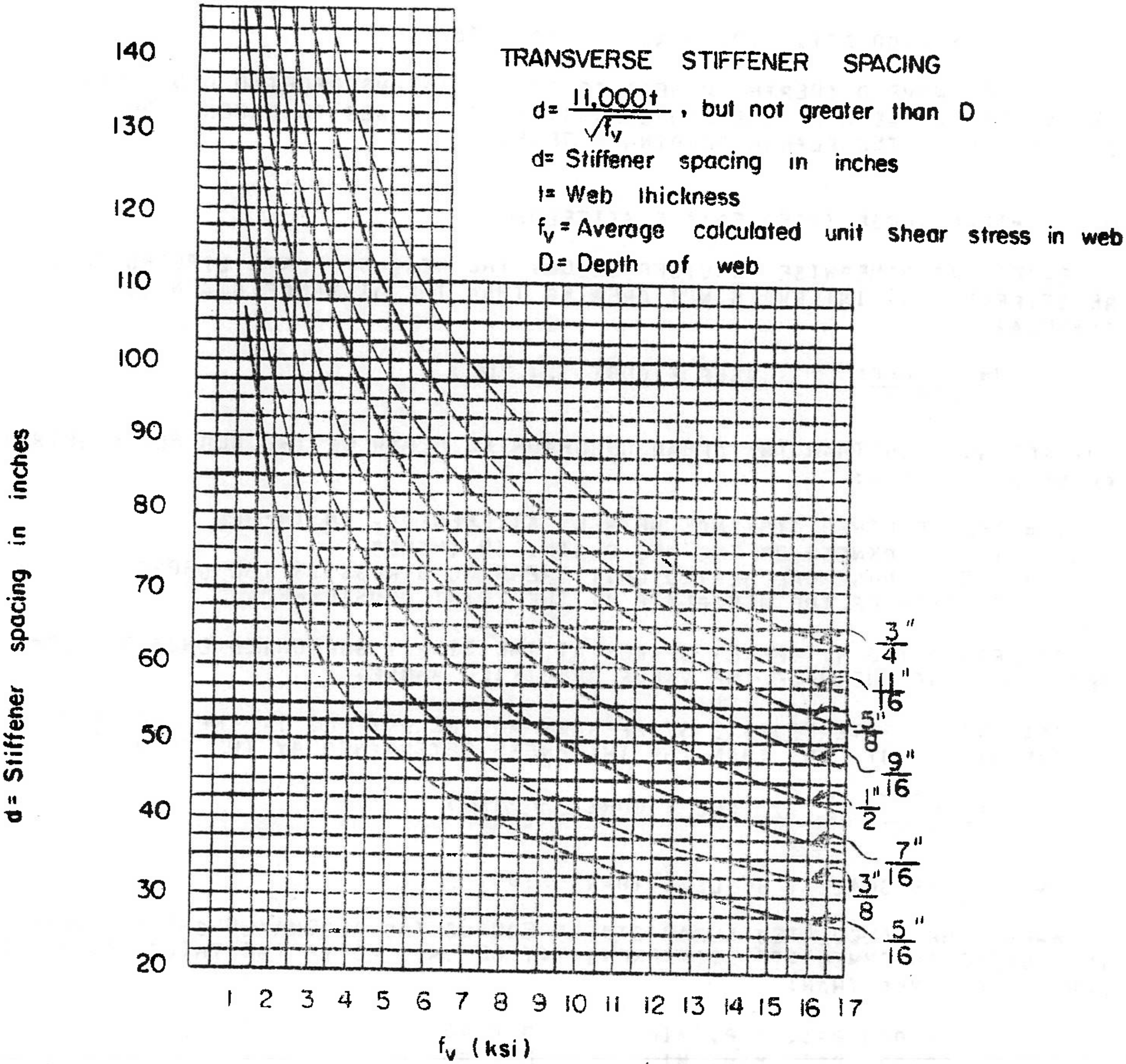


FIGURE 1.7.71A

WEB PLATE WITHOUT STIFFENERS

V = total shear

f_v = average calculated unit shear stress in web

t = thickness of web plate

D = depth of web plate in inches

$$t = \frac{D\sqrt{f_v}}{7500} = \frac{D\sqrt{\frac{V}{Dt}}}{7500}$$

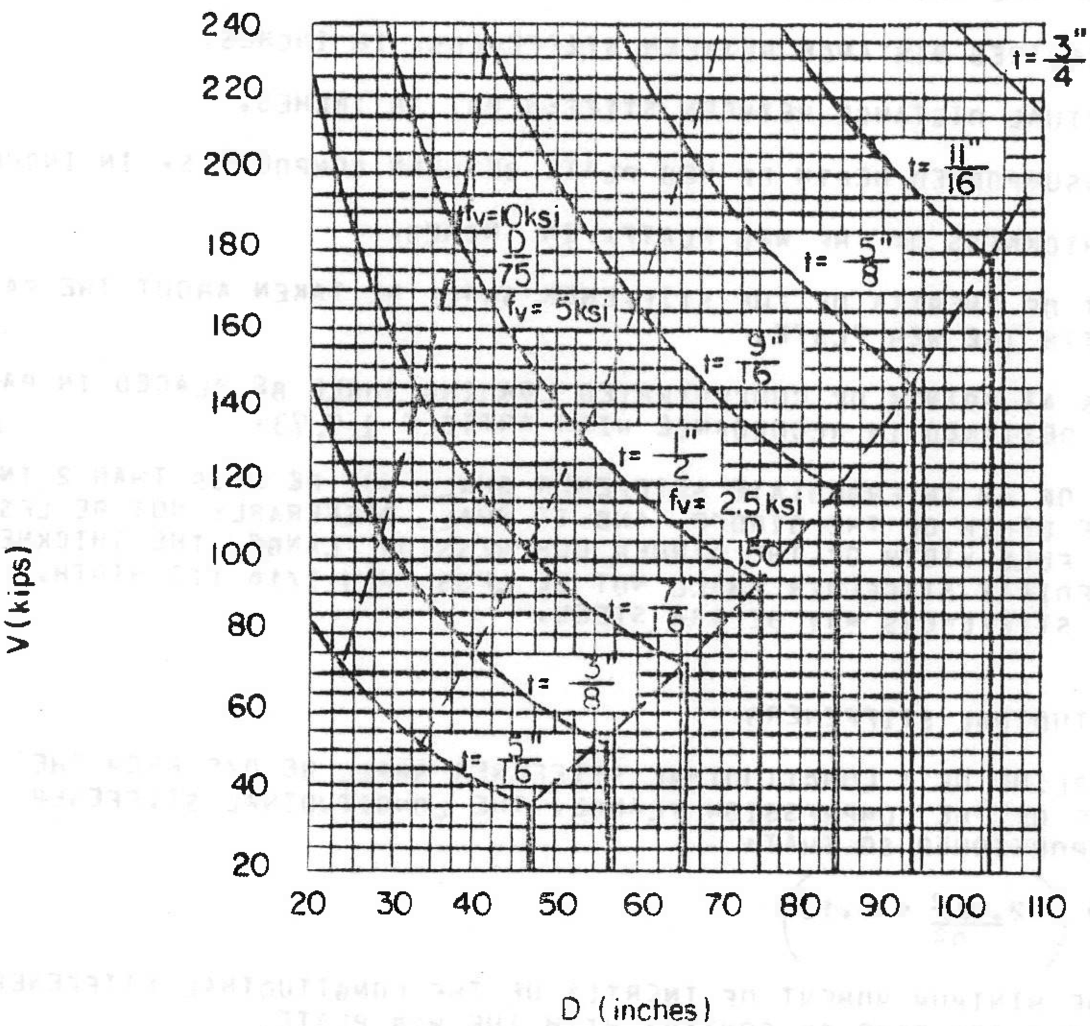


FIGURE 1.7.71B

BE LESS THAN:

$$I = \frac{d_o t^3 J}{10.92}$$

WHERE $J = 25 \frac{D^2}{d^2} - 20$, BUT NOT LESS THAN 5.0.

IN THESE EXPRESSIONS,

I = THE MINIMUM PERMISSIBLE MOMENT OF INERTIA OF ANY TYPE OF TRANSVERSE INTERMEDIATE STIFFENER.

J = THE REQUIRED RATIO OF RIGIDITY OF ONE TRANSVERSE STIFFENER TO THAT OF THE WEB PLATE.

d = THE REQUIRED DISTANCE BETWEEN STIFFENERS, IN INCHES.

d_o = THE ACTUAL DISTANCE BETWEEN STIFFENERS, IN INCHES.

D = THE UNSUPPORTED DEPTH OF WEB PLATE BETWEEN COMPONENTS, IN INCHES

t = THE THICKNESS OF THE WEB PLATE, IN INCHES.

THE MOMENT OF INERTIA OF THE STIFFENER SHALL BE TAKEN ABOUT THE FACE IN CONTACT WITH THE WEB PLATE.

STIFFENERS AT POINTS OF CONCENTRATED LOADING SHALL BE PLACED IN PAIRS AND SHALL BE DESIGNED IN ACCORDANCE WITH ARTICLE 1.7.73.

THE WIDTH OF AN INTERMEDIATE STIFFENER SHALL NOT BE LESS THAN 2 INCHES PLUS 1/30 THE DEPTH OF THE GIRDER, AND IT SHALL PREFERABLY NOT BE LESS THAN 1/4 THE FULL WIDTH OF THE GIRDER COMPRESSION FLANGE. THE THICKNESS OF AN INTERMEDIATE STIFFENER SHALL NOT BE LESS THAN 1/16 ITS WIDTH. INTERMEDIATE STIFFENERS MAY BE A36 STEEL.

1.7.72 -LONGITUDINAL STIFFENERS

THE CENTERLINE OF A LONGITUDINAL STIFFENER SHALL BE D/5 FROM THE INNER SURFACE OF THE COMPRESSION FLANGE. THE LONGITUDINAL STIFFENER SHALL BE PROPORTIONED SO THAT:

$$I = Dt^3 \left(\frac{2.4d_o^2}{D^2} - 0.13 \right)$$

WHERE I = THE MINIMUM MOMENT OF INERTIA OF THE LONGITUDINAL STIFFENER ABOUT ITS EDGE IN CONTACT WITH THE WEB PLATE.

D = THE UNSUPPORTED DISTANCE BETWEEN FLANGE COMPONENTS, IN INCHES.

t = THE THICKNESS OF THE WEB PLATE, IN INCHES.

d_o = The actual distance between transverse stiffeners, in inches

The thickness of the longitudinal stiffener shall not be less than:

$$\frac{b'\sqrt{f_b}}{2250}$$

b' = Width of stiffeners

f_b = Calculated compressive bending stress in the flange.

The stress in the stiffener shall not be greater than the basic allowable bending stress for the material used in the stiffener.

Longitudinal stiffeners are usually placed on one side only of the web plate. They shall be continuous and shall be assembled full length using full penetration butt welds before attachment to the web. Transverse connection plates shall be notched to fit around longitudinal stiffeners.

1.7.73 - BEARING STIFFENERS

Over the end bearings of welded plate girders and over the intermediate bearings of continuous welded plate girders there shall be stiffeners. They shall extend as nearly as practicable to the outer edges of the flange plates. They preferably shall be made of plates placed on both sides of web plate. Bearing stiffeners shall be designed as columns, and their connection to the web shall be designed to transmit the entire end reaction to the bearings. For stiffeners consisting of two plates, the column section shall be assumed to comprise the two plates and a centrally located strip of the web plate whose width is equal to not more than 18 times its thickness. (See Art. 1.7.113 for Hybrid Girders). For stiffeners consisting of four or more plates, the column section shall be assumed to comprise the four or more plates and a centrally located strip of the web plate whose width is equal to that enclosed by the four or more plates plus a width of not more than 18 times the web plate thickness. The radius of gyration shall be computed about the axis through the center line of the web plate. The stiffeners shall be attached to the flange through which they receive their reaction by full penetration groove weld, or in the case of straight girders they may be milled to bear, only the portions of the stiffener outside the flange to web weld shall be considered effective in bearing. The thickness of the bearing stiffener plates shall not be less than:

$$\frac{b'}{12}\sqrt{\frac{F_y}{33{,}000}}$$

Bearing stiffeners may be skewed up to 30 degrees and used as connection plates. They shall not extend beyond the sole plate of the rearing.

The allowable compressive stress and the bearing pressure on the stiffeners shall not exceed the values specified in art. 1.7.1.

1.7.74 CAMBER

GIRDERS SHOULD BE CAMBERED TO COMPENSATE FOR DEAD LOAD DEFLECTIONS AS SPECIFIED IN ARTICLE 1.7.12., AND IN ADDITION THERETO THE CAMBER SHOULD BE INCREASED AND/OR DECREASED FOR THE FLANGES TO PARALLEL THE PROFILE GRADE LINE WHEN IT IS ON A VERTICAL CURVE. GENEARALLY, A SAGGING APPEARANCE OF THE LOWER FLANGE OF THE GIRDER SHOULD BE AVOIDED. CAMBER FOR FASCIA GIRDERS SHALL BE NOT LESS THAN THAT PROVIDED FOR INTERIOR GIRDERS.

TRUSSES

1.7.75-TRUSSES, GENERAL

COMPONENT PARTS OF INDIVIDUAL TRUSS MEMBERS MAY BE CONNECTED BY WELDS, OR HIGH STRENGTH BOLTS.

PREFERENCE SHOULD BE GIVEN TO TRUSSES WITH SINGLE INTERSECTION WEB SYSTEMS. MEMBERS SHALL BE SYMMETRICAL ABOUT THE CENTRAL PLANE OF THE TRUSS.

TRUSSES PREFERABLY SHALL HAVE INCLINED END POSTS. LATERALLY UNSUPPORTED HIP JOINTS SHALL BE AVOIDED.

MAIN TRUSSES SHALL BE SPACED A SUFFICIENT DISTANCE APART CENTER TO CENTER, TO BE SECURE AGAINST OVERTURNING BY THE ASSUMED LATERAL FORCES.

FOR THE CALCULATION OF STRESSES THE EFFECTIVE DEPTH SHALL BE THE DISTANCE BETWEEN CENTERS OF GRAVITY OF THE CHORDS.

1.7.76-TRUSS MEMBERS

CHORD AND WEB TRUSS MEMBERS SHALL USUALLY BE MADE IN THE FOLLOWING SHAPES:

"H" SECTIONS, MADE WITH TWO SIDE PLATES AND SOLID WEB, PERFORATED WEB OR STAY PLATES.

SINGLE BOX SECTIONS MADE WITH SIDE CHANNELS OR PLATES CONNECTED TOP AND BOTTOM WITH PERFORATED PLATES OR STAY PLATES.

SINGLE BOX SECTIONS, MADE WITH SIDE CHANNELS OR PLATES CONNECTED AT TOP WITH SOLID COVER PLATES AND AT THE BOTTOM WITH PERFORATED PLATES OR STAY PLATES.

DOUBLE BOX SECTIONS, MADE WITH SIDE CHANNELS OR PLATES CONNECTED WITH A CONVENTIONAL SOLID WEB, TOGETHER WITH TOP AND BOTTOM PERFORATED COVER PLATES OR STAY PLATES.

IF THE SHAPE OF THE TRUSS PERMITS, COMPRESSION CHORDS SHALL BE CONTINUOUS.

1.7.77 -SECONDARY STRESSES

THE DESIGN AND DETAILS SHALL BE SUCH THAT SECONDARY STRESSES WILL BE AS SMALL AS PRACTICABLE. SECONDARY STRESSES DUE TO TRUSS DISTORTION OR FLOORBEAM DEFLECTION USUALLY NEED NOT BE CONSIDERED IN ANY MEMBER THE WIDTH OF WHICH, MEASURED PARALLEL TO THE PLANE OF DISTORTION, IS LESS THAN ONE-TENTH OF ITS LENGTH. IF THE SECONDARY STRESS EXCEEDS 4,000 POUNDS PER SQUARE INCH FOR TENSION MEMBERS AND 3,000 FOR COMPRESSION MEMBERS, THE EXCESS SHALL BE TREATED AS A PRIMARY STRESS. STRESSES DUE TO THE FLEXURAL DEAD LOAD MOMENT OF THE MEMBER SHALL BE CONSIDERED AS ADDITIONAL SECONDARY STRESS.

