TO:

SUPERSEDED BY EI 79-015
EFFECTIVE 4/24/1979

ENGINEERING INSTRUCTION

NEW YORK STATE DEPARTMENT OF TRANSPORTATION

SUBJECT: STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, Revision to

Subject Code: 7.35-4

Distribution: ☐ Main Office ☐ Regions ☒ Special

Code: EI 76-34

Date: May 4, 1976

Supersedes:
SEE BELOW

APPROVED: J. V. Hourigan
DEPUTY CHIEF ENGINEER (STRUCTURES)

Enclosed is the reprinted manual of the Standard Specification for Highway Bridges, dated April 1976.

This manual replaces in its entirety, the present manual, which should no longer be used.

Revisions not covered by previous Engineering Instructions, as well as those originating from the 1975 AASHTO Interim Specifications, are detailed below.

All page numbers and Article numbers refer to the 1976 manual.

SUPERSEDES:

- EI 72-017 DATE 2/24/72
- EI 72-034 DATE 4/3/72
- EI 72-045 DATE 4/18/72
- EI 73-021 DATE 3/21/73
- EI 73-088 DATE 12/3/73
- EI 74-086 DATE 9/6/74
- EI 74-101 DATE 10/29/74
- EI 75-003 DATE 1/8/75

PAGE	
1	Table of Contents - Design Section - Revised Design & (Detailing) Index - deleted. (Subject to future revision)
5	Article 1.1.8 - Curbs and Sidewalks - paragraph 3, revised
5	Article 1.1.9 - Railings - entire article rewritten
18	Article 1.2.6 - Traffic Lanes - paragraph 1 revise width from 10 to 12 feet
20	Article 1.2.11(c) - Railing Loading - revised
27	Article 1.2.22 - Loading Combinations - revised
39	Article 1.3.2(H)(2) - Railing Loads - revised
59	Article 1.4.7 - A. General - paragraphs 1 & 2 replace old paragraph 1
73	Article 1.5.7 - Minimum Reinforcement of Flexural Member - revised formula
75	Article 1.5.11(B) - Lateral Reinforcement - Sub item 2 - revised sentence now reading "The ties shall be so arranged...."
88	Article 1.5.23(J)(3) - Slab Thickness - added (4) - Diaphrams - Formerly (3)
88	Article 1.5.23(K)(2) - Compression Flange Width - revised

PAGE

89 Article 1.5.23(K)(3)(a) - Top and Bottom Slab Thickness - revised

98 Article 1.5.30(B) - Required Strength - revised

116 Article 1.6.5 - Load Factors - revised

120 Article 1.6.7(B) - Revised, except 1.6.7(B)(d) Covering Stress Relieved Strand and "Stablized Strand." Commentary - added pages 124 thru 136