1.7.78-DIAPHRAGMS

THERE SHALL BE DIAPHRAGMS IN THE TRUSSES AT THE END CONNECTIONS OF FLOOR BEAMS. DIAPHRAGMS PREFERABLY SHALL EXTEND THE FULL DEPTH OF THE CHORD.

THE GUSSET PLATES ENGAGING THE PEDESTAL PIN AT THE END OF THE TRUSS SHALL BE CONNECTED BY A DIAPHRAGM. SIMILARLY, THE WEBS OF THE PEDESTAL SHALL, IF PRACTICABLE, BE CONNECTED BY A DIAPHRAGM.

THERE SHALL BE A DIAPHRAGM BETWEEN GUSSET PLATES ENGAGING MAIN MEMBERS IF THE END STAY PLATE IS 4 FEET OR MORE FROM THE POINT OF INTERSECTION OF THE MEMBERS.

1.7.79-CAMBER

THE LENGTH OF THE TRUSS MEMBERS SHALL BE SUCH THAT THE CAMBER WILL BE EQUAL TO OR GREATER THAN THE DEFLECTION PRODUCED BY THE DEAD LOAD, AS SPECIFIED IN ARTICLE 1.7.13 AND IN ADDITION THERETO, THE CAMBER SHALL BE INCREASED OR DECREASED AS NECESSARY WHEN THE PROFILE GRADE IS ON A VERTICAL CURVE.

1.7.80-WORKING LINES AND GRAVITY AXES

MAIN MEMBERS SHALL BE PROPORTIONED SO THAT THEIR GRAVITY AXES WILL BE AS NEARLY AS PRACTICABLE IN THE CENTER OF THE SECTION.

IN COMPRESSION MEMBERS OF UNSYMMETRICAL SECTION, SUCH AS CHORD

SECTIONS FORMED OF SIDE SEGMENTS AND A COVER PLATE, THE GRAVITY AXIS OF THE SECTION SHALL COINCIDE AS NEARLY AS PRACTICABLE WITH THE WORKING LINE, EXCEPT THAT ECCENTRICITY MAY BE INTRODUCED TO COUNTERACT DEAD LOAD BENDING.

1.7.81 -PORTAL AND SWAY BRACING

THROUGH TRUSS SPANS SHALL HAVE PORTAL BRACING, PREFERABLY, OF THE 2-PLANE OR BOX TYPE, RIGIDLY CONNECTED TO THE END POST AND THE TOP CHORD FLANGES, AND AS DEEP AS THE CLEARANCE WILL ALLOW. IF A SINGLE PLANE PORTAL IS USED, IT SHALL BE LOCATED, PREFERABLY, IN THE CENTRAL TRANSVERSE PLANE OF THE END POSTS, WITH DIAPHRAGMS BETWEEN THE WEBS OF THE POSTS TO PROVIDE FOR A DISTRIBUTION OF THE PORTAL STRESSES. THE PORTAL BRACING SHALL BE DESIGNED TO TAKE THE FULL END REACTION OF THE TOP CHORD LATERAL SYSTEM AND THE END POSTS SHALL BE DESIGNED TO TRANSFER THIS REACTION TO THE TRUSS BEARINGS.

THROUGH TRUSS SPANS SHALL HAVE SWAY BRACING 5 FEET OR MORE DEEP AT EACH INTERMEDIATE PANEL. TOP LATERAL STRUTS SHALL BE AT LEAST AS DEEP AS THE TOP CHORD.

DECK TRUSS SPANS SHALL HAVE SWAY BRACING IN THE PLANE OF THE END POSTS AND AT ALL INTERMEDIATE PANEL POINTS. THIS BRACING SHALL EXTEND THE FULL DEPTH OF THE TRUSSES BELOW THE FLOOR SYSTEM. THE END SWAY BRACING SHALL BE PROPORTIONED TO CARRY THE ENTIRE UPPER LATERAL STRESS TO THE SUPPORTS THROUGH THE END POSTS OF THE TRUSS.

1.7.82 -LATERAL BRACING

FOR REQUIREMENTS FOR LATERAL BRACING REFER TO ARTICLE 1.7.22.

1.7.83 -PERFORATED COVER PLATES

THE SHEARING FORCE NORMAL TO THE MEMBER IN THE PLANES OF CONTINUOUS PERFORATED PLATES SHALL BE ASSUMED DIVIDED EQUALLY BETWEEN ALL SUCH PARALLEL PLANES. THE SHEARING FORCE SHALL INCLUDE THAT DUE TO THE WEIGHT OF THE MEMBERS PLUS ANY OTHER EXTERNAL FORCE. FOR COMPRESSION MEMBERS, AN ADDITIONAL FORCE SHALL BE ADDED AS OBTAINED BY THE FOLLOWING FORMULA:

$$V = \frac{P}{100}\left[\frac{100}{\frac{l}{r}+10} + \frac{\frac{l}{r}}{\frac{3,300,000}{F_y}}\right]$$

IN THE ABOVE EXPRESSION:

V = NORMAL SHEARING STRESS IN POUNDS.

P = ALLOWABLE COMPRESSIVE AXIAL LOAD ON MEMBERS, IN POUNDS.

l = LENGTH OF MEMBER IN INCHES.

r = RADIUS OF GYRATION OF SECTION ABOUT THE AXIS PERPENDICULAR TO PLANE OF THE PERFORATED PLATE IN INCHES.

F_y = SPECIFIED MINIMUM YIELD POINT OF TYPE OF STEEL BEING USED.

THE FOLLOWING PROVISIONS SHALL GOVERN THE DESIGN OF PERFORATED COVER PLATES:

(1) THE RATIO OF LENGTH, IN DIRECTION OF STRESS, TO WIDTH OF PERFORATION, SHALL NOT EXCEED TWO.

(2) THE CLEAR DISTANCE BETWEEN PERFORATIONS IN THE DIRECTION OF STRESS, SHALL NOT BE LESS THAN THE DISTANCE BETWEEN POINTS OF SUPPORT.

(3) THE CLEAR DISTANCE BETWEEN THE END PERFORATION AND THE END OF THE COVER PLATE SHALL NOT BE LESS THAN 1.25 TIMES THE DISTANCE BETWEEN POINTS OF SUPPORT.

(4) THE POINT OF SUPPORT SHALL BE THE INNER LINE OF FILLET WELDS CONNECTING THE PERFORATED PLATE TO THE FLANGES. FOR PLATES BUTT WELDED TO THE FLANGE EDGE OF ROLLED SEGMENTS THE POINT OF SUPPORT MAY BE TAKEN AS THE WELD WHENEVER THE RATIO OF THE OUTSTANDING FLANGE WIDTH TO FLANGE THICKNESS OF THE ROLLED SEGMENT IS LESS THAN SEVEN. OTHERWISE POINT OF SUPPORT SHALL BE THE ROOT OF THE FLANGE OF THE ROLLED SEGMENT.

(5) THE PERIPHERY OF THE PERFORATION AT ALL POINTS SHALL HAVE A MINIMUM RADIUS OF 1 1/2 INCHES.

(6) FOR THICKNESS OF METAL SEE ARTICLE 1.7.88.

1.7.84 -GUSSET PLATES

THE FASTENERS CONNECTING EACH MEMBER SHALL BE SYMMETRICAL WITH THE AXIS OF THE MEMBER, SO FAR AS PRACTICABLE, AND THE FULL DEVELOPMENT OF THE ELEMENTS OF THE MEMBER SHALL BE GIVEN CONSIDERATION. THE GUSSET PLATES SHALL BE OF AMPLE THICKNESS TO RESIST SHEAR, DIRECT STRESS, AND FLEXURE, ACTING ON THE WEAKEST OR CRITICAL SECTION OF MAXIMUM STRESS.

RE-ENTRANT CUTS, SHALL BE AVOIDED AS FAR AS PRACTICABLE.

IF THE LENGTH OF UNSUPPORTED EDGE OF A GUSSET PLATE EXCEEDS THE VALUE OF THE EXPRESSION $11,000/\sqrt{F_y}$ TIMES ITS THICKNESS, THE EDGE SHALL

BE STIFFENED. LISTED BELOW ARE THE VALUES OF THE EXPRESSION $11,000/\sqrt{F_y}$ FOR THE FOLLOWING GRADES OF STEEL:

36,000 PSI., Y.P. MIN. 58
42,000 PSI., Y.P. MIN. 54
46,000 PSI., Y.P. MIN. 51
50,000 PSI., Y.P. MIN. 49

1.7.85 -HALF-THROUGH TRUSS SPANS (DELETED)

1.7.86 -FASTENER PITCH IN ENDS OF COMPRESSION MEMBERS (DELETED)

1.7.87 -NET SECTION OF TENSION MEMBERS AT SPLICES OR CONNECTIONS

REFER TO ARTICLE 1.7.19

1.7.88 -COMPRESSION MEMBERS-THICKNESS OF METAL

COMPRESSION MEMBERS SHALL BE SO DESIGNED THAT THE MAIN ELEMENTS OF THE SECTION WILL BE CONNECTED DIRECTLY TO THE GUSSET PLATES OR OTHERS MEMBERS.

THE CENTER OF GRAVITY OF A BUILT UP SECTION SHALL COINCIDE AS NEARLY AS PRACTICABLE WITH THE CENTER OF THE SECTION. PREFERABLY, SEGMENTS SHALL BE CONNECTED BY SOLID WEBS OR PERFORATED COVER PLATES.

PLATES SUPPORTED ON ONE SIDE AND PERFORATED PLATES. FOR OUTSTANDING PLATES AND PERFORATED PLATES AT THE PERFORATIONS, THE b/t RATIO OF THE PLATES, WHEN USED IN COMPRESSION, SHALL NOT BE GREATER THAN THE VALUE OBTAINED BY THE FORMULA:

$b/t = \frac{1625}{\sqrt{f_a}}$ BUT IN NO CASE SHALL b/t BE GREATER THAN 12 FOR MAIN MEMBERS AND 16 FOR SECONDARY MEMBERS.

(NOTE-b IS THE DISTANCE FROM THE EDGE OF PLATE OR EDGE OF PERFORATION TO THE POINT OF SUPPORT.)

WHEN THE COMPRESSIVE STRESS EQUALS THE LIMITING FACTOR $.55F_y/1.25$, THE b/t RATIO OF THE SEGMENTS INDICATED ABOVE SHALL NOT BE GREATER THAN THE RATIOS SHOWN FOR THE FOLLOWING GRADES OF STEEL:

36,000 PSI., Y.P. MIN. b/t = 12
42,000 PSI., Y.P. MIN.
TO 50,000 PSI., Y.P. MIN. b/t = 11

PLATES SUPPORTED ON TWO EDGES OR WEBS OF MAIN COMPONENT SEGMENTS.

FOR MEMBERS OF BOX SHAPE, CONSISTING OF MAIN PLATES, ROLLED SECTIONS, OR MADE UP COMPONENT SEGMENTS, WITH COVER PLATES, THE b/t RATIO OF THE MAIN PLATES OR WEBS OF THE SEGMENTS, WHEN USED IN COMPRESSION SHALL NOT BE GREATER THAN THE VALUE OBTAINED BY USE OF THE FORMULA:

$b/t = \frac{4000}{\sqrt{f_a}}$ BUT IN NO CASE SHALL b/t BE GREATER THAN 45.

(NOTE- b IS THE DISTANCE BETWEEN POINTS OF SUPPORT FOR THE PLATE AND BETWEEN ROOTS OF FLANGES FOR THE WEBS OF ROLLED SEGMENTS.)

WHEN THE COMPRESSIVE STRESSES EQUAL THE LIMITING FACTOR $.55F_y$ /1.25, THE b/t RATIO OF THE PLATES AND SEGMENTS INDICATED ABOVE SHALL NOT BE GREATER THAN THE RATIOS SHOWN FOR THE FOLLOWING GRADES OF STEEL:

36,000 PSI., Y.P. MIN. b/t = 32
42,000 PSI., Y.P. MIN. b/t = 29
46,000 PSI., Y.P. MIN. b/t = 28
50,000 PSI., Y.P. MIN. b/t = 27

SOLID COVER PLATES SUPPORTED ON TWO EDGES OR WEBS CONNECTING MAIN MEMBERS OR SEGMENTS. FOR MEMBERS OF H OR BOX SHAPE CONSISTING OF SOLID COVER PLATES OR SOLID WEBS CONNECTING MAIN PLATES OR SEGMENTS, THE b/t RATIO OF THE SOLID COVER PLATES OR WEBS WHEN USED IN COMPRESSION SHALL NOT BE GREATER THAN THE VALUE OBTAINED BY USE OF THE FORMULA:

$b/t = \frac{5000}{\sqrt{f_a}}$ BUT IN NO CASE SHALL b/t BE GREATER THAN 50.

(NOTE- b IS THE UNSUPPORTED DISTANCE BETWEEN POINTS OF SUPPORT.)

WHEN THE COMPRESSIVE STRESSES EQUAL THE LIMITING FACTOR $.55F_y$ /1.25, THE b/t RATIO OF THE COVER PLATE AND WEBS INDICATED ABOVE SHALL NOT BE GREATER THAN THE RATIOS SHOWN FOR THE FOLLOWING GRADES OF STEEL:

36,000 PSI., Y.P. MIN. b/t = 40
42,000 PSI., Y.P. MIN. b/t = 37
46,000 PSI., Y.P. MIN. b/t = 35
50,000 PSI., Y.P. MIN. b/t = 34

PERFORATED COVER PLATES SUPPORTED ON TWO EDGES. FOR MEMBERS OF BOX SHAPE CONSISTING OF PERFORATED COVER PLATES CONNECTING MAIN PLATES OR SEGMENTS, THE b/t RATIO OF THE PERFORATED COVER PLATES WHEN USED IN COMPRESSION SHALL NOT BE GREATER THAN THE VALUE OBTAINED BY USE OF THE FORMULA:

$b/t = \frac{6000}{\sqrt{f_a}}$ BUT IN NO CASE SHALL b/t BE GREATER THAN 55.

(NOTE- b IS THE DISTANCE BETWEEN POINTS OF SUPPORT. ATTENTION IS DIRECTED TO REQUIREMENTS FOR PLATE THICKNESS AT PERFORATIONS, NAMELY PLATE SUPPORTED ON ONE SIDE, WHICH ALSO SHALL BE SATISFIED.)

WHEN THE COMPRESSIVE STRESSES EQUAL THE LIMITING FACTOR $.55F_y$ /1.25, THE b/t RATIO OF THE PERFORATED COVER PLATES SHALL NOT BE GREATER THAN THE RATIOS SHOWN FOR THE FOLLOWING GRADES OF STEEL:

36,000 PSI., Y.P. MIN. b/t = 48
42,000 PSI., Y.P. MIN. b/t = 44
46,000 PSI., Y.P. MIN. b/t = 42
50,000 PSI., Y.P. MIN. b/t = 41

IN THE ABOVE EXPRESSIONS-

f_a = THE CALCULATED COMPRESSIVE STRESS.
b = THE WIDTH (DEFINED AS INDICATED FOR EACH EXPRESSION).
t = THE PLATE OR WEB THICKNESS.

THE POINT OF SUPPORT SHALL BE THE INNER LINE OF FASTENERS OR FILLET WELDS CONNECTING THE PLATE TO THE MAIN SEGMENT. FOR PLATES BUTT WELDED TO THE FLANGE EDGE OF ROLLED SEGMENTS THE POINT OF SUPPORT MAY BE TAKEN AS THE WELD WHENEVER THE RATIO OF OUTSTANDING FLANGE WIDTH TO FLANGE THICKNESS OF THE ROLLED SEGMENT IS LESS THAN SEVEN. OTHERWISE POINT OF SUPPORT SHALL BE THE ROOT OF FLANGE OF ROLLED SEGMENT. TERMINATIONS OF THE BUTT WELDS ARE TO BE GROUND SMOOTH.

1.7.89-STAY PLATES

THE SEPARATE SEGMENTS OF TENSION MEMBERS COMPOSED OF SHAPES OR PLATES MAY BE CONNECTED BY PERFORATED PLATES OR BY STAY PLATES. END STAY PLATES SHALL HAVE A LENGTH NOT LESS THAN 1 1/4 TIMES THE DISTANCE BETWEEN POINTS OF SUPPORT AND INTERMEDIATE STAY PLATES SHALL HAVE A LENGTH NOT LESS THAN 3/4 OF THAT DISTANCE. THE CLEAR DISTANCE BETWEEN STAY PLATES ON TENSION MEMBERS SHALL NOT EXCEED 3 FEET.

THE SEPERATE SEGMENTS OF LATERAL STRUTS AND OTHER SECONDARY MEMBERS MAY BE CONNECTED BY STAY PLATES. THE LENGTH OF THESE STAY PLATES, BOTH END AND INTERMEDIATE SHALL BE NOT LESS THAN 3/4 OF THE DISTANCE BETWEEN POINTS OF SUPPORT.

THE POINT OF SUPPORT SHALL BE THE INNER LINE WELDS CONNECTING THE STAY PLATES. FOR STAY PLATES BUTT WELDED TO THE FLANGE EDGE OF ROLLED SEGMENTS THE POINT OF SUPPORT MAY BE TAKEN AS THE WELD WHENEVER THE RATIO OF OUTSTANDING FLANGE WIDTH TO FLANGE THICKNESS OF THE ROLLED SEGMENT IS LESS THAN SEVEN. OTHERWISE THE POINT OF SUPPORT SHALL BE THE ROOT OF FLANGE OF ROLLED SEGMENT. WHEN STAY PLATES ARE BUTT WELDED TO ROLLED SEGMENTS OF A MEMBER, THE ALLOWABLE STRESS IN THE MEMBER SHALL BE DETERMINED IN ACCORDANCE WITH ARTICLE 1.7.3. TERMINATIONS OF BUTT WELDS SHALL BE GROUND SMOOTH.

THE THICKNESS OF STAY PLATES SHALL BE NOT LESS THAN 1/50 OF THE DISTANCE BETWEEN POINTS OF SUPPORT FOR MAIN MEMBERS, AND 1/60 OF THAT DISTANCE FOR BRACING MEMBERS. STAY PLATES SHALL BE CONNECTED BY

CONTINUOUS WELD.

IN THE DESIGN OF THE END CONNECTIONS OF MEMBERS WITH STAY PLATES, CONSIDERATION SHALL BE GIVEN TO THE REQUIREMENTS OF ARTICLE 1.7.79.

RIBBED ARCHES

1.7.90 -THICKNESS OF WEB PLATES, SOLID RIB ARCHES

THE THICKNESS RATIO D/t OF EACH WEB PLATE IN SOLID RIB ARCHES HAVING NO LONGITUDINAL STIFFENERS SHALL NOT BE GREATER THAN THE VALUE OBTAINED BY USE OF THE FOLLOWING FORMULA:

$D/t = \frac{7200}{\sqrt{f_a}}$ BUT IN NO CASE SHALL D/t BE GREATER THAN 60.

THE THICKNESS RATIO D/t OF WEB PLATES IN SOLID RIB ARCHES EQUIPPED WITH LONGITUDINAL STIFFENERS, THAT IS WHEN THE WEB IS REINFORCED ALONG ITS AXIS WITH A LONGITUDINAL STIFFENER OF AMPLE CROSS-SECTIONAL AREA AND RIGIDITY, SHALL NOT BE GREATER THAN TWICE THE VALUE OBTAINED BY USE OF THE ABOVE FORMULA.

WHEN THE COMPRESSIVE STRESSES EQUAL THE LIMITING FACTOR $.55F_y$ /1.25 THE D/t RATIO OF THE WEB PLATES SHALL NOT BE GREATER THAN THE RATIOS SHOWN FOR THE FOLLOWING GRADES OF STEEL:

	WITHOUT LONGIT. REINF.	WITH LONGIT. REINF.
36,000 PSI., Y.P. MIN.	D/t = 57	D/t = 114
42,000 PSI., Y.P. MIN.	D/t = 53	D/t = 106
46,000 PSI., Y.P. MIN.	D/t = 51	D/t = 102
50,000 PSI., Y.P. MIN.	D/t = 48	D/t = 96

WHERE:

D = DEPTH OF WEB IN INCHES.
t = THICKNESS OF WEB IN INCHES.

BENTS AND TOWERS

1.7.91-BENTS AND TOWERS, GENERAL

BENTS, PREFERABLY SHALL BE COMPOSED OF TWO SUPPORTING COLUMNS, AND THE BENTS USUALLY SHALL BE UNITED IN PAIRS TO FORM TOWERS. THE DESIGN OF MEMBERS FOR BENTS AND TOWERS IS GOVERNED BY THE APPLICABLE ARTICLES UNDER

"TRUSSES" AND "DETAILS OF DESIGN".

1.7.92 SINGLE BENTS

SINGLE BENTS SHALL HAVE HINGED ENDS OR ELSE SHALL BE DESIGNED TO RESIST BENDING.

1.7.93 -BATTER

BENTS, PREFERABLY, SHALL HAVE A SUFFICIENT SPREAD AT THE BASE TO PREVENT UPLIFT UNDER THE ASSUMED LATERAL LOADINGS. IN GENERAL, THE WIDTH OF A BENT AT ITS BASE SHALL BE NOT LESS THAN ONE-THIRD OF ITS HEIGHT.

1.7.94 -BRACING

TOWERS SHALL BE BRACED, BOTH TRANSVERSELY AND LONGITUDINALLY, WITH STIFF MEMBERS HAVING EITHER WELDED OR HIGH STRENGTH BOLTED CONNECTIONS. THE SECTIONS OF MEMBERS OF LONGITUDINAL BRACING IN EACH PANEL SHALL NOT BE LESS THAN THOSE OF THE MEMBERS IN CORRESPONDING PANELS OF THE TRANSVERSE BRACING.

THE BRACING OF LONG COLUMNS SHALL BE DESIGNED TO FIX THE COLUMN ABOUT BOTH AXES AT OR NEAR THE SAME POINT.

COLUMN SPLICES SHALL BE AT OR CLOSE ABOVE THE PANEL POINTS OF THE BRACING.

HORIZONTAL DIAGONAL BRACING SHALL BE PLACED IN ALL TOWERS HAVING MORE THAN TWO VERTICAL PANELS, AT ALTERNATE INTERMEDIATE PANEL POINTS.

1.7.95 -BOTTOM STRUTS

THE BOTTOM STRUTS OF TOWERS SHALL BE STRONG ENOUGH TO SLIDE THE MOVABLE SHOES WITH THE STRUCTURE UNLOADED, THE COEFFICIENT OF FRICTION BEING ASSUMED AT 0.25. PROVISION FOR EXPANSION OF THE TOWER BRACING SHALL BE MADE IN THE COLUMN BEARINGS.

COMPOSITE GIRDERS

1.7.96 -COMPOSITE GIRDERS, GENERAL

THIS SECTION PERTAINS TO STRUCTURES COMPOSED OF STEEL GIRDERS WITH CONCRETE SLABS CONNECTED BY SHEAR CONNECTORS.

GENERAL SPECIFICATIONS PERTAINING TO THE DESIGN OF CONCRETE AND STEEL STRUCTURES SHALL APPLY TO STRUCTURES UTILIZING COMPOSITE GIRDERS WHERE SUCH SPECIFICATIONS ARE APPLICABLE. COMPOSITE GIRDERS AND SLABS SHALL BE DESIGNED AND THE STRESSES COMPUTED BY THE COMPOSITE MOMENT OF INERTIA METHOD AND SHALL BE CONSISTENT WITH THE PREDETERMINED PROPERTIES OF THE VARIOUS MATERIALS USED.

THE RATIO OF THE MODULI OF ELASTICITY OF STEEL (29,000,000 PSI) TO THOSE OF CONCRETE OF VARIOUS DESIGN STRENGTHS SHALL BE AS FOLLOWS:

f'_c = UNIT ULTIMATE COMPRESSIVE STRENGTH OF CONCRETE AS DETERMINED BY CYLINDER TESTS AT THE AGE OF 28 DAYS, PSI.

n = RATIO OF MODULUS OF ELASTICITY OF STEEL TO THAT OF CONCRETE. THE VALUE OF n, AS A FUNCTION OF THE ULTIMATE CYLINDER STRENGTH OF CONCRETE, SHALL BE ASSUMED AS FOLLOWS:

f'_c = 2000 - 2400	n = 15
f'_c = 2500 - 2900	n = 12
f'_c = 3000 - 3900	n = 10
f'_c = 4000 - 4900	n = 8
f'_c = 5000 OR MORE	n = 6

THE EFFECT OF CREEP SHALL BE CONSIDERED IN THE DESIGN OF COMPOSITE GIRDERS WHICH HAVE DEAD LOADS ACTING ON THE COMPOSITE SECTION. IN SUCH STRUCTURES, STRESSES AND HORIZONTAL SHEARS PRODUCED BY DEAD LOADS ACTING ON THE COMPOSITE SECTION SHALL BE COMPUTED FOR "n" AS GIVEN ABOVE OR FOR THIS VALUE MULTIPLIED BY 3, WHICHEVER GIVES THE HIGHER STRESSES AND SHEARS.

IF CONCRETE WITH EXPANSIVE CHARACTERISTICS IS USED, COMPOSITE DESIGN SHOULD BE USED WITH CAUTION AND PROVISION MUST BE MADE IN THE DESIGN TO ACCOMMODATE THE EXPANSION.

COMPOSITE SECTIONS SHOULD PREFERABLY BE PROPORTIONED SO THAT THE NEUTRAL AXIS LIES BELOW THE TOP SURFACE OF THE STEEL BEAM. IF CONCRETE IS ON THE TENSION SIDE OF THE NEUTRAL AXIS, IT SHALL NOT BE CONSIDERED IN COMPUTING MOMENTS OF INERTIA OR RESISTING MOMENTS EXCEPT FOR DEFLECTION CALCULATIONS. MECHANICAL ANCHORAGES SHALL BE PROVIDED TO TIE THE SECTIONS TOGETHER AND TO DEVELOP STRESSES ON THE PLANE JOINING THE CONCRETE AND THE STEEL.

THE STEEL BEAMS, ESPECIALLY IF NOT SUPPORTED BY INTERMEDIATE FALSE-WORK SHALL BE INVESTIGATED FOR STABILITY DURING THE TIME THE CONCRETE IS IN PLACE AND BEFORE IT HAS HARDENED.

1.7.97 - SHEAR CONNECTORS

THE MECHANICAL MEANS WHICH ARE USED AT THE JUNCTION OF THE GIRDER AND SLAB FOR THE PURPOSE OF DEVELOPING THE SHEAR RESISTANCE NECESSARY TO

PRODUCE COMPOSITE ACTION SHALL CONFORM TO THE CURRENT REQUIREMENTS FOR STUD SHEAR CONNECTORS OF THE NEW YORK STATE PUBLIC WORKS SPECS. THE SHEAR CONNECTORS SHALL BE 3/4 IN. DIAMETER STUD SHEAR CONNECTORS HAVING A MINIMUM LENGTH OF 4 IN.

THE CAPACITY OF THE WELDS AT PERMISSIBLE WORKING STRESSES SHALL EQUAL OR EXCEED THE DESIGN LOAD "Z" OF THE SHEAR CONNECTOR. THE CAPACITY OF STUD SHEAR CONNECTORS SHALL BE AS GIVEN IN ARTICLE 1.7.101.

THE CLEAR DEPTH OF CONCRETE COVER OVER THE TOPS OF THE SHEAR CONNECTORS SHALL BE NOT LESS THAN 2 INCHES. SHEAR CONNECTORS SHALL PENETRATE AT LEAST 2 INCHES ABOVE BOTTOM OF SLAB.

THE CLEAR DISTANCE BETWEEN THE EDGE OF A GIRDER FLANGE AND THE EDGE OF THE SHEAR CONNECTORS SHALL BE NOT LESS THAN ONE INCH.

1.7.98)-EFFECTIVE FLANGE WIDTH

IN COMPOSITE GIRDER CONSTRUCTION THE ASSUMED EFFECTIVE WIDTH OF THE SLAB AS A T-BEAM FLANGE SHALL NOT EXCEED THE FOLLOWING:

(1) ONE-FOURTH OF THE SPAN LENGTH OF THE GIRDER.
(2) THE DISTANCE CENTER TO CENTER OF GIRDERS.
(3) TWELVE TIMES THE LEAST THICKNESS OF THE SLAB.

FOR GIRDERS HAVING A FLANGE ON ONE SIDE ONLY, THE EFFECTIVE FLANGE WIDTH SHALL NOT EXCEED ONE-TWELFTH OF THE SPAN LENGTH OF THE GIRDER, NOR SIX TIMES THE THICKNESS OF THE SLAB, NOR ONE-HALF THE DISTANCE CENTER TO CENTER OF THE NEXT GIRDER.

1.7.99)-STRESSES

MAXIMUM COMPRESSIVE AND TENSILE STRESSES IN GIRDERS WHICH ARE NOT PROVIDED WITH TEMPORARY SUPPORTS DURING THE PLACING OF THE PERMANENT DEAD LOAD, SHALL BE THE SUM OF THE STRESSES PRODUCED BY THE DEAD LOADS ACTING ON THE STEEL GIRDERS ALONE AND THE STRESSES PRODUCED BY THE SUPERIMPOSED LOADS ACTING ON THE COMPOSITE GIRDER. WHEN GIRDERS ARE PROVIDED WITH EFFECTIVE INTERMEDIATE SUPPORTS WHICH ARE KEPT IN PLACE UNTIL THE CONCRETE HAS ATTAINED 75 PER CENT OF ITS REQUIRED 28-DAY STRENGTH, THE DEAD AND LIVE LOAD STRESSES SHALL BE COMPUTED ON THE BASIS OF THE COMPOSITE SECTION.

IN CONTINUOUS SPANS, THE POSITIVE MOMENT PORTION MAY BE DESIGNED WITH COMPOSITE SECTIONS AS IN SIMPLE SPANS. SHEAR CONNECTORS SHALL BE PROVIDED IN THE NEGATIVE MOMENT PORTION IN WHICH THE REINFORCEMENT STEEL EMBEDDED IN THE CONCRETE IS CONSIDERED A PART OF THE COMPOSITE SECTION. IN CASE THE REINFORCEMENT STEEL EMBEDDED IN THE CONCRETE IS NOT USED IN COMPUTING SECTION PROPERTIES FOR NEGATIVE MOMENTS SHEAR CONNECTORS NEED NOT BE PROVIDED IN THESE PORTIONS OF THE SPANS, BUT ADDITIONAL CONNECTORS SHALL

BE PLACED IN THE REGION OF THE POINT OF DEAD LOAD CONTRAFLEXURE. SHEAR CONNECTORS SHALL BE PROVIDED IN ACCORDANCE WITH ARTICLE 1.7.100(A)(3).

1.7.100 -SHEAR

A. HORIZONTAL SHEAR

THE MAXIMUM PITCH OF SHEAR CONNECTORS SHALL NOT EXCEED 24 INCHES, EXCEPT OVER THE INTERIOR SUPPORTS OF CONTINUOS BEAMS WHERE WIDER SPACING MAY BE USED TO AVOID PLACING CONNECTORS AT LOCATIONS OF HIGH STRESS IN THE TENSION FLANGE.

RESISTANCE TO HORIZONTAL SHEAR SHALL BE PROVIDED BY MECHANICAL SHEAR CONNECTORS AT THE JUNCTION OF THE CONCRETE SLAB AND THE STEEL GIRDER. THE SHEAR CONNECTORS SHALL BE MECHANICAL DEVICES PLACED TRANSVERSELY ACROSS THE FLANGE OF THE GIRDER SPACED AT REGULAR OR VARIABLE INTERVALS. THE SHEAR CONNECTORS SHALL BE DESIGNED FOR FATIGUE AND CHECKED FOR ULTIMATE STRENGTH.