139 Article 1.6.12(B) - Cast-in-Place Post Tensioned Bridges - revised. Commentary added - pages 139 thru 147

152 Article 1.6.17 - Post Tensioning Anchorages and Couplers - revised. Commentary added - pages 153 and 154

157 Article 1.6.24(B) - Effective Compression Flange Width - paragraph 1, last sentence - revised

166 Table 1.7.3c - Third "Stress Category" from top - revised from D to C

168 Article 1.7.4 - Paragraph now reading "steel forgings, carbon and alloy..." - revised

168 Table - revised Headings

170 Article 1.7.5 - Information on this page replaces former text at comparible location in Article.

175 Article 1.7.16 - Expansion and Contraction - revised Commentary added

221 Commentary added

232 Article 1.7.123 - Revised and Commentary deleted.

NEW YORK STATE DEPARTMENT OF TRANSPORTATION

Keep in file Room

STANDARD SPECIFICATION FOR HIGHWAY BRIDGES

Official Issuance System

Structures Design And Construction Subdivision

APRIL, 1976

STANDARD SPECIFICATION FOR HIGHWAY BRIDGES

APRIL, 1976

NEW YORK STATE DEPARTMENT OF TRANSPORTATION
RAYMOND T. SCHULER, Commissioner

STATE OF NEW YORK
HUGH L. CAREY, Governor

TABLE OF CONTENTS

DESIGN

Article		Page
	E. Maximum Design Load for Piles	52
	F. Uplift	52
	G. Group Pile Loading	52
1.4.5	Piles	52
	A. General	52
	B. Limitation of Use	53
	C. Design Loads	53
	D. Spacing, Clearances and Embedment	53
	E. Batter Piles	54
	F. Buoyancy	54
	G. Concrete Piles (Precast)	54
	H. Concrete Piles (Cast-in-Place)	55
	I. Steel H Piles	55
	J. Unfilled Tubular Steel Pipes	56
	K. Steel Pile and Steel Pile Shell Protection	57
1.4.6	Footings	57
	A. Depth	57
	B. Anchorage	57
	C. Distribution of Pressure	57
	D. Spread Footings	58
	E. Internal Stresses in Spread Footings	58
	F. Reinforcement	59
	G. Transfer of Stress from Vertical Reinforcement	59
1.4.7	Abutments	59
	A. General	59
	B. Reinforcement for Temperature	60
	C. Wing Walls	60
	D. Drainage	60
	E. Approach Slabs	60
1.4.8	Retaining Walls	61
	A. General	61
	B. Base or Footing Slabs	61
	C. Vertical Walls	61
	D. Counterforts and Buttresses	61
	E. Reinforcement for Temperature	61
	F. Expansion and Contraction Joints	61
	G. Drainage	61
1.4.9	Piers	62
	A. General	62
	B. Pier Nose	62
	C. Reinforcement	62
1.4.10	Tubular Steel Piers	62
	A. Use	62

Article		Page

Article		Page

Article		Page

Article		Page

* DESIGN *

DESIGN ANALYSIS

In any case where the specifications provide for an empirical formula as a design convenience, a rational analysis based on a theory accepted by the Committee on Bridges and Structures of the American Association of State Highway Officials, with stresses in accordance with the specifications, will be considered as compliance with the specifications.

Section 1 – GENERAL FEATURES OF DESIGN

1.1.1 – BRIDGE LOCATIONS

Selecting favorable stream crossings should be considered in the preliminary route determination to minimize construction, maintenance and replacement costs. Natural stream meanders should be studied and, if necessary, channel changes, river training works and other construction which would reduce erosion problems and prevent possible loss of the structure should be considered. Foundations of bridges placed across channel changes should be designed for possible deepening and widening of the relocated channel. On wide flood plains the lowering of approach fills to provide overflow sections designed to pass unusual floods over the highway is a means of preventing loss of structures. Where relief bridges are needed to maintain the natural flow distribution and reduce backwater, caution must be exercised in proportioning the size and in locating such structures to avoid undue scour or changes in the course of the main river channel.

1.1.2 - BRIDGE WATERWAYS

The determination of adequate waterway openings for stream crossings is essential to the design of safe and economical bridges. Hydraulic studies of bridge sites are a necessary part of the preliminary design of a bridge and reports of such studies should include applicable parts of the following outline:

A. Site Data******

1. Cross sections of the channel at right angles to the stream approximately 500 feet upstream and downstream and at the centerline of improvement. These sections must extend far enough up the banks that the bank height will be above any anticipated extreme high water.
2. The upstream and downstream channel sections should be indicative of the average channel section. The 500 feet distance upstream or downstream may be modified in order to provide a section which more closely provides an average channel section.
3. An estimate of the "n" factors applicable to the main channel flow and overbank flow for the stream must be provided with submission of bridge data. Examples of

applicable "n" factors may be obtained from roughness characteristics of natural channels (Geological Survey Water Supply paper no. 1849).

4. A stream channel profile must be provided for all bridge stream crossings for a minimum of 500 feet upstream and 500 feet downstream from the centerline of improvement.
5. In addition, all information requested on the bridge data sheet must be provided.

B. Hydraulic Analysis*****

1. A flood frequency curve must be provided with each bridge stream crossing.
2. A stage discharge curve must be provided for each bridge crossing a stream with a discharge greater than 2000 c.f.s.
3. The flood frequency curve and stage discharge curve together with a copy of all computations will be forwarded by the regional office.
4. If flood of record is known the frequency and stage of this flood should be noted in the bridge data submission.
5. If the regional office for any reason, finds that it cannot provide these curves the bridge data will be forwarded and the curves will be worked up by the Structures Design and Construction Subdivision.
6. If the regional office desires help in compiling these curves the Structures Design and Construction Subdivision will complete the work up of the curves and provide a copy to the regional office with any required explanation.
7. If the regional office does not have better information available, such as a gaging station at or near structure location etc. The flood frequency curve will be determined as follows.
8. When the drainage area is greater than 10 square miles use U.S. Geological Survey Circular No. 454 to determine the discharge for a 10 year storm, 25 year storm and 50 year storm. Use these points to draw a flood frequency curve. Extend the curve to determine a 100 year flood.
9. When the drainage area is less than 10 square miles, Bureau of Public Roads Engineering Circular No. 4, "Estimating Peak Rates Of Runoff From Small Watersheds In Portions Of New York", will be used to determine the flood frequency curve.
10. The stage discharge curve will be determined by the use of the electronic computer program for hydraulics of bridge waterways (Bureau of Public Roads Program HY-4).

If the bridge opening or type of structure is to be revised any required revisions of the stage discharge curve will be made by the Structures Design and Construction Subdivision.