(1) FATIGUE

THE RANGE OF HORIZONTAL SHEAR SHALL BE COMPUTED BY THE FORMULA:

$$S_r = \frac{V_r Q}{I} \qquad \text{IN WHICH}$$

S_r = THE RANGE OF HORIZONTAL SHEAR PER LINEAR INCH AT THE JUNCTION OF THE SLAB AND GIRDER AT THE POINT IN THE SPAN UNDER CONSIDERATION.

V_r = THE RANGE OF SHEAR DUE TO LIVE LOADS AND IMPACT. AT ANY SECTION, THE RANGE OF SHEAR SHALL BE TAKEN AS THE DIFFERENCE IN THE MINIMUM AND MAXIMUM SHEAR ENVELOPES (EXCLUDING DEAD LOADS).

Q = THE STATICAL MOMENT ABOUT THE NUETRAL AXIS OF THE COMPOSITE SECTION, OF THE TRANSFORMED COMPRESSIVE CONCRETE AREA OR THE AREA OF REINFORCEMENT EMBEDDED IN THE CONCRETE FOR NEGATIVE MOMENT.

I = THE MOMENT OF INERTIA OF THE TRANSFORMED COMPOSITE GIRDER IN POSITIVE MOMENT REGIONS OR THE MOMENT OF INERTIA PROVIDED BY THE STEEL BEAM INCLUDING OR EXCLUDING THE AREA OF REINFORCEMENT EMBEDDED IN THE CONCRETE IN NEGATIVE MOMENT REGIONS.

(IN THE ABOVE, THE COMPRESSIVE CONCRETE AREA IS TRANSFORMED INTO AN EQUIVALENT AREA OF STEEL BY DIVIDING THE EFFECTIVE CONCRETE FLANGE WIDTH BY THE MODULAR RATIO, "n").

THE ALLOWABLE RANGE OF HORIZONTAL SHEAR, "Z_r", IN POUNDS ON AN INDIVIDUAL CONNECTOR IS AS FOLLOWS:

$$Z_r = \alpha d^2 \quad \text{IN WHICH}$$

d = DIAMETER OF STUD, IN INCHES

α = 13,000 FOR 100,000 CYCLES
10,600 FOR 500,000 CYCLES
7,850 FOR 2,000,000 CYCLES

THE REQUIRED PITCH OF SHEAR CONNECTORS IS DETERMINED BY DIVIDING THE ALLOWABLE RANGE OF HORIZONTAL SHEAR OF ALL CONNECTORS AT ONE TRANSVERSE GIRDER CROSSECTION ($\sum Z_r$) BY THE HORIZONTAL RANGE OF SHEAR S PER LINEAR INCH. OVER THE INTERIOR SUPPORTS OF CONTINUOUS BEAMS THE PITCH MAY BE MODIFIED TO AVOID PLACING THE CONNECTORS AT LOCATIONS OF HIGH STRESSES IN THE TENSION FLANGE PROVIDED THAT THE TOTAL NUMBER OF CONNECTORS REMAINS UNCHANGED.

(2) ULTIMATE STRENGTH

THE NUMBER OF CONNECTORS SO PROVIDED FOR FATIGUE SHALL BE CHECKED TO ENSURE THAT ADEQUATE CONNECTORS ARE PROVIDED FOR ULTIMATE STRENGTH. THE NUMBER OF SHEAR CONNECTORS REQUIRED BETWEEN THE POINTS OF MAXIMUM POSITIVE MOMENT AND THE END SUPPORTS OR DEAD LOAD POINTS OF CONTRAFLEXURE, AND BETWEEN POINTS OF MAXIMUM NEGATIVE MOMENT AND THE DEAD LOAD POINTS OF CONTRAFLEXURE SHALL EQUAL OR EXCEED THE NUMBER GIVEN BY THE FORMULA:

$$N = \frac{P}{\emptyset S_u}$$

WHERE N = THE NUMBER OF CONNECTORS BETWEEN POINTS OF MAXIMUM POSITIVE MOMENT AND ADJACENT END SUPPORTS OR DEAD LOAD POINTS OF CONTRAFLEXURE, OR BETWEEN POINTS OF MAXIMUM NEGATIVE MOMENT AND ADJACENT DEAD LOAD POINTS OF CONTRAFLEXURE.

S_u = THE ULTIMATE STRENGTH OF THE SHEAR CONNECTOR AS GIVEN BELOW.

$\emptyset$ = A REDUCTION FACTOR = 0.85.

P = FORCE IN THE SLAB AS DEFINED HEREAFTER AS P_1, P_2, OR P_3.

AT POINTS OF MAXIMUM POSITIVE MOMENT, THE FORCE IN THE SLAB IS TAKEN AS THE SMALLER VALUE OF THE FORMULAS:

$$P_1 = A_s F_y$$

OR

$$P_2 = 0.85 f'_c bc$$

WHERE A_s = TOTAL AREA OF THE STEEL SECTION INCLUDING COVERPLATES.

F_y = SPECIFIED MINIMUM YIELD POINT OF STEEL BEING USED.

f'_c = COMPRESSIVE STRENGTH OF CONCRETE AT AGE OF 28 DAYS.

b = EFFECTIVE FLANGE WIDTH GIVEN IN ART. 1.7.98.

c = THICKNESS OF CONCRETE SLAB.

AT POINTS OF MAXIMUM NEGATIVE MOMENT THE FORCE IN THE SLAB IS TAKEN AS:

$$P_3 = A_s^r F_y^r \quad \text{IN WHICH}$$

A_s^r = TOTAL AREA OF LONGITUDINAL REINFORCING STEEL AT THE INTERIOR SUPPORT WITHIN THE EFFECTIVE FLANGE WIDTH.

F_y^r = SPECIFIED MINIMUM YIELD POINT OF THE REINFORCING STEEL.

THE ULTIMATE STRENGTH OF THE SHEAR CONNECTOR, IS GIVEN AS FOLLOWS:
WELDED STUDS:

$$S_u = 930 d^2 \sqrt{f'_c}$$

WHERE S_u = ULTIMATE STRENGTH OF INDIVIDUAL SHEAR CONNECTORS, IN POUNDS.
f'_c = COMPRESSIVE STRENGTH OF CONCRETE AT 28 DAYS, PSI.

d = DIAMETER OF STUD IN INCHES.

(3) ADDITIONAL CONNECTORS TO DEVELOP SLAB STRESS

THE NUMBER OF ADDITIONAL CONNECTORS REQUIRED AT POINTS OF CONTRAFLEXURE, WHEN REINFORCEMENT STEEL EMBEDDED IN THE CONCRETE IS NOT USED IN COMPUTING SECTION PROPERTIES FOR NEGATIVE MOMENTS SHALL BE COMPUTED BY THE FORMULA:

$$N_c = \frac{A_r f_r}{Z_r}$$

WHERE N_c = NUMBER OF ADDITIONAL CONNECTORS FOR EACH BEAM AT POINT OF CONTRAFLEXURE.

A_r = TOTAL AREA OF LONGITUDINAL SLAB REINFORCEMENT STEE FOR EACH BEAM OVER INTERIOR SUPPORT.

f_r = RANGE OF STRESS DUE TO LIVE LOAD AND IMPACT, IN THE SLAB REINFORCEMENT OVER THE SUPPORT (IN LIEU OF MORE ACCURATE COMPUTATIONS, f_r MAY BE TAKEN AS EQUAL TO 10,000 PSI).

Z_r = THE ALLOWABLE RANGE OF HORIZONTAL SHEAR ON AN INDIVIDUAL SHEAR CONNECTOR.

THE ADDITIONAL CONNECTORS, N_c, SHALL BE PLACED ADJACENT TO THE POINT OF DEAD LOAD CONTRAFLEXURE WITHIN A DISTANCE EQUAL TO 1/3 THE EFFECTIVE SLAB WIDTH, THAT IS, PLACED EITHER SIDE OF THIS POINT OR CENTERED ABOUT IT.

B. VERTICAL SHEAR

THE INTENSITY OF UNIT SHEARING STRESS IN A COMPOSITE GIRDER MAY BE DETERMINED ON THE BASIS THAT THE WEB OF THE STEEL GIRDER CARRIES THE TOTAL EXTERNAL SHEAR, NEGLECTING THE EFFECTS OF THE STEEL FLANGES AND OF THE CONCRETE SLAB. THE SHEAR MAY BE ASSUMED TO BE UNIFORMLY DISTRIBUTED THROUGHOUT THE GROSS AREA OF THE WEB.

1.7.101 -DEFLECTION

THE PROVISIONS OF ARTICLE 1.7.12 IN REGARD TO DEFLECTIONS FROM LIVE LOAD PLUS IMPACT ALSO SHALL BE APPLICABLE TO COMPOSITE GIRDERS.

WHEN THE GIRDERS ARE NOT PROVIDED WITH FALSEWORK OR OTHER EFFECTIVE INTERMEDIATE SUPPORT DURING THE PLACING OF THE CONCRETE SLAB, THE DEFLECTION DUE TO THE WEIGHT OF THE SLAB AND OTHER PERMANENT DEAD LOADS ADDED BEFORE THE CONCRETE HAS ATTAINED 75 PER CENT OF ITS REQUIRED 28-DAY STRENGTH SHALL BE COMPUTED ON THE BASIS OF NON-COMPOSITE ACTION.

1.7.102 -COMPOSITE BOX GIRDERS, GENERAL

THIS SECTION PERTAINS TO THE DESIGN OF SIMPLE AND CONTINUOUS SPAN STEEL-CONCRETE COMPOSITE MULTI-BOX GIRDER BRIDGES OF MODERATE LENGTH. IT IS APPLICABLE TO BOX GIRDERS OF SINGLE CELL, HAVING WIDTH CENTER TO CENTER OF WEBS APPROXIMATELY EQUAL TO THE DISTANCE CENTER TO CENTER OF ADJACENT WEBS OF ADJACENT BOX GIRDERS. THE CANTILEVER OVERHANG OF THE DECK SLAB (INCLUDING CURBS AND PARAPETS) BEYOND THE EXTERIOR WEB. SHALL BE LIMITED TO 60 PERCENT OF THE DISTANCE BETWEEN THE CENTERS OF ADJACENT WEBS OF ADJACENT BOX GIRDERS BUT IN NO CASE GREATER THAN 6 FEET.

THE PROVISIONS OF DIVISION I, DESIGN, SHALL GOVERN WHERE APPTCABLE, EXCEPT AS SPECIFICALLY MODIFIED BY ARTICLES 1.7.102 THROUGH 1.7.109 .

1.7.103 -LATERAL DISTRIBUTION OF LOADS FOR BENDING MOMENT

THE LIVE LOAD BENDING MOMENT FOR EACH BOX GIRDER SHALL BE DETERMINED BY APPLYING TO THE GIRDER, THE FRACTION W_L OF A WHEEL LOAD (BOTH FRONT AND REAR), DETERMINED BY THE FOLLOWING EQUATION.

$$W_L = 0.1+1.7R+0.85/N_w$$

WHERE $R = N_w /$ NUMBER OF BOX GIRDERS
$N_w = W_c/12$, REDUCED TO THE NEAREST WHOLE NUMBER
$W_c =$ ROADWAY WIDTH BETWEEN CURBS (IN FEET), OR BARRIERS IF CURBS ARE NOT USED. R SHALL NOT BE LESS THAN 0.5 NOR GREATER THAN 1.5.

THE PROVISION OF ARTICLE 1.2.9, REDUCTION IN LOAD INTENSITY, SHALL NOT APPLY IN THE DESIGN OF BOX GIRDERS WHEN USING DESIGN LOAD W_L GIVEN BY THE ABOVE EQUATION.

1.7.104 -DESIGN OF WEB PLATES

A. VERTICAL SHEAR

THE DESIGN SHEAR V_w FOR A WEB SHALL BE CALCULATED USING THE FOLLOWING THE EQUATION:

$$V_w = V_v / \cos \phi$$

WHERE V_v = VERTICAL SHEAR
ϕ = ANGLE OF INCLINATION OF THE WEB PLATE TO THE VERTICAL

B. SECONDARY BENDING STRESSES

WEB PLATES MAY BE NORMAL TO THE BOTTOM OF FLANGE OR INCLINED. IF THE INCLINATION OF THE WEB PLATES TO A PLANE NORMAL TO BOTTOM FLANGE IS NO GREATER THAN 1 TO 4, AND THE WIDTH OF THE BOTTOM FLANGE IS NO GREATER THAN 20 PERCENT OF THE SPAN, THE TRANSVERSE BENDING STRESSES RESULTING FROM DISTORTION OF GIRDER CROSS SECTION AND FROM VIBRATIONS OF THE BOTTOM PLATE, NEED NOT BE CONSIDERED, FOR STRUCTURES IN THIS CATEGORY TRANSVERSE BENDING STRESSES DUE TO SUPPLEMENTARY LOADINGS, SUCH AS UTILITIES, SHALL NOT EXCEED 5,000 PSI.

FOR STRUCTURES EXCEEDING THESE LIMITS OR FOR CURVED GIRDERS, A DETAILED EVALUATION OF THE TRANSVERSE BENDING STRESSES DUE TO ALL CAUSES SHALL BE MADE. THESE STRESSES SHALL BE LIMITED TO A MAXIMUM STRESS OR RANGE OF STRESS OF 20,000 PSI.

1.7.105 -DESIGN OF BOTTOM FLANGE PLATES

A. TENSION FLANGES

IN CASES OF SIMPLY SUPPORTED SPANS, THE BOTTOM FLANGE SHALL BE CONSIDERED COMPLETELY EFFECTIVE IN RESISTING BENDING IF ITS WIDTH DOES NOT EXCEED ONE-FIFTH (1/5) THE SPAN LENGTH. IF THE FLANGE PLATE WIDTH EXCEEDS ONE-FIFTH (1/5) OF THE SPAN, AN AMOUNT EQUAL TO ONE-FIFTH (1/5) OF THE SPAN ONLY SHALL BE CONSIDERED EFFECTIVE

FOR CONTINUOUS SPANS, THE CRITERIA ABOVE SHALL BE APPLIED TO THE LENGTHS BETWEEN-POINTS OF CONTRAFLEXURE.

B. COMPRESSION FLANGES UNSTIFFENED

UNSTIFFENED COMPRESSION FLANGES DESIGNED FOR BASIC ALLOWABLE STRESS OR 0.55 F_y SHALL HAVE A WIDTH TO THICKNESS RATIO EQUAL TO OR LESS THAN

THE VALUE OBTAINED BY THE USE OF THE FORMULA:

$$b/t = 6140/\sqrt{F_y}$$

WHERE b = FLANGE WIDTH BETWEEN WEBS IN INCHES
t = FLANGE THICKNESS IN INCHES

FOR GREATER b/t RATIOS, BUT NOT EXCEEDING 60, THE STRESS IN AN UNSTIFFENED BOTTOM FLANGE SHALL NOT EXCEED THE VALUE DETERMINED BY THE USE OF THE FORMULA:

$$f_b = 0.55F_y - 0.224F_y\left[1 - \left(\text{SIN } 2.92 - \frac{b\sqrt{F_y}}{4560t}\right)\right]$$

FOR VALUES OF b/t EXCEEDING $13{,}300/\sqrt{F_y}$, THE STRESS IN THE FLANGE SHALL NOT EXCEED THE VALUE GIVEN BY THE FORMULA:

$$f_b = 57{,}600{,}000(t/b)^2$$

THE b/t RATIO PREFERABLY SHOULD NOT EXCEED 60 EXCEPT IN AREAS OF LOW STRESS NEAR POINTS OF DEAD LOAD CONTRAFLEXURE.

SHOULD b/t RATIO EXCEED 45, LONGITUDINAL STIFFENERS SHOULD BE CONSIDERED.