Bridge waterways are sized to pass a design flood of a 50 year frequency or greatest flood of record, whichever is larger. In the selection of the waterway opening, consideration should be given to the amount of upstream ponding, the passage of ice and debris and possible scour of the bridge foundations. Where floods exceeding the design flood have occurred or where superfloods would cause extensive damage to adjoining property or the loss of a costly structure, a larger waterway opening may be warranted. Due consideration should be given to any Federal, State and local requirements.

Relief openings, spur-dikes, debris deflectors and channel training works should be used where needed to minimize the effect of adverse flood flow conditions. Where scour is likely to occur, protection against damage from scour should be provided for in the design of bridge piers and abutments. Embankment slopes adjacent to structures subject to erosion, should be adequately protected by rip-rap, flexible mattresses, retards, spur dikes or other appropriate construction. Clearing of brush and trees along embankments in the vicinity of bridge openings should be avoided to prevent high flow velocities and possible scour. Borrow pits should not be located in areas which would increase velocities and the possibility of scour at bridges.

1.1.3 – PIER SPACING, ORIENTATION AND TYPE

Piers shall be located to meet navigational clearance requirements and to give a minimum interference to flood flow. In general, piers should be placed parallel with the direction of the stream current at flood stage. Adequate provision should be made for drift and ice by increasing span lengths and vertical clearances, by selecting proper pier types and by using debris deflectors. Special precautions against scour are required when larger cofferdams are placed in unstable stream beds.

1.1.4 – CULVERT WATERWAY OPENINGS

Culverts, as distinguished from bridges, are usually smaller in waterway opening, covered with embankment material and composed of structure around the entire perimeter of the culvert barrel. Some culverts are supported on spread footings when the natural stream bed can serve as the bottom of the waterway without undermining the foundation.

Criteria for design, discharges, allowable headwater depth and outlet velocities will vary, depending upon the class of highway, hazards to traffic, risks of flooding adjacent property, risks of damaging embankments, stream bed material and other factors. Based on conditions at and in the vicinity of the culvert site, it may be determined that the culvert can be designed to operate satisfactorily under submergence. Generally, designs for culverts to operate under submergence are limited to floods of infrequent occurrence. Culverts should be designed to resist the hydraulic forces to be encountered and should be protected from undermining by means of adequate aprons, wingwalls, cutoff walls or other appropriate devices. Adjacent embankments should be protected against erosion as necessary by rip-rap or other suitable means.

1.1.5 – CULVERT LOCATION AND LENGTH

In general, culverts are located in natural stream channels. If foundations are poor, bearing piles, selected backfill material, a cambered profile or slip-collars can be provided to assure good alignment and a culvert that is structurally sound.

The length of culverts should be sufficient to prevent the embankment material from encroaching on a culvert end. Headwalls and endwalls with cutoff walls and aprons are used to protect the fill slopes and stream beds from erosion and to secure the culvert end against hydraulic forces. If head walls and end walls are required, they should be designed not to protrude above the ground line. Culvert openings shall be placed a minimum of 30 feet from the edge of the traffic lanes or protected by guard rail or other means. Where feasible, culverts shall be continuous across medians to avoid the traffic hazard presented by additional openings. Where needed, debris control devices should be constructed to prevent clogging. If backfill and embankment materials are subject to piping, consideration should be given to the use of cutoff walls or impervious material placed at the entrance.

1.1.6 – WIDTH OF ROADWAY AND SIDEWALK

The width of roadway shall be the clear width measured at right angles to the longitudinal center line of the bridge between the bottoms of curbs or if curbs are not used, the clear width shall be the minimum measured between the nearest faces of the bridge railing.

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 Guard, must be obtained from the U.S. Coast Guard 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 generally not be used on structures over features other than highways or railroads. However, curbs shall be used under the following conditions:

1. Where the approach highway is curbed.
2. On a high level bridge.
3. Where the shoulders are less than five feet in width.
4. Where curbs are required to control 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 on other than urban expressways, a traffic railing or barrier may be provided between the two with a pedestrian railing outside.

A. Traffic Railing

1. General

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 overcrossed, 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" (.01 m), whichever is less.

Careful attention shall be given to the treatment of railing at the bridge ends. Exposed rail ends, posts, and sharp changes in the geometry of the railing shall be avoided. A smooth transition by means of a continuation of the bridge barrier, guard rail anchored to the bridge end, or other effective means shall be provided to protect the traffic from direct collision with the bridge rail ends.

2. Loadings and Geometrics

Traffic railing and traffic railing portions of combination railings shall be no less than 2'-3" (.686 m) in height measured from the top of the roadway, the top of future overlay if future resurfacing is anticipated, or from the top of curb to the top of the upper rail member when the curb projection is greater than 9 inches (.229 m), 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″ (.813 m) in height.