C. COMPRESSION FLANGES STIFFENED LONGITUDINALLY %IN SOLVING THESE EQUATIONS A VALUE OF k BETWEEN 2 AND 4 SHOULD BE GENERALLY ASSUMED).

LONGITUDINAL STIFFENERS SHALL BE AT EQUAL SPACINGS ACROSS THE FLANGE WIDTH AND SHALL BE PROPORTIONED SO THAT THE MOMENT OF INERTIA OF EACH STIFFENER ABOUT AN AXIS PARALLEL TO THE FLANGE AND AT THE BASE OF THE STIFFENER IS AT LEAST EQUAL TO:

$$I_s = \phi t^3 w$$

WHERE $\phi = 0.07k^3n^4$ FOR VALUE OF n GREATER THAN 1.
$\phi = 0.125\ k^3$ FOR A VALUE OF n = 1
w = WIDTH OF FLANGE BETWEEN LONGITUDINAL STIFFENER OR DISTANCE FROM A WEB TO THE NEAREST LONGITUDINAL STIFFENER
n = NUMBER OF LONGITUDINAL STIFFENERS
k = BUCKLING COEFFICIENT WHICH SHALL NOT EXCEED 4

FOR A FLANGE, INCLUDING STIFFENERS, TO BE DESIGNED FOR THE BASIC ALLOWABLE STRESS OF 0.55 F_y, THE RATIO w/t SHALL NOT EXCEED THE VALUE GIVEN BY THE FORMULA:

$$w/t = \frac{3070\sqrt{k}}{\sqrt{F_y}}$$

FOR GREATER VALUES OF w/t BUT NOT EXCEEDING $6650\sqrt{k}/\sqrt{F_y}$ OR 60, WHICHEVER IS LESS, THE STRESS IN THE FLANGE, INCLUDING STIFFENERS, SHALL NOT EXCEED THE VALUE DETERMINED BY THE FORMULA:

$$f_b = 0.55F_y - 0.224F_y\left[1 - \sin\left(2.92 - \frac{w\sqrt{F_y}}{2280t\sqrt{k}}\right)\right]$$

1.7.106 - DESIGN OF FLANGE TO WEB WELDS

The total effective thickness of the web-flange welds shall not be less than the thickness of the web. If fillet webs are used, they shall be on both sides of the connecting flange or web plate.

1.7.107 - DIAPHRAGMS

Diaphragms, cross-frames, or other means shall be provided within the box girders at each support to resist transverse rotation, displacement, and distortion.

Intermediate diaphragms or cross-frames are not required for steel box girder bridges designed in accordance with this specification.

1.7.108 - LATERAL BRACING

Generally no lateral bracing system is required between box girders. A horizontal load equal to 25 pounds per square foot acting on the area exposed in elevation shall be applied in the plane of the bottom flange. The section assumed to resist the horizontal load shall consist of the bottom flange acting as a web and 12 times the thickness of the webs acting as flanges. A lateral bracing system shall be provided if the combined stresses due to specified horizontal force and dead load of steel and deck exceed 150 percent of the allowable design stress.

1.7.109 - ACCESS AND DRAINAGE

Consistent with climate, location, and materials, consideration shall be given to the providing of man-holes or other openings, either in the deck slab or in the steel box for form removal, inspection, maintenance, drainage, etc.

HYBRID GIRDERS

1.7.110 - HYBRID GIRDERS, GENERAL

This section pertains to the design of (1) noncomposite girders that have both flanges of the same minimum specified yield strength and a web with a lower minimum specified yield strength, (2) composite girders that have a tension flange with a higher minimum specified yield strength than the web and a compression flange with a minimum specified yield strength not less than that of the web, and (3) girders that utilize an orthotropic deck as the top flange and have a web with a lower minimum specified yield strength than the bottom flange. It is applicable to both simple and continuous span girders. In non-composite girders and in the negative moment portion of continuous span composite girders, the compression flange area shall be equal to the tension flange area or larger than the tension flange area by an amount not exceeding 15 percent. In composite girders, excluding the negative moment portion in continuous span girders, the compression flange area shall be equal to or smaller than the tension flange area, steel girders that support dead weight of the slab without composite action, but act compositely with the slab in supporting the live load, shall be considered to be composite girders. In either composite or noncomposite girders, the minimum specified yield strength of the web shall not be less than 35 percent of the minimum specified yield strength of the tension flange.

In girders that utilize an orthotropic deck as the top flange, the minimum specified yield strength of the web shall not be less than 35 percent of the minimum specified yield strength of the bottom flange in regions of negative bending moment. As used in this section, flange refers to the flange of the steel girders and excludes the slab and reinforcing bars.

The provisions of Division I, Design, shall govern where applicable, except as specifically modified by Articles 1.7.110 through 1.7.113.

1.7.111 - ALLOWABLE STRESSES

A. BENDING

The bending stress in the web may exceed the allowable stress for the web steel provided that the stress in each flange does not exceed the allowable stress from Art. 1.7.1 or 1.7.3 for the steel in that flange multiplied by the reduction factor

$$R = 1 - \frac{\beta\psi(1-\alpha)^2\,(3-\psi+\psi\alpha)}{6+\beta\psi(3-\psi)}$$

(See figures 1.7.111A and 1.7.111B)

WHERE

α = The minimum specified yield strength of the web divided by the minimum specified yield strength of the bottom flange

β = The area of the web divided by the area of the bottom flange.

ψ = The distance from the outer edge of the bottom flange to the neutral axis (of the transformed section for composite girders) divided by depth of the steel section.

The bending stress in the concrete slab in composite girders shall not exceed the allowable stress for the concrete multiplied by R.

B. SHEAR

The shear stress in the web (the shear force divided by the web area) shall not exceed the allowable shear stress for the web steel.

C. FATIGUE

Hybrid Girders shall be designed for fatigue as if they were homogeneous girders of flange steel. The allowable fatigue stresses for web splices and for attachments to the web and flange-web fillet web connections shall be based on the flange steel.

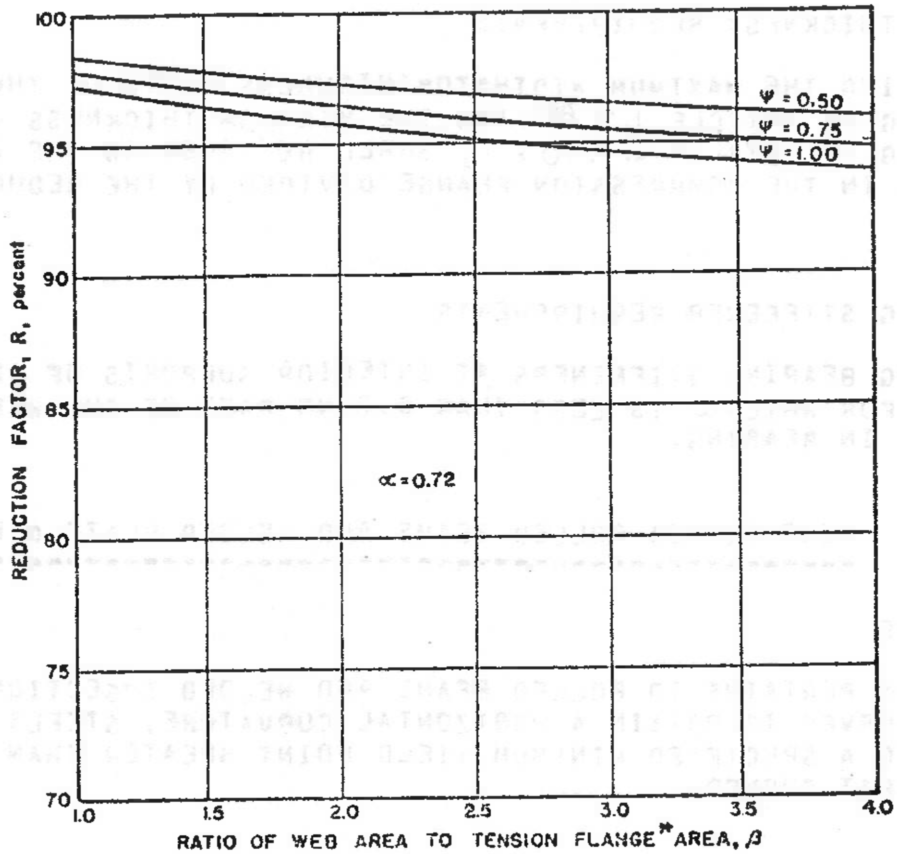


FIGURE I.7.III A

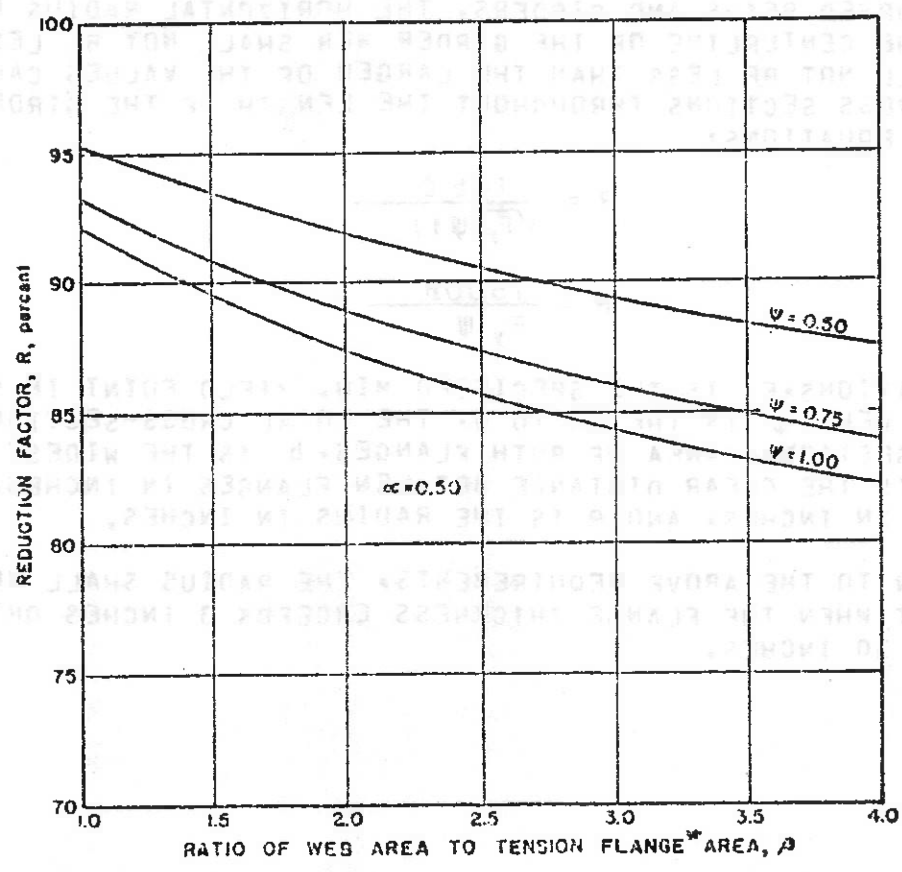


FIGURE I.7.III B

* Bottom flange of orthotropic deck bridges.

1.7.112 - PLATE THICKNESS REQUIREMENTS

IN CALCULATING THE MAXIMUM WIDTH-TO-THICKNESS RATIO OF THE FLANGE PLATE ACCORDING TO ARTICLE 1.7.69 AND THE MINIMUM THICKNESS OF THE WEB PLATE ACCORDING TO ARTICLE 1.7.70, f_b SHALL BE TAKEN AS THE CALCULATED BENDING STRESS IN THE COMPRESSION FLANGE DIVIDED BY THE REDUCTION FACTOR, R.

1.7.113 - BEARING STIFFENER REQUIREMENTS

IN DESIGNING BEARING STIFFENERS AT INTERIOR SUPPORTS OF CONTINUOUS HYBRID GIRDER FOR WHICH α IS LESS THAN 0.7 NO PART OF THE WEB SHALL BE ASSUMED TO ACT IN BEARING.

HEAT-CURVED ROLLED BEAMS AND WELDED PLATE GIRDERS

1.7.114 - SCOPE

THIS SECTION PERTAINS TO ROLLED BEAMS AND WELDED I-SECTION PLATE GIRDERS HEAT-CURVED TO OBTAIN A HORIZONTAL CURVATURE. STEELS THAT ARE MANUFACTURED TO A SPECIFIED MINIMUM YIELD POINT GREATER THAN 50,000 PSI. SHALL NOT BE HEAT CURVED.

1.7.115 - MINIMUM RADIUS OF CURVATURE

FOR HEAT-CURVED BEAMS AND GIRDERS, THE HORIZONTAL RADIUS OF CURVATURE MEASURED TO THE CENTERLINE OF THE GIRDER WEB SHALL NOT BE LESS THAN 150 FEET, AND SHALL NOT BE LESS THAN THE LARGER OF THE VALUES CALCULATED (AT ANY AND ALL CROSS SECTIONS THROUGHOUT THE LENGTH OF THE GIRDER) FROM THE FOLLOWING TWO EQUATIONS:

$$R = \frac{14\,b\,D}{\sqrt{F_y}\,(\psi\,t)}$$

$$R = \frac{7500\,b}{F_y\,\psi}$$

IN THESE EQUATIONS, F_y IS THE SPECIFIED MIN. YIELD POINT IN KSI OF STEEL IN THE GIRDER WEB, ψ IS THE RATIO OF THE TOTAL CROSS-SECTIONAL AREA TO THE CROSS-SECTIONAL AREA OF BOTH FLANGES, b IS THE WIDEST FLANGE WIDTH IN INCHES, D IS THE CLEAR DISTANCE BETWEEN FLANGES IN INCHES, t IS THE WEB THICKNESS IN INCHES, AND R IS THE RADIUS IN INCHES.

IN ADDITION TO THE ABOVE REQUIREMENTS, THE RADIUS SHALL NOT BE LESS THAN 1000 FEET WHEN THE FLANGE THICKNESS EXCEEDS 3 INCHES OR THE FLANGE WIDTH EXCEEDS 30 INCHES.

1.7.116 - CAMBER

TO COMPENSATE FOR POSSIBLE LOSS OF CAMBER OF HEAT-CURVED GIRDERS IN SERVICE AS RESIDUAL STRESSES DISSIPATE, THE AMOUNT OF CAMBER IN INCHES, Δ, AT ANY SECTION ALONG THE LENGTH OF THE GIRDER SHALL BE EQUAL TO:

$$\Delta = \frac{\Delta DL}{\Delta m}\left(\Delta m + \frac{0.02L^2 F_y}{EY_o}\right)$$

WHERE ΔDL IS THE CAMBER IN INCHES AT ANY POINT ALONG THE SPAN CALCULATED BY USUAL PROCEDURES TO COMPENSATE FOR DEFLECTION DUE TO DEAD LOADS OR ANY OTHER SPECIFIED LOADS, Δm IS THE MAXIMUM VALUE OF ΔDL IN INCHES WITHIN THE SPAN, E IS THE MODULUS OF ELASTICITY IN KSI, F_y IS THE SPECIFIED MINIMUM YIELD POINT IN KSI OF THE GIRDER FLANGE, Y_o IS THE DISTANCE FROM THE NEUTRAL AXIS TO THE EXTREME OUTER FIBER IN INCHES (MAXIMUM DISTANCE FOR NONSYMMETRICAL SECTIONS), AND L IS THE SPAN LENGTH OR DISTANCE BETWEEN POINTS OF DEAD-LOAD CONTRAFLEXURE IN INCHES.*

*PART OF THE CAMBER LOSS IS ATTRIBUTABLE TO CONSTRUCTION LOADS AND WILL OCCUR DURING CONSTRUCTION OF THE BRIDGE; TOTAL CAMBER LOSS WILL BE COMPLETE AFTER SEVERAL MONTHS OF IN-SERVICE LOADS. THEREFORE, A PORTION OF THE CAMBER INCREASE (APPROXIMATELY 50 PERCENT) SHOULD BE INCLUDED IN THE BRIDGE PROFILE. CAMBER LOSSES OF THIS NATURE (BUT, GENERALLY, SMALLER IN MAGNITUDE) ARE ALSO KNOWN TO OCCUR IN STRAIGHT BEAMS AND GIRDERS.