The lower element of a traffic or combination railing should consist of either a parapet projecting at least 18″ (.457 m) above the reference surface or a rail centered between 15 and 20 inches (.381 and .508 m) above the reference surface. The roadway surface, or the top of the overlay, if future resurfacing is anticipated, is the reference surface unless there is a curb or sidewalk projecting more than 9 inches (.229 m) from the traffic face of the railing, in which case the surface of the curb or sidewalk is the reference surface.

When the height of the top of the top traffic rail exceeds 2′-9″ (.839 m) the total transverse load distributed to the traffic rails and posts shall be increased by the factor "C". However, the maximum load applied to any one element need not exceed "P".

For traffic railings, the maximum clear vertical opening between the lowest rail and reference surface shall not exceed 17 inches (.432 m) and the maximum opening between succeeding rails shall not exceed 15 inches (.381 m). For combination and pedestrian railings, the maximum clear vertical opening between the lowest rail and reference surface shall not exceed 15 inches (.381 m) and the maximum opening between suceeding rails not exceed 15 inches (.381 m).

The traffic faces of all traffic rails must be within one inch (.025 m) of a vertical plane through the traffic face of the rail closest to traffic. Rails a greater distance behind this plane or centered less than 15 inches (.381 m) above the reference surface shall not be considered traffic rails for the purpose of distributing the transverse highway design loading, "P", or "CP", but may be considered in determining the maximum clear vertical opening, provided they are designed for a transverse loading equal to that applied to an adjacent traffic rail or "P/2" whichever is the lesser.

Transverse loads on posts, equal to "P", or "CP", shall be distributed as shown in Figure 1.1.9. A load equal to ½ 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 ¼ the outward transverse load.

The attachment of each rail required in a traffic or combination railing shall be designed to resist a vertical load equal to ¼ of the transverse design load of the rail. The vertical load shall be applied alternately upward or downward. The attachment shall also be designed to resist an inward transverse load equal to ¼ the transverse rail design load.

Rail members shall be designed for a moment, due to concentrated loads, at the center of the panel and at the posts of $P'L/6$ where P' is equal to P, P/2, or P/3, or as modified by the factor "C", where required. The handrail members of combination railing shall be designed for a moment at the center of the panel and at the posts of $0.1wL^2$. L is the post spacing.

The transverse force on concrete parapet and barrier walls shall be spread over a longitudinal length of 5 feet (1.524 m).

Refer to Figure 1.1.9 for the loads and points of application.

Railings other than those shown in Figure 1.1.9 are permissible provided they meet the requirements of this Article, except that railing configurations which have been successfully tested by full scale impact tests are exempt from the provisions of this Article. (The national Cooperative Highway Procedures for Vehicle Crash Testing of Highway Appurtenances, describes procedures for crash testing bridge railings.)

3. Design

Railings shall be designed by the elastic method to the allowable stresses for the appropriate materiate material. For aluminum alloys 6061-T6, 6063-T6, and 6351-T5, the design stresses given in Tables 1.5.2A(1), 1.5.2A(2), and 1.5.2A(3) of the "AASHTO Standard Specifications for Structural Supports for Signs, Luminaires, and Traffic Signals." For Alloy A444-T4, 35% of the values listed in Table 1.5.2A(1), and for Alloys A356-T61 and 356-T6, T7, 45% of the values listed in Table 1.5.2A(1) shall be used for design. Aluminum railings shall be fabricated and built in accordance with the provisions of Section 6 of April 1969 "Specifications for Aluminum Bridge and other Highway Structures" published by the Aluminum Association for riveted and bolted fabrication, and in accordance with Section 1.5.5 of the "AASHTO Standard Specifications for Structural Supports for Signs, Luminaires, and Traffic Signals," for welded fabrication.

The allowable unit stresses for steel shall be as given in Article 1.7.1, except as modified below.

For steels not generally covered by the "Standard Specifications," but having a guaranteed yield strength. F_y, the allowable unit stress, shall be derived by applying the general formulas as given in the "Standard Specifications" under "Unit Stresses" except as indicated below.

The allowable unit stress for shear shall be $F_y = 0.33\ F_y$.

Steel round or oval tubes may be proportioned using an allowable bending stress, $F_b = 0.66\ F_y$ provided the R/t ratio (radius/thickness) is less than or equal to 40.

Square and rectangular tubes, and WF and I sections in bending with tension and compression on extreme fibers of laterally supported compact sections having an axis of symmetry in the plane of loading may be designed for an allowable stress $F_b = 0.60\ F_y$.

The requirements for a compact section are as follows:

a. The width to thickness ratio of projecting elements of the compression flange of WF and I sections shall not exceed $1600/\sqrt{F_y}$ or $(133/\sqrt{F_y})$ in S.I.
b. The width to thickness ratio of the compression flange of square or rectangular tubes shall not exceed $6000/\sqrt{F_y}$ or $(499/\sqrt{F_y})$ in S. I.
c. The D/t ratio of webs shall not exceed $13{,}300/\sqrt{F_y}$ or $(1106/\sqrt{F_y})$ in S.I.

d. If subject to combined axial force and bending, the D/t ratio of webs shall not exceed

$$13{,}300\,\frac{1-1.43\left(\frac{f_a}{F_a}\right)}{\sqrt{F_y}}$$

$$\text{or}\quad\left(1106\,\frac{1-1.43\left(\frac{f_a}{F_a}\right)}{\sqrt{F_y}}\right)\ \text{in S.I.}$$

but need not be less than $7000/\sqrt{F_y}$ or $(581/\sqrt{F_y})$ in S.I.

e. The distance between lateral supports in inches (metres) of WF or I sections shall not exceed $2400b/\sqrt{F_y}$ or $(199.2\ b/\sqrt{F_y})$ in S.I.

$$\frac{20{,}000{,}000\ A_f}{dF_y}\quad\text{or}\quad\left(\frac{137{,}640\ A_f}{dF_y}\right)\ \text{in S.I.}$$

B. Pedestrian Railing

1. General

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 thereof.

2. Loading and Geometrics

The minimum height of pedestrian railing shall be 3'-0" (.914 m) [a preferred height is 3'-6" (1.066 m)]measured from the top of the walkway to the top of the upper rail member.

The minimum design loading for pedestrian railing shall be = 50 lbs. per lin. ft. (74.5 kg/m) acting simultaneously transversely and vertically on each longitudinal member. Rail members located more than 5'-0" (1.524 m) above the walkway are excluded from these requirements.

Posts shall be designed for a transverse load of wl acting at the center of gravity of the upper rail, or for high rails, or high rails, at 5'-0" (1.524 m) maximum above the walkway.

Refer to Figure 1.1.9 for more information concerning the application of loads.

3. Design

See Article 1.1.9(A)(3).

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 gradient. 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 the FHWA circular, HEC No. 12 – Drainage of Highway Pavements, March 1969, Memorandum No. 2 on surface drainage, the design being on the rainfall intensity of the most severe storn of five-minute duration likely to occur in a ten-year period.

Bridge drains are to be provided to maintain a maximum puddle width determined by the following conditions:

A. Maximum puddle width is limited to twelve feet (12′).
B. Maximum puddle depth is one-half inch less than the curb height.
C. Puddle may encroach into the through lane or auxiliary lane nearest the curb only to a point where eight feet (8′) of lane width are left open for traffic.

1.1.11 - SUPERELEVATION*****

The superelevation 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 characteristics.

1.1.13 – BLAST PROTECTION (Deleted)

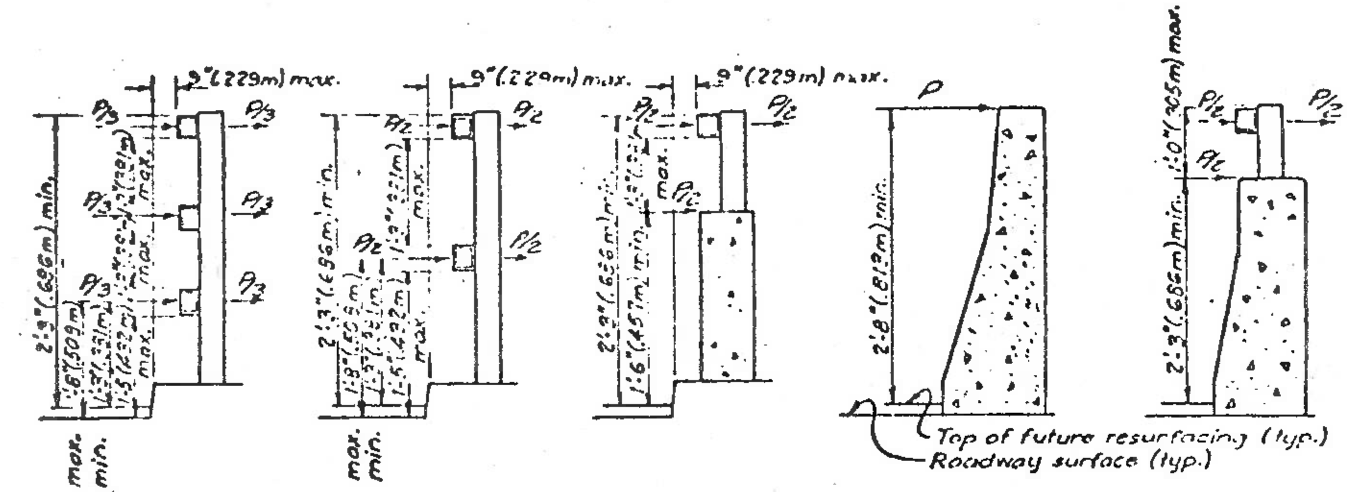
1.1.14 – UTILITIES

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)