LOAD FACTOR DESIGN

1.7.117 — SCOPE

Load Factor design is an alternate method for design of simple and continuous beam and girder structures of moderate length. It is a method of proportioning structural members for multiples of the design loads. To insure serviceability and durability, consideration is given to the control of permanent deformations under overloads, to the fatigue characteristics under service loadings and to the control of live load deflections under service loadings.

1.7.118 — NOTATION

A = area of cross section (in.^{2})
A_f = area of one flange of beam or girder (in.^{2})
A_s = total area of steel section including cover plates (in.^{2})
A_s = gross effective area of column cross section (in.^{2})
A_w = area of web of beam (in.^{2})
b' = width of projecting flange element (in.)
b' = width of outstanding stiffener element (in.)
D = dead load
D = distance center to center of box girder flange plates (in.)
d = depth of member (in.)
d_b = depth of beam
d_c = depth of column
d_o = distance between tranverse stiffeners (in.)
d_w = depth of steel web of a composite section (in.)
E = modulus of elasticity (29,000,000 psi)
F = stress (psi)
F_{cr} = buckling stress (psi)

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* Part of the camber loss is attributable to construction loads and will occur during construction of the bridge: total camber loss will be complete after several months of in-service loads. Therefore, a portion of the camber increase (approximately 50 percent) should be included in the bridge profile. Camber losses of this nature (but, generally, smaller in magnitude) are also known to occur in straight beams and girders.

F_y = specified minimum yield point or yield strength of the type of steel being used (psi)
f'_c = specified 28-day compressive strength of concrete (psi)
I = impact
I = moment of inertia (in.^{4})
L_c = length of a compression member (in.)
L_b = distance between points of bracing of compression flange (in.)
L = live load
M, M_1, M_2 = moment on a cross section (in.-lb)
M_u = maximum moment capacity (in.-lb)
P = axial compression on the member (lb)
P_u = maximum axial compression capacity (lb)
r = radius of gyration (in.)
r_y = radius of gyration with respect to Y-Y axis (in.)
S = section modulus (in.^{3})
t = flange thickness (in.)
t = thickness of thinnest part connected by bolts (in.)
t_w = web thickness (in.)
V = shear force on the cross section (lb)
V_u = maximum shear capacity (lb)
Z = Plastic Section Modulus (in.^{3})
ϕ = reduction factor

1.7.119 — LOADS

Service live loads are vehicles which may operate on a highway legally without special load permit.

For design purposes, the service loads are taken as the dead, live and impact loadings described in Section 1.2 (except Art. 1.2.4).

Overloads are the live loads that can be allowed on a structure on infrequent occasions without causing permanent damage. For design purposes the maximum overload is taken as $\frac{5}{3}$(L+I).

The maximum loads are the loadings specified in Article 1.7.123.

1.7.120 — DESIGN THEORY

The moments, shears and other forces shall be determined by assuming elastic behavior of the structure except as modified in Article 1.7.124(A) (3).

The members shall be proportioned by the methods specified in Articles 1.7.124 through 1.7.135 so that their computed maximum strengths shall be at least equal to the total effects of design loads multiplied by their respective load factors specified in Groups I, II and III of Article 1.7.123.

Service behavior shall be investigated as specified in Articles 1.7.136 through 1.7.138.

1.7.121 — ASSUMPTIONS

(1) Strain in flexural members shall be assumed directly proportional to the distance from the neutral axis.

(2) Stress in steel below the yield strength, F_y, of the grade of steel used shall be taken as 29,000,000 psi times the steel strain. For strain greater than that corresponding to the yield strength, F_y, the stress shall be considered independent of strain and equal to the yield strength, F_y. This assumption shall apply also to the longitudinal reinforcement in the concrete floor slab in the region of negative moment when shear developers are provided to secure composite action in this region.

(3) At maximum strength the compressive stress in the concrete slab of a composite beam shall be assumed independent of strain and equal to $0.85f'_c$.

(4) Tensile strength of concrete shall be neglected in flexural calculations.

1.7.122 — DESIGN STRENGTH FOR STEEL

The design strength for steel shall be the specified minimum yield point or yield strength, F_y, of the steel used as set forth in Article 1.7.1.

1.7.123 — MAXIMUM DESIGN LOADS

The maximum moments, shears or forces to be sustained by a stress-carrying steel member shall be computed from formulas listed below. Members subject to combinations of loads and forces shall be designed for the combined effects.

$$\textit{Group } I = 1.30\left[D + \frac{5}{3}(L+I)\right]$$

For all loadings less than *H*20, provision shall be made for an infrequent heavy load by applying Group *IA* loading, with the live load assumed to occupy a single lane without concurrent loading in any other lane.

$$\textit{Group } IA = 1.30[D + 2.2(L+I)]$$

$$\textit{Group } II = 1.30[D + W + F + SF + B + S + T]$$

When earthquake loading is taken into account, the Group II loading shall be used substituting *EQ* for *W*. When ice pressure is taken into account, the Group II loading shall be used substituting *ICE* for *SF*.

$$\textit{Group } III = 1.30[D + L + I + CF + 0.3W + WL + F + LF]$$

The symbols in the above formulas represent the moments, shears or forces caused by the loads and effects described in Article 1.2.22.

1.7.124 — SYMMETRICAL BEAMS AND GIRDERS

(A) Compact Sections

Symmetrical I-shaped beams with high resistance to local buckling and proper bracing to resist lateral torsional buckling qualify as compact sections. Compact sections are able to form plastic hinges which rotate at near constant moment.

Rolled or fabricated I-shaped beams meeting the requirements of paragraph (1) below shall be considered compact sections and the maximum strength shall be as computed:

$$M_u = F_y Z$$

where F_y is the specified yield point of the steel being used,
Z is the plastic section modulus.*

(1) Beams designed as compact sections shall meet the following requirements: (for certain frequently used steels these requirements are listed in Table 1).

(a) Projecting flange element

$$b'/t \leq \frac{1600}{\sqrt{F_y}}$$

where b' is the width of the projecting flange element,
t is the flange thickness.

(b) Web thickness

$$d/t_w \leq \frac{13{,}300}{\sqrt{F_y}}$$

where d is the depth of the beam,
t_w is the web thickness.

(c) Lateral bracing

$$L_b/r_y \leq \frac{7000}{\sqrt{F_y}} \quad \text{when } M_2 \geq 0.7M_1$$

or

$$L_b/r_y \leq \frac{12{,}000}{\sqrt{F_y}} \quad \text{when } M_2 < 0.7M_1$$

where L_b is the distance between points of bracing of the compression flange,
r_y is the radius of gyration with respect to the Y-Y axis,
M_1 and M_2 are the moments at the two adjacent braced points.
In no case shall L_b exceed the value given in Article 1.7.124 (B) (1) (c).

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* See Commentary of AISI Bulletin 15 for method of computing Z. Values for rolled sections are listed in the "Manual of Steel Construction," Seventh Edition, 1970, American Institute of Steel Construction.

The required lateral bracing shall be provided by braces capable of preventing lateral displacement and twisting of the main members or by embedment of the top and sides of the compression flange in concrete.

(d) Maximum axial compression

$$P \leq 0.15F_yA$$

where A is the area of the cross section.

(e) Maximum shear force

$$V \leq 0.55F_ydt_w$$

(2) Article 1.7.124(A) is applicable to steels with stress-strain diagrams which exhibit a yield plateau followed by a strain hardening range.
Steels such as ASTM A36, A242, A440, A441, A572 and A588 meet these requirements. The limitations set forth in Article 1.7.124(A) are given in Table 1.

TABLE 1

F_y (psi)	36,000	42,000	46,000	50,000	55,000
b'/t	8.4	7.8	7.5	7.2	6.8
d/t	70	65	62	59	57
L_b/r_y $M_2 \geq 0.7M_1$	37	34	33	31	30
L_b/r_y $M_2 < 0.7M_1$	63	59	56	54	51

(3) In the design of a continuous beam of compact section complying with the provisions of Article 1.7.124(A) (1), negative moments over supports determined by elastic analysis may be reduced by a maximum of 10%. Such reduction shall be accompanied by an increase in maximum positive moment in the span equal to the average decrease of the negative moments in the span. The reduction shall not apply to negative moments produced by cantilever loading.

(B) Braced Non-Compact Sections

For rolled or fabricated I-shaped beams not meeting the requirements of Article 1.7.124(A)(1) but meeting the requirements of paragraph (1) below, the maximum strength shall be computed as:

$$M_u = F_yS$$

where S is the section modulus.

(1) The above equation is applicable to beams meeting the following requirements:

(a) Projecting flange element

$$b'/t \leq 2200/\sqrt{F_y}$$

When

$M < M_u$, b'/t may be increased by the ratio $\sqrt{M_u/M}$

(b) Web thickness

$$D/t_w \leq 150$$

where D is the clear unsupported distance between flange components.

(c) Spacing of lateral bracing for compression flange

$$L_b \leq \frac{20{,}000{,}000A_f}{F_y d}$$

where d is the depth of beam or girder,
A_f is the flange area.

(d) Maximum axial compression
Axial compression shall not exceed the value given by Article 1.7.124(A)(1)(d).

(e) Maximum shear force

$$V \leq \frac{3.5Et^3_w}{D}$$

but not more than $0.58F_yDt_w$

(2) The limitations set forth in paragraph (1) above are given in Table 2.

TABLE 2

F_y (psi)	36,000	42,000	46,000	50,000	55,000	90,000	100,000
b'/t	11.6	10.7	10.3	9.8	9.4	7.3	7.0
$\frac{L_b d}{A_f}$	556	476	435	400	364	222	200

(C) Transition

The maximum strength of members with geometric properties falling between the limits of Articles 1.7.124(A) and (B) may be computed by straight line interpolation, except that the web thickness must always satisfy Article 1.7.124(A)(1)(b).

(D) Unbraced Sections

(1) For members not meeting the lateral bracing requirement of Article 1.7.124(B)(1)(c) the maximum strength shall be computed as:

$$M_u = F_y S\left[1 - \frac{3F_y}{4\pi^2 E}\left(\frac{L_b}{b'}\right)^2\right]$$

When the ratio of stresses at the two ends of the braced length, L_b, is less than 0.7, the maximum strength, M_u, as computed by the above formula may be increased 20% but not to exceed F_yS.

(2) In members not meeting the requirements of Article 1.7.124 (B)(1)(e) the web shall be provided with transverse stiffeners as specified in Article 1.7.124(E).

(3) Members with axial loads in excess of $0.15F_yA$ should be designed as beam-columns as specified in Article 1.7.134

(E) Transversely Stiffened Girders

(1) For girders not meeting the shear requirements of Articles 1.7.124(A)(1)(e) and 1.7.124(B)(1)(e) transverse stiffeners are required for the web. For girders with transverse stiffeners but without longitudinal stiffeners the thickness of the web shall meet the requirement:

$$D/t_w \leq \frac{36,500}{\sqrt{F_y}}$$

For different grades of steel this limit is:

D/t_w	F_y (psi)
192	36,000
178	42,000
170	46,000
163	50,000
156	55,000
122	90,000
115	100,000

(2) The maximum bending strength of transversely stiffened girders meeting the requirements of Article 1.7.124(E)(1) shall be computed by Articles 1.7.124(B) or 1.7.124(D)(1) as applicable subject to the requirement of Article 1.7.124(E)(4).

(3) The shear capacity of beams and girders with webs fulfilling the requirements of Article 1.7.124(E)(1) shall be computed as:

$$V_u = V_p\left[C + \frac{0.87(1-C)}{\sqrt{1+(d_o/D)^2}}\right]$$

where:

$$V_p = 0.58F_yDt_w$$

$$C = 18,000(t_w/D)\sqrt{\frac{1+(D/d_o)^2}{F_y}} - 0.3 \leq 1.0$$

D=clear, unsupported distance between flange components.

d_o=distance between transverse stiffeners.

(4) If a girder panel is subjected to simultaneous action of shear and bending moment with the magnitude of the shear higher than $0.6V_u$, then the moment shall be limited to not more than:

$$M/M_u = 1.375 - 0.625\,V/V_u$$

(5) Transverse stiffeners shall be spaced at a distance, d_o, according to shear capacity as specified in Article 1.7.124(E)(3) but not more than 1.5D. Transverse stiffeners may be omitted in those portions of the girders where the maximum shear force is less than the value given by Article 1.7.124(B)(1)(e).

The first stiffener space at the ends of girders with simple supports shall not be greater than D nor:

$$d_o = 14{,}500\sqrt{Dt_w{}^3/V}$$

The width-to-thickness ratio of transverse stiffeners shall be such that

$$b'/t \leq \frac{2{,}600}{\sqrt{F_y}}$$

where b′ is the projecting width of the stiffener.

The gross cross-sectional area of intermediate transverse stiffeners shall not be less than:

$$A = [0.15\ BDt_w(1-C)(V/V_u) - 18t_w{}^2]Y$$

where Y is the ratio of web plate yield strength to stiffener plate yield strength

B = 1.0 for stiffener pairs,
1.8 for single angles,
2.4 for single plates.

C is computed by Article 1.7.124 (E) (3)

The moment of inertia of transverse stiffeners with reference to the mid-plane of the web shall be not less than:

$$I = d_o\ t_w{}^3\ J$$

where:

$$J = 2.5(D/d_o)^2 - 2, \text{ but not less than } 0.5.$$

Transverse stiffeners need not be in bearing with the tension flange. The maximum distance between the stiffener-to-web connection and the face of the tension flange shall not be more than $4t_w$. Stiffeners provided on only one side of the web must be in bearing against but need not be attached to the compression flange.

(F) Longitudinally Stiffened Girders

(1) Longitudinal stiffeners shall be required when the web thickness is less than that specified by Article 1.7.124(E)(1) and shall be placed at a distance D/5 from the inner surface of the compression flange.

The web thickness of plate girders with transverse stiffeners and one longitudinal stiffener shall meet the requirement:

$$D/t_w \leq \frac{73{,}000}{\sqrt{F_y}}$$

For different grades of steel, this limit is:

D/t_w	F_y (psi)
385	36,000
356	42,000
340	46,000
326	50,000
311	55,000
243	90,000
231	100,000

(2) The maximum bending strength of longitudinally stiffened girders meeting the requirements of Article 1.7.124(F)(1) shall be computed by Articles 1.7.124(B) or Article 1.7.124(D)(1) as applicable, subject to the requirement of Article 1.7.124(E)(4).

(3) The shear capacity of girders with one longitudinal stiffener shall be computed by Article 1.7.124(E)(3).

The dimensions of the longitudinal stiffener shall be such that:

(a) the width-to-thickness ratio is not greater than that given by Article 1.7.124(E)(5).

(b) the rigidity of the stiffener is not less than:

$$I \geq Dt_w^3 \left[2.4\left(\frac{d_o}{D}\right)^2 - 0.13 \right]$$

(c) the radius of gyration of the stiffener is not less than:

$$r \geq \frac{d_o\sqrt{F_y}}{23{,}000}$$

In computing I and r values above, a centrally located web strip not more than $18t_w$ in width shall be considered as a part of the longitudinal stiffener. Transverse stiffeners for girder panels with longitudinal stiffeners shall be designed according to Article 1.7.124(E)(5) except that the depth of subpanels shall be used instead of the total panel depth, D. In addition the section modulus of the transverse stiffener shall be not less than:

$$S_t = \frac{1}{3}(D/d_o)S_l$$

where D is the total panel depth (clear distance between flange components) and S_l is the section modulus of the longitudinal stiffener at D/5.

1.7.125 — UNSYMMETRICAL BEAMS AND GIRDERS

(A) General

For beams and girders symmetrical about the vertical axis of the cross section but unsymmetrical with respect to the horizontal centroidal axis, the provisions of Articles 1.7.124(A) through 1.7.124(D) shall be applicable except that in computing the maximum strength by Article 1.7.124(D)(1) the term b′ is replaced by 0.9b′.