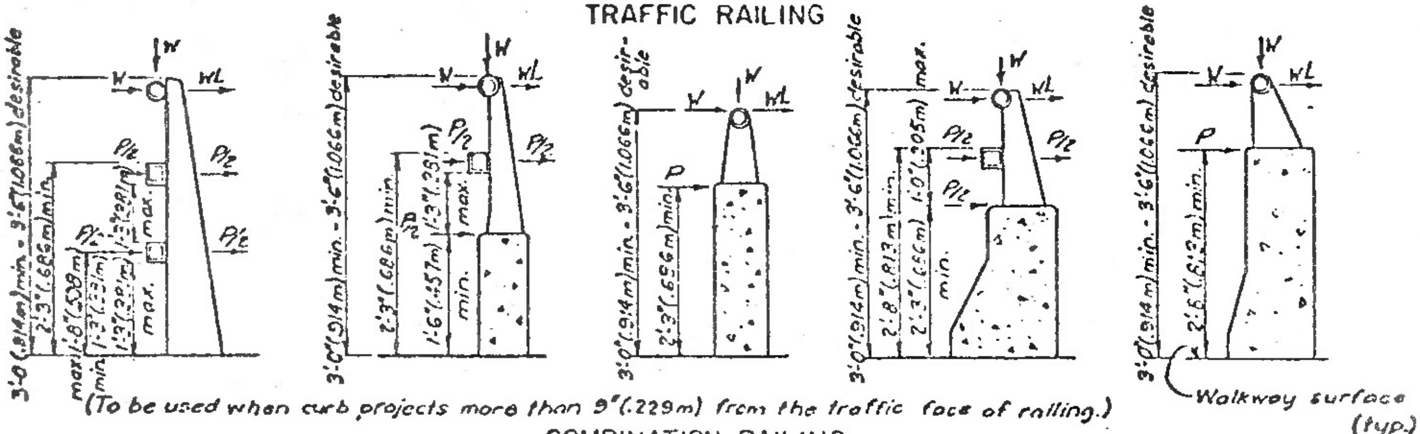
1.1.16 and 1.1.17 – ROADWAY WIDTH, CURBS AND CLEARANCES FOR DEPRESSED ROADWAYS AND UNDERPASSES

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



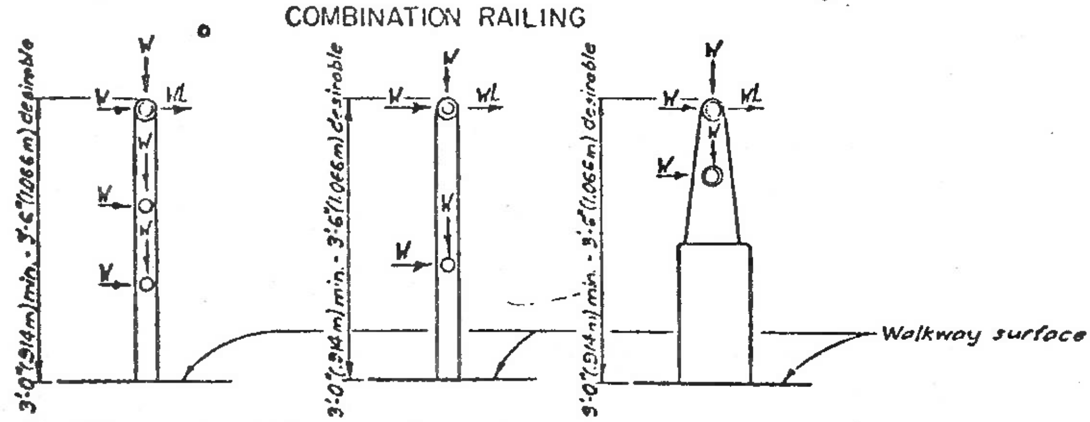
(To be used where there is no curb or curb projects 9"(.229m) or less from traffic face of railing.)

TRAFFIC RAILING



(To be used when curb projects more than 9"(.229m) from the traffic face of railing.)

COMBINATION RAILING



(To be used adjacent to a sidewalk when highway traffic is separated from pedestrian traffic by a traffic railing.)

PEDESTRIAN RAILING

Notes: Loadings on left are applied to rails.
Loadings on right are applied to posts.
The shapes of rail members are illustrative only. Any material or combination of materials listed in Art. 1.1.9 may be used in any configuration.