(B) Unsymmetrical Sections with Transverse Stiffeners

Girders with transverse stiffeners shall be designed and evaluated by the provisions of Article 1.7.124(E) except that when D_c, the clear distance between the neutral axis and the compression flange, exceeds D/2 the web thickness, t_w, shall meet the requirement:

$$\frac{D_c}{t_w} \leq \frac{18{,}250}{\sqrt{F_y}}$$

(C) Longitudinally Stiffened Unsymmetrical Sections

Longitudinal stiffeners shall be required on unsymmetrical sections when the web thickness is less than that specified by Articles 1.7.124(E)(1) or 1.7.125(B).

For girders with one longitudinal stiffener and transverse stiffeners, the provisions of Article 1.7.124(F) for symmetrical sections shall be applicable provided that:

(a) the longitudinal stiffener is placed $2D_c/5$ from the inner surface or the leg of the compression flange element.

(b) When D_c exceeds D/2, the web thickness, t_w, shall meet the requirement:

$$\frac{D_c}{t_w} \leq \frac{36{,}500}{\sqrt{F_y}}$$

1.7.126 — COMPOSITE BEAMS AND GIRDERS

Composite beams shall be so proportioned that the following criteria are satisfied:

(a) The maximum strength of any section shall not be less than the sum of the computed moments at that section multiplied by the appropriate load factors.

(b) The web of the steel section shall be designed to carry the total external shear and must satisfy the applicable provisions of Articles 1.7.124 and 1.7.125. In such application the value of D_c shall be taken as the clear distance between the neutral axis of the composite section for live loads and the compression flange.

1.7.127 — POSITIVE MOMENT SECTIONS OF COMPOSITE BEAMS AND GIRDERS

(A) Compact Sections

When the steel section satisfies the compactness requirements of Article 1.7.127(A)(2), the maximum strength shall be computed as the resultant moment of the fully plastic stress distribution acting on the section (Figure 1.7.127).

(1) The resultant moment of the fully plastic stress distribution may be computed as follows:

(a) the compressive force in the slab, C, is equal to the smallest of the values given by the following Equations:

(1) $C = 0.85\, f'_c b t_s + (A\, F_y)_c$

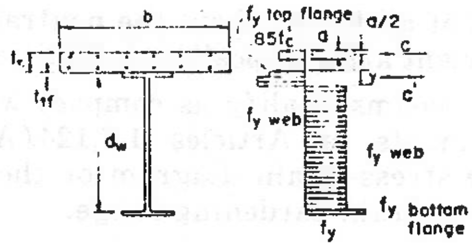

Fig. I.7.127

where b is the effective width of slab,

t_s is the slab thickness.

$(A\,F_y)_c$ is the product of the area and yield point of that part of reinforcement which lies in the compression zone of the slab.

$$(2)\quad C=(A\,F_y)_{bf}+(A\,F_y)_{tf}+(A\,F_y)_w$$

where $(A\,F_y)_{bf}$ is the product of area and yield point for bottom flange of steel section (including cover plate if any),

$(A\,F_y)_{tf}$ is the product of area and yield point for top flange of steel section,

$(A\,F_y)_w$ is the product of area and yield point for web of steel section.

$$(3)\quad C=\Sigma Q_u$$

where ΣQ_u is sum of ultimate strengths of shear connectors between the section under consideration and the section of zero moment.

(b) the depth of the stress block is computed from the compressive force in the slab.

$$a=\frac{C-(A\,F_y)_c}{0.85f'_cb}$$

(c) when the compressive force in the slab is less than the value given by Equation (2) above the top portion of the steel section will be subjected to the following compressive force:

$$C'=\frac{\Sigma(A\,F_y)-C}{2}$$

(d) The location of the neutral axis within the steel section measured from the top of the steel section may be determined as follows:

for $C'<(A\,F_y)_{tf}$

$$\bar{y}=\frac{C'}{(A\,F_y)_{tf}}t_{tf}$$

for $C'\geq(A\,F_y)_{tf}$

$$\bar{y}=t_{tf}+\frac{C'-(A\,F_y)_{tf}}{(A\,F_y)_w}d_w$$

(e) the maximum strength of the section in bending is the first moment of all forces about the neutral axis, taking all forces and moment arms as positive quantities.

(2) Composite beams qualify as compact when their steel section meets the requirements of Articles 1.7.124(A)(1)(b) and 1.7.124 (A(1)(e), and the stress-strain diagram of the steel exhibits a yield plateau followed by a strain hardening range.

(B) Non-compact Sections

When the steel section does not satisfy the compactness requirements of Article 1.7.127(A)(2) the maximum strength of the section shall be taken as the moment at first yielding.

(C) General

Maximum compressive and tensile stresses in girders which are not provided with temporary supports during the placing of dead loads shall be the sum of the stresses produced by $1.30D_s$ acting on the steel girder alone and the stresses produced by $1.30[D_c+5/3(L+I)]$ acting on the composite girder, where D_s and D_c are the moment caused by the dead load acting on the steel girder and composite girder, respectively.

When the girders are provided with effective intermediate supports which are kept in place until the concrete has attained 75% of its required 28-day strength, stresses are produced by the loading, $1.30[D+5/3(L+I)]$, acting on the composite girder.

1.7.128 — NEGATIVE MOMENT SECTIONS OF COMPOSITE BEAMS AND GIRDERS

The maximum strength of beams and girders in the negative moment regions shall be computed in accordance with Articles 1.7.124 and 1.7.125 as applicable. It shall be assumed that the concrete slab does not carry tensile stresses. In cases where the slab reinforcement is continuous over interior supports, the reinforcement may be considered to act compositely with the steel section.

1.7.129 — COMPOSITE BOX GIRDERS

This section pertains to the design of simple and continuous bridges of moderate length supported by two or more single-cell composite box girders. It is applicable to box girders, having width center-to-center of top steel flanges approximately equal to the distance center-to-center of adjacent top steel flanges of adjacent box girders. The cantilever overhang of the deck slab, including curbs and parapet, shall be limited to 60 percent of the distance between the centers of adjacent top steel flanges of adjacent box girders, but in no case greater than 6 feet.

(A) Maximum Strength

The maximum strength of box girders shall be determined according to the applicable provisions of Article 1.7.126, 1.7.127 and 1.7.128. In addition, the maximum strength of the negative moment sections shall be limited by

$$M_u = F_{cr}S$$

where F_{cr} is the buckling stress of the bottom flange plate as given in Article 1.7.129(E).

(B) Lateral Distribution

The live load bending moment for each box girder shall be determined in accordance with Article 1.7.103.

(C) Web Plates

The design shear V_w for a web shall be calculated using the following equation:

$$V_w = V/\cos\theta$$

where V = one half of the total vertical shear force on one box girder,
θ = the angle of inclination of the web plate to the vertical.
The inclination of the web plates to the vertical shall not exceed 1 to 4.

(D) Tension Flanges

In the case of simply supported spans, the bottom flange shall be considered fully effective in resisting bending if its width does not exceed one-fifth the span length. If the flange plate width exceeds one-fifth of the span, only an amount equal to one-fifth of the span shall be considered effective.

For continuous spans, the requirements above shall be applied to the distance between points of contraflexure.

(E) Compression Flanges

(1) Unstiffened compression flanges designed for the yield stress, F_y, shall have a width-to-thickness ratio equal to or less than the value obtained from the formula:

$$b/t = \frac{6140}{\sqrt{F_y}}$$

where b = flange width between webs in inches,
t = flange thickness in inches.

(2) For greater b/t ratios, but not exceeding $13{,}300/\sqrt{F_y}$, the buckling stress of an unstiffened bottom flange is given by the formula:

$$F_{cr} = 0.592\, F_y \left(1 + 0.687 \sin\frac{c\pi}{2}\right)$$

in which c shall be taken as

$$c = \frac{13{,}300 - \frac{b}{t}\sqrt{F_y}}{7160}$$

(3) For values of b/t exceeding $13{,}300\sqrt{F_y}$, the buckling stress of the flange is given by the formula:

$$F_{cr} = 105\ (t/b)^2 \times 10^6$$

(4) If longitudinal stiffeners are used, they shall be equally spaced across the flange width and shall be proportioned so that the moment of inertia of each stiffener about an axis parallel to the flange and at the base of the stiffener is at least equal to:

$$I_s = \phi t^3 w$$

where $\phi = 0.07k^3n^4$ when n equals 2, 3, 4 or 5.

$\phi = 0.125k^3$ when $n=1$.
w = width of flange between longitudinal stiffeners or distance from a web to the nearest longitudinal stiffener.
n = number of longitudinal stiffeners.
k = buckling coefficient which shall not exceed 4.

For a longitudinally stiffened flange designed for the yield stress F_y, the ratio w/t shall not exceed the value given by the formula

$$w/t = \frac{3070\sqrt{k}}{\sqrt{F_y}}$$

For greater values of w/t, but not exceeding $6650\sqrt{k}/\sqrt{F_y}$, the buckling stress of the flange, including stiffeners is given by Article 1.7.129 (E) (2) in which c shall be taken as:

$$c = \frac{6650\sqrt{k} - (w\sqrt{F_y}/t)}{3580\sqrt{k}}$$

For values of w/t exceeding $6650\sqrt{k}/\sqrt{F_y}$ the buckling stress of the flange, including stiffeners, is given by the formula:

$$F_{cr} = 26.2k(t/w)^2 \times 10^6$$

When longitudinal stiffeners are used, it is preferable to have at least one transverse stiffener placed near the point of dead load contraflexure. The stiffener should have a size equal to that of a longitudinal stiffener.

(5) The width-to-thickness ratio of any outstanding element of the flange stiffeners shall not exceed the value determined by the formula:

$$b'/t' = \frac{2600}{\sqrt{F_y}}$$

where b' = width of any outstanding stiffener element,
t' = thickness of outstanding stiffener element.

(F) Diaphragms

Diaphragms, cross-frames, or other means shall be provided within the box girders at each support to resist transverse rotation, displacement and distortion.

Intermediate diaphragms or cross-frames are not required for box girder bridges designed in accordance with this specification.

1.7.130 — SHEAR CONNECTORS

(A) General

The horizontal shear at the interface between the concrete slab and the steel girder shall be provided for by mechanical shear connectors throughout the simple spans and the positive moment regions of continuous spans. In the negative moment regions, shear connectors shall be provided when the reinforcement steel imbedded in the concrete is considered a part of the composite section. In case the reinforcement steel imbedded in the concrete is not considered in computing section properties of negative moment sections, shear connectors need not be provided in these portions of the span, but additional connectors shall be placed in the region of the points of dead load contraflexure as specified in Article 1.7.100(A)(3).

(B) Design of Connectors

The number of shear connectors shall be determined in accordance with Article 1.7.100(A)(2), and checked for fatigue in accordance with Article 1.7.100(A)(1) and 1.7.100(A)(3).

(C) Maximum Spacing

The maximum pitch shall not exceed 24 inches except over the interior supports of continuous beams where wider spacing may be used to avoid placing connectors at locations of high stresses in the tension flange.

1.7.131 — HYBRID GIRDERS

This section pertains to the design of (1) noncomposite beams and girders that have flanges of the same minimum specified yield strength and a web with a lower minimum specified yield strength, and (2) composite girders that have a tension flange with a higher minimum specified yield strength than the web and a compression flange with a minimum specified yield strength not less than that of the web. It is applicable to both simple and continuous girders. In noncomposite girders and in the negative moment portion of continuous composite girders, the area of the compression flange shall be equal to the area of the tension flange, or larger than the area of the tension flange by an amount not exceeding 25 percent. In composite girders, excluding the negative moment portion in continuous girders, the area of the compression flange shall be equal to or smaller than the area of the tension flange. The minimum specified yield strength of the web shall not be less than 35 percent of the minimum specified yield strength of the tension flange.

The provisions of Articles 1.7.124 through 1.7.130 shall apply to hybrid beams and girders except as modified below. In all equations of these Articles, F_y shall be taken as the minimum specified yield strength of the steel of the element under consideration.

1.7.132 — NONCOMPOSITE HYBRID GIRDERS

(A) Compact Sections

The equation of Article 1.7.124(A) for the maximum strength of compact sections shall be replaced by the expression

$$M_u = F_{yf}Z$$

where F_{yf} is the specified minimum yield strength of the flange and Z is the plastic section modulus.

In computing Z, the web thickness shall be multiplied by the ratio of the minimum specified yield strength of the web, F_{yw}, to the minimum specified yield strength F_{yf}.

(B) Braced Non-compact Sections

The equation of Article 1.7.124(B) for the maximum strength of compact sections shall be replaced by the expression

$$M_u = F_{yf}SR$$

For symmetrical sections,

$$R = \frac{12 + \beta(3\rho - \rho^3)}{12 + 2\beta}$$

where

$$\rho = F_{yw}/F_{yf}$$
$$\beta = A_w/A_f$$

For unsymmetrical sections,

$$R = 1 - \frac{\beta\psi(1-\rho)^2\,(3 - \psi + \rho\psi)}{6 + \beta\psi(3-\psi)}$$

where ψ is the distance from the outer fiber of the tension flange to the neutral axis divided by the depth of the steel section.

(C) Unbraced Noncompact Sections

The equation of Article 1.7.124(D)(1) for the maximum strength of unbraced noncompact sections shall be replaced by the expression

$$M_u = F_{yf}S\left[1 - \frac{3F_{yf}}{4\pi^2E}\left(\frac{L_b}{b'}\right)^2\right]R$$

where the appropriate R is determined from (B) above.

(D) Transversely Stiffened Girders

The equation of Article 1.7.124(E)(3) for the shear capacity of transversely stiffened girders shall be replaced by the expression

$$V_u = V_pC$$

The equation for A in Article 1.7.124(E)(5) is not applicable to hybrid girders.

1.7.133 — COMPOSITE HYBRID GIRDERS

The maximum strength of the composite section shall be the moment at first yielding of the flanges times R (for unsymmetrical sections) from Article 1.7.132(B), in which ψ is the distance from the outer fiber of the tension flange to the neutral axis of the transformed section divided by the depth of the steel section.

1.7.134 — COMPRESSION MEMBERS

(A) Axial Loading

(1) Maximum Capacity

The maximum strength of concentrically loaded columns shall be computed as:

$$P_u = 0.85\, A_s F_{cr}$$

where A_s is the gross effective area of the column cross section and F_{cr} is determined by one of the following two formulas:

$$F_{cr} = F_y\left[1 - \frac{F_y}{4\pi^2 E}\left(\frac{KL_c}{r}\right)^2\right]$$

for $\frac{KL_c}{r}$ less than or equal to $\sqrt{\frac{2\pi^2 E}{F_y}}$

$$F_{cr} = \frac{\pi^2 E}{\left(\frac{KL_c}{r}\right)^2}$$

for $\frac{KL_c}{r}$ more than $\sqrt{\frac{2\pi^2 E}{F_y}}$

where

- K is effective length factor in the plane of buckling
- L_c is length of the member between points of support, in inches
- r is radius of gyration in the plane of buckling, in inches
- F_y is yield stress of the steel, in psi
- E is 29,000,000 psi
- F_{cr} is buckling stress, in psi

(2) Effective Length

The effective length factor K shall be determined as follows:

(a) For members having lateral support in both directions at its ends:
$K=0.75$ for riveted, bolted or welded end connections.
$K=0.875$ for pinned ends.

(b) For members having ends not fully supported laterally by diagonal bracing or an attachment to an adjacent structure, the effective length factor shall be determined by a rational procedure.*

(B) Combined Axial Load and Bending

(1) Maximum Capacity

The combined maximum axial force P and the maximum bending moment M acting on a beam-column subjected to eccentric loading shall satisfy the following equations:

$$\frac{P}{0.85A_sF_{cr}} + \frac{MC}{M_u\left(1 - \frac{P}{A_sF_e}\right)} \leq 1.0$$

$$\frac{P}{0.85A_sF_y} + \frac{M}{M_p} \leq 1.0$$

where

F_{cr} is buckling stress as determined by the equations of Article 1.7.134 (A) (1)

M_u is the maximum strength as determined by Articles 1.7.124 (A) (B) or (D)

$$F_e = \frac{\pi^2 E}{\left(\frac{KL_c}{r}\right)^2}$$ = the Euler buckling stress in the plane of bending,

C is the equivalent moment factor, as defined below.

$M_p = F_yZ$ the full plastic moment of the section,

Z is the plastic section modulus,

$\frac{KL_c}{r}$ is the effective slenderness ratio in the plane of bending.

(2) Equivalent Moment Factor C

If the ends of the beam-column are restrained from sidesway in the plane of bending by diagonal bracing or attachment to an adjacent laterally braced structure, then the value of equivalent moment factor, C, may be computed by the formula:

$$C = 0.6 + 0.4a, \text{ but not less than } 0.4$$

where a is the ratio of the numerically smaller to the larger end moment. The ratio a is positive when the two end moments act in an opposing sense (i.e., one acts clockwise and the other acts counterclockwise) and negative when they act in the same sense. In all cases, factor C may be taken conservatively as unity.

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* B. G. Johnston, "Guide to Design Criteria for Metal Compression Members," John Wiley and Sons, Inc., New York, 1966.

1.7.135 — SPLICES, CONNECTIONS & DETAILS

(A) Connectors

(1) General

Connectors shall be proportioned so that their maximum strength multiplied by the reduction factor, ϕ, shall be at least equal to the effects of design loads multiplied by their respective load factors specified in Article 1.7.123. The maximum strengths multiplied by the reduction factors are listed in Table 3.

(2) Welds

The ultimate strength of weld metal in groove welds shall be equal to or greater than that of the base metal. The ultimate strength of the weld metal in fillet welds need not match the strength of the base metal. However, the welding procedure and weld metal shall be selected to insure sound welds. The effective weld area shall be taken as defined in Article 1.7.28.

(3) Bolts and Rivets

In proportioning fasteners, the nominal diameter shall be used except when a shear plane intersects the threads.

High-strength bolts preferably shall be used for fasteners subject to tension or combined shear and tension.

TABLE 3

Type of Fastener	Strength (ϕF)
Groove Weld [1]	1.00 F_y
Fillet Weld [2]	0.45 f_u
Low-Carbon Steel Bolts	
ASTM A307	
Tension	27 ksi
Shear [3]	25 ksi
Power-Driven Rivets	
ASTM A502	
Shear — Grade 1	25 ksi
Shear — Grade 2	30 ksi
High-Strength Bolts	
ASTM A325	
Tension [5]	76 ksi
Shear (Bearing-Type) [3 4 5]	54 ksi

(1)—F_y = yield point of connected material.

(2)—F_u = minimum strength of the welding rod metal but not greater than the tensile strength of the connected parts.

(3)—When a shear plane intersects the bolt threads, the root area shall be used.

(4)—Bearing stresses in bearing-type connections shall not exceed the tensile strength of the connected material.

(5)—For A235 bolts the tensile strength decreases for diameters greater than ⅞ in. The design value listed is for bolts up to ⅞ in. diameter. For diameters greater than ⅞ in. diameter the design value shall be computed as 0.56 F_u for tension and 0.45 F_u for shear where F_u is the ASTM minimum tensile strength of the bolt.

For combined tension and shear in bearing type connections, bolts and rivets shall be proportioned so that the shear stress does not exceed:

$$F_{vc} \leq \sqrt{F^2 - (0.6f_t)^2}$$

where F_v = shear strength of the fastener, ϕF, as given in Table 3.
f_t = tensile stress due to the applied load.

(4) Friction Joints

Friction joints shall be designed to prevent slip at the overload in accordance with Article 1.7.136(C). Maximum strength of the bolts need not be considered in the design of such joints.

(B) Connections

(1) Splices

Splices may be made with rivets, with high-strength bolts or by the use of welding. Splices, whether in tension, compression, bending or shear, shall be designed for not less than the average of the calculated stress resultant at the point of the splice and the strength of the member at the same point, but in any event not less than 75% of the maximum strength of the member. Where a section changes at a splice, the maximum strength of the splice shall be at least 75% of the smaller section spliced.

The maximum strength of the member shall be determined by the gross section for compression members. For members primarily in bending, the gross section shall be used, except that if more than 15% of each flange area is removed, that amount removed in excess of 15% shall be deducted. For tension members and splice material, the gross section shall be used unless the net section area is less than 85% of the corresponding gross area, in which case that amount removed in excess of 15% shall be deducted.

(2) Bolts Subjected to Prying Action by Connected Parts

Bolts required to support applied load by means of direct tension shall be proportioned for the sum of the external load and tension resulting from prying action produced by deformation of the connected parts. The total tension should not exceed the values given in Table 3 of Article 1.7.135.

The tension due to prying actions shall be computed as:

$$Q = \left[\frac{3b}{8a} - \frac{t^3}{20}\right] T$$

where

Q = the prying force per bolt (taken as zero when negative),
T = the direct tension per bolt due to external load,
a = distance from center of bolt to edge of plate,

b = distance from center of bolt to toe of fillet of connected part,
t = thickness of thinnest part connected, in.

(3) Rigid Connections

All rigid frame connections, the rigidity of which is essential to the continuity assumed as the basis of design, shall be capable of resisting the moments, shears, and axial loads to which they are subjected by maximum loads.

The beam web shall equal or exceed the thickness given by:

$$t_w \geq \sqrt{3}\left(\frac{M_c}{F_y d_b d_c}\right)$$

where

M_c is the column moment,
d_b the beam depth,
d_c the column depth.

When the thickness of the connection web is less than that given by the above formula, the web shall be strengthened by diagonal stiffeners or by a reinforcing plate in contact with the web over the connection area.

At joints where the flanges of one member are rigidly framed into one flange of another member, the thickness of the web (t_w) supporting the latter flange and the thickness of the latter flange (t_c) shall be checked by the formulas below. Stiffeners are required on the web of the second member opposite the compression flange of the first member when

$$t_w < \frac{A_f}{t_b + 5k}$$

and opposite the tension flange of the first member when

$$t_c < 0.4\sqrt{A_f}$$

where

t_w = thickness of web to be stiffened.
k = distance from outer face of flange to toe of web fillet of member to be stiffened,
t_b = thickness of flange delivering concentrated force,
t_c = thickness of flange of member to be stiffened,
A_f = area of flange delivering concentrated load.

1.7.136 — OVERLOAD

(A) Noncomposite Beams

For noncomposite beams the moment caused by $D + \frac{5}{3}(L + I)$ shall not exceed 0.8 $F_y S$. For such beams designed for Group IA loading, the moment caused by $D + 2.2(L + I)$ shall not exceed 0.8 $F_y S$. In the case of moment redistribution under the provisions of Article 1.7.124(A)(3), the above limitation shall apply to the modified moments but not to the original moments.

(B) Composite Beams

For composite beams the moment caused by $D+\frac{5}{3}(L+I)$ shall not exceed 95% of the moment at first yielding in the section. For such beams designed for Group IA loading, the moment caused by $D+2.2(L+I)$ shall not exceed 95% of the moment at first yielding in the section. In computing dead load stresses the presence or absence of temporary supports during the construction shall be considered.

(C) Friction Joints

The shear caused by the loading, $D+\frac{5}{3}(L+I)$ in friction-type high-strength bolted joints shall not exceed 21,000 psi for ASTM 325 bolts.

For combined shear and tension in friction-type joints where applied forces reduce the total clamping force on the friction plane, the maximum shear stress shall not exceed the values obtained from the following equations:

For A325 bolts

$$f_v = 21{,}000\ [1 - f_t/0.53F_u]$$

where F_u is the tensile strength of the bolt,

f_t is the applied tensile stress.

1.7.137 — FATIGUE

(A) General

The analysis of the probability of fatigue of steel members or connections under working loads and the allowable fatigue stresses, F_r, shall conform to Article 1.7.3, except that the limitation imposed by the basic design criteria given in Articles 1.7.1 and 1.7.2, shall not apply.

(B) Composite Construction

(1) Slab Reinforcement

When composite action is provided in the negative moment region, the range of stress in slab reinforcement shall be limited to 20,000 psi.

(2) Shear Connectors

The shear connectors shall be designed for fatigue in accordance with Article 1.7.100(A).

(C) Hybrid Beams and Girders

Hybrid girders shall be designed for fatigue in accordance with Article 1.7.111(C).

1.7.138 — DEFLECTION

The control of deflection of steel or of composite steel and concrete structures shall conform to the provision of Article 1.7.12.

ORTHOTROPIC-DECK BRIDGES

1.7.139 — ORTHOTROPIC-DECK BRIDGES, GENERAL

This section pertains to the design of steel bridges that utilize a stiffened steel plate as a deck. Usually the deck plate is stiffened by longitudinal ribs and transverse beams; effective widths of deck plate act as the top flanges of these ribs and beams. Usually the deck, including longitudinal ribs, acts as the top flange of the main box or plate girders. As used in Articles 1.7.139 through 1.7.148, the terms, rib and beam, refer to sections that include an effective width of deck plate.

The provisions of Division I, Design, shall govern where applicable, except as specifically modified by Articles 1.7.139 through 1.7.148.

An appropriate method of elastic analysis, such as the equivalent-orthotropic-slab method or the equivalent-grid method, shall be used in designing the deck. The equivalent stiffness properties shall be selected to correctly simulate the actual deck. An appropriate method of elastic analysis, such as the thin-walled-beam method, that accounts for the effects of torsional distortions of the cross-sectional shape shall be used in designing the girders of orthotropic-deck box-girder bridges. The box-girder design shall be checked for lane or truck loading arrangements that produce maximum distortional (torsional) effects.

1.7.140 — WHEEL-LOAD CONTACT AREA

The wheel loads specified in Article 1.2.5 shall be uniformly distributed to the deck plate over the rectangular area defined below:

Wheel Load, kip	Width Perpendicular to Traffic, inch	Length in Direction of Traffic, inch
8	20+2t	8+2t
12	20+2t	8+2t
16	24+2t	8+2t

In the above table, t is the thickness of the wearing surface in inches.

1.7.141 — EFFECTIVE WIDTH OF DECK PLATE

(A) Ribs and Beams

The effective width of deck plate acting as the top flange of a longitudinal rib or a transverse beam may be calculated by accepted approximate methods.*

(B) Girders

The full width of deck plate may be considered effective in acting as the top flange of the girders if the effective span of the girders is not less than: (1) 5 times the maximum distance between girder webs and (2) 10 times the maximum distance from edge of the deck to the nearest girder web. The effective span shall be taken as the actual span for simple spans and the distance between points of contraflexure for continuous spans. Alternatively, the effective width may be determined by accepted analytical methods.

The effective width of the bottom flange of a box girder shall be determined according to the provisions of Article 1.7.105(A).

1.7.142 — ALLOWABLE STRESSES

(A) Local Bending Stresses in Deck Plate

The term local bending stresses refers to the stresses caused in the deck plate as it carries a wheel load to the ribs and beams. The local transverse bending stresses caused in the deck plate by the specified wheel load plus 30 percent impact shall not exceed 30,000 psi unless a higher allowable stress is justified by a detailed fatigue analysis or by applicable fatigue-test results. For deck configurations in which the spacing of transverse beams is at least 3 times the spacing of longitudinal-rib webs, the local longitudinal and transverse bending stresses in the deck plate need not be combined with the other bending stresses covered in paragraphs (B) and (C) below.

(B) Bending Stresses in Longitudinal Ribs

The total bending stresses in longitudinal ribs due to a combination of (1) bending of the rib and, (2) bending of the girders may exceed the allowable bending stresses in Articles 1.7.1 and 1.7.3 by 25 percent. The bending stress due to each of the two individual modes shall not exceed the allowable bending stresses in Articles 1.7.1 and 1.7.3.

(C) Bending Stresses in Transverse Beams

The bending stresses in transverse beams shall not exceed the allowable bending stresses in Articles 1.7.1 and 1.7.3.

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* Design Manual for "Orthotropic Steel Plate Deck Bridges," AISC, 1963 or "Orthotropic Bridges, Theory and Design," by M. S. Troitsky, Lincoln Arc Welding Foundation, 1967.

(D) Intersections of Ribs, Beams, and Girders

Connections between ribs and the webs of beams, holes in the webs of beams to permit passage of ribs, connections of beams to the webs of girders, and rib splices may affect the fatigue life of the bridge when they occur in regions of tensile stress. Where applicable, the number of cycles of maximum stress and the allowable fatigue stresses given in Section 1.7.3 shall be applied in designing these details; elsewhere, a rational fatigue analysis shall be made in designing the details. Connections between webs of longitudinal ribs and the deck plate shall be designed to sustain the transverse bending fatigue stresses caused in the webs by wheel loads.

1.7.143 — THICKNESS OF PLATE ELEMENTS

(A) Longitudinal Ribs and Deck Plate

Plate elements comprising longitudinal ribs, and deck-plate elements between webs of these ribs, shall meet the minimum thickness requirements of Article 1.7.88; f_a may be taken as 75 percent of the sum of the compressive stresses due to (1) bending of the rib and, (2) bending of the girder, but not less than the compressive stress due to either of these two individual bending modes.

(B) Girders and Transverse Beams

Plate elements of box girders, plate girders, and transverse beams shall meet the requirements of Articles 1.7.69, 1.7.70, 1.7.71, 1.7.72, 1.7.73, and 1.7.105.

1.7.144 — MAXIMUM SLENDERNESS OF LONGITUDINAL RIBS

The slenderness, L/r, of a longitudinal rib shall not exceed the value given by the following formula unless it can be shown by a detailed analysis that overall buckling of the deck will not occur as a result of compressive stress induced by bending of the girders:

$$\left(\frac{L}{r}\right)_{max} = 1000\sqrt{\frac{1500}{F_y} - \frac{2700F}{F_y^2}}$$

where

L = distance between transverse beams

r = radius of gyration about the horizontal centroidal axis of the rib including an effective width of deck plate

F = maximum compressive stress (in psi) in the deck plate as a result of the deck acting as the top flange of the girders; this stress shall be taken as positive

F_y = yield strength of rib material in psi

1.7.145 — DIAPHRAGMS

Diaphragms, cross frames, or other means shall be provided at each support to transmit lateral forces to the bearings and to resist transverse rotation, displacement, and distortion. Intermediate diaphragms or cross frames shall be provided at locations consistent with the analysis of the girders. The stiffness and strength of the intermediate and support diaphragms or cross frames shall be consistent with the analysis of the girders.

1.7.146 — STIFFNESS REQUIREMENTS

(A) Deflections

The deflections of ribs, beams, and girders due to live load plus impact may exceed the limitations in Article 1.7.12, but preferably shall not exceed 1/500 of their span. The calculation of the deflections shall be consistent with the analysis used to calculate the stresses.

To prevent excessive deterioration of the wearing surface, the deflection of the deck plate due to the specified wheel load plus 30 percent impact preferably shall be less than 1/300 of the distance between webs of ribs. The stiffening effect of the wearing surface shall not be included in calculating the deflection of the deck plate.

(B) Vibrations

The vibrational characteristics of the bridge shall be considered in arriving at a proper design.

1.7.147 — WEARING SURFACE

A suitable wearing surface shall be adequately bonded to the top of the deck plate to provide a smooth, nonskid riding surface and to protect the top of the plate against corrosion and abrasion. The wearing surface material shall provide (1) sufficient ductility to accommodate, without cracking or debonding, expansion and contraction imposed by the deck plate, (2) sufficient fatigue strength to withstand flexural cracking due to deck-plate deflections, (3) sufficient durability to resist rutting, shoving, and wearing, (4) imperviousness to water and motor-vehicle fuels and oils, and (5) resistance to deterioration from deicing salts, oils, gasolines, diesel fuels, and kerosenes.

1.7.148 — CLOSED RIBS

Closed ribs without access holes for inspection, cleaning, and painting are permitted. Such ribs shall be sealed against the entrance of moisture by continuously welding (1) the rib webs to the deck plate, (2) splices in the ribs, and (3) diaphragms, or transverse beam webs, to the ends of the ribs.