Nomenclature: P = Highway design loading = 10 kips (44.48 kN)
h = Height of top of top rail above reference surface (in.) (metre)
L = Post spacing (ft.) (metre)
W = Pedestrian loading = 50 pounds per linear foot (74.45 kg/m)
$C = 1 + \frac{h-33}{18} \geq 1$ $\left[1 + \frac{h(m)}{.457} - \frac{.33}{.13}\right]$ in S.I. units

FIGURE 1.1.9

Section 2 – LOADS

1.2.1 – LOADS*****

Structures shall be proportioned for the following loads and forces when they exist:

Dead Load.

Live Load.

Impact or Dynamic Effect of the Live Load.

Wind Loads.

Other Forces, when they exist, as follows:

Longitudinal forces, centrifugal force, thermal forces, earth pressure, buoyancy, shrinkage stresses, rib shortening, erection stresses, ice and current pressure, earthquake stresses, and construction conditions.

Members shall be proportioned using allowable stresses and design limitations for the appropriate material.

Upon the stress sheets a diagram or notation of the assumed loads shall be shown and the moments, shears, and stresses due to the various loads shall be shown separately.

Where required by design conditions, the concrete placing sequence shall be indicated on the plans or in the special provisions.

The loading combinations shall be in accordance with Article 1.2.22.

1.2.2 – DEAD LOAD*****

The dead load shall consist of the weight of the structure complete, including the roadway, sidewalks and car tracks. pipes, conduits, cables and other public utility services.

The snow and ice load is considered to be offset by an accompanying decrease in live load and impact and shall not be included except under special conditions.

Where traffic is to bear directly on the concrete slab, an additional 1 inch cover over the top of the reinforcement shall be added.

All structures having a monolithic wearing surface shall be designed for a possible additional wearing surface weighting 20 pounds per sq. ft.

The following weights are to be used in computing the dead load:

	Weight per cubic foot pounds
Steel or Cast steel	490
Cast iron	450
Aluminum alloys	175
Timber (treated or untreated)	50
Concrete, plain or reinforced	150
Compacted sand, earth, gravel or ballast	120
Loose sand, earth and gravel	100
Macadam or gravel, rolled	140
Cinder filling	60

Pavement 150
Stone masonry 170
Asphalt plank. i inch thick . . . . 9 lbs. per square foot

A. Unit Load on Culverts

Earth pressures or loads on culverts may be computed ordinarily as the weight of earth directly above the structure. For box culverts and culverts with cast-in-place inverts or footings, the weight of the earth may be taken at 70 percent of its actual weight. This will have the effect of increasing the allowable design dead load stresses 40% more than allowed for live load. For flexible and rigid pipes, not cast-in-place, the weight of the earth may be taken at 83% of its actual weight. This will have the effect of increasing the allowable design load stresses 20% more than allowed for live load.

For definite conditions of bedding and backfill, the principles of soil mechanics may be applied. The following are recommended formulas for these conditions:

1. Culvert in trench on unyielding subgrade, or culvert untrenched on yielding foundation.

$$P = WH$$

2. Culvert untrenched on unyielding foundation (such as rock or piles)

$$P = W(1.92H - 0.87B) \text{ for } H > 1.7B \qquad (1A)$$

$$P = 2.59BW\ (e \quad - 1) \text{ for } H > = \frac{0.385H}{R} \qquad (1B)$$

where:

P = The unit pressure, in pounds per sq. ft., due to the backfill.
B = width, in ft., of trench, or if untrenched, the overall width of culvert.
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.

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 loading of weights other than those designated are desired, they shall be obtained by proportionately changing the weight shown for both the standard truck and the corresponding lane loads.

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

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:

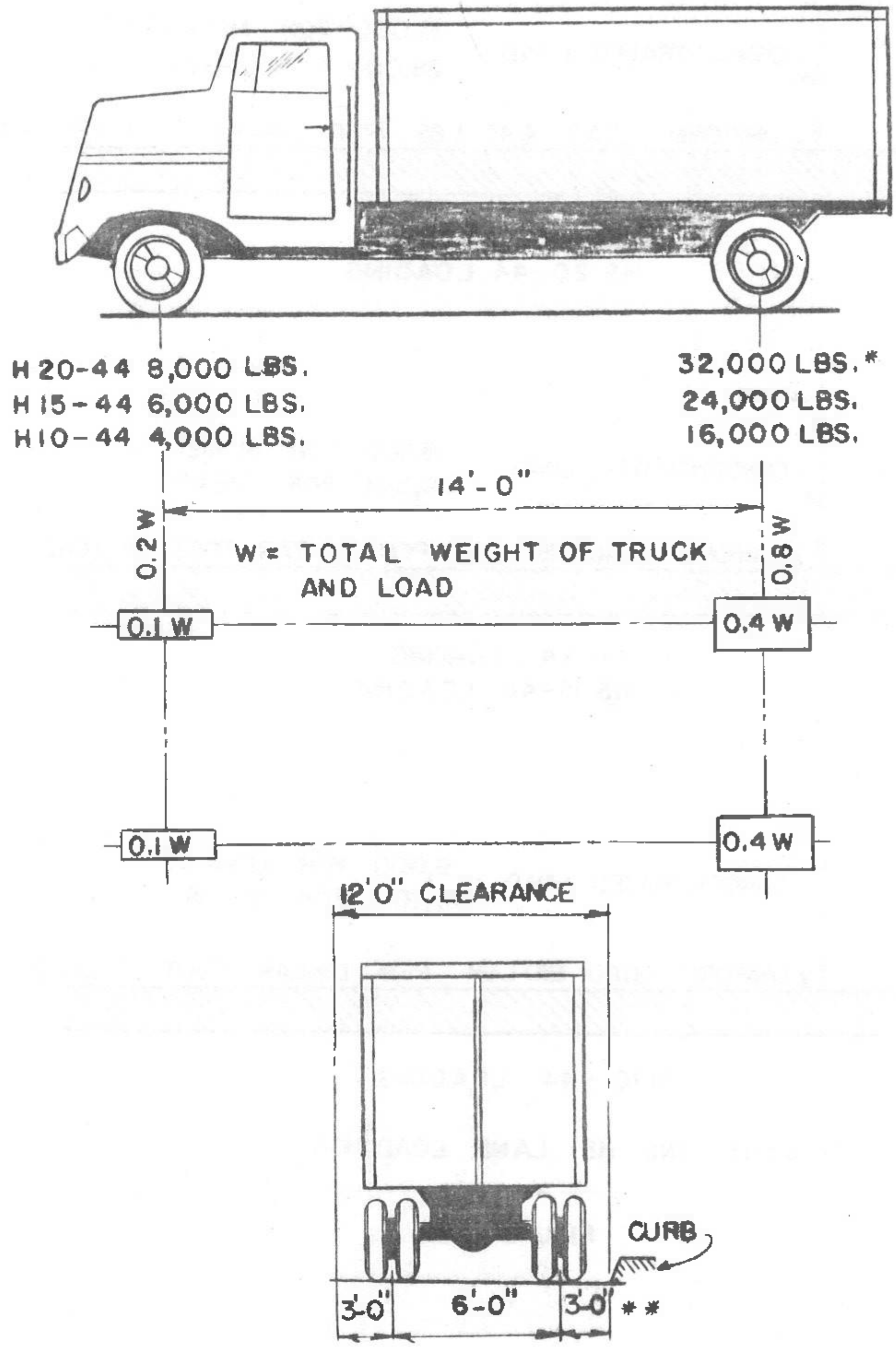
H10 Loading, 1944 Edition shall be designated H10-44
H15 Loading, 1944 Edition shall be designated H15-44
H20 Loading, 1944 Edition shall be designated H20-44
H15-S12 Loading, 1944 Edition shall be designated HS15-44
H20-S16 Loading, 1944 Edition shall be designated HS20-44

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 loading 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.

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.

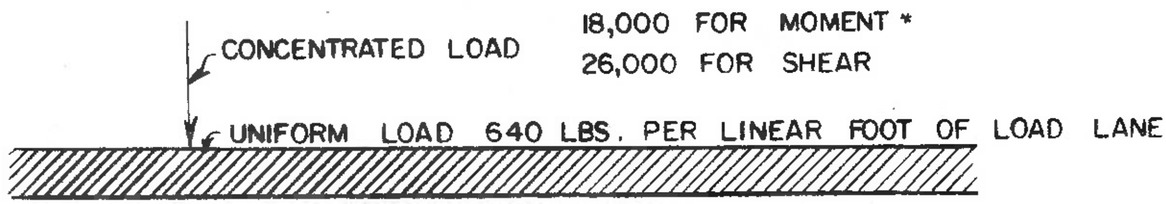


STANDARD H TRUCKS

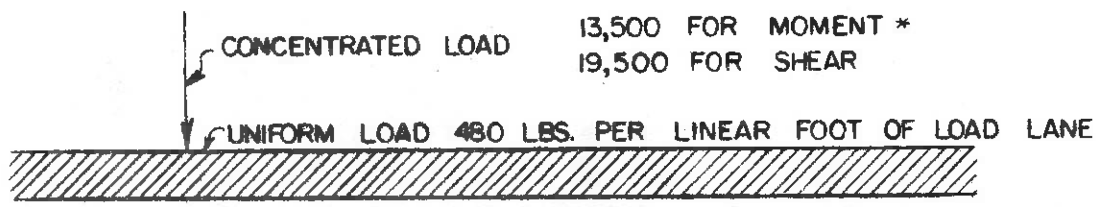
FIGURE 1.2.5 A

*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.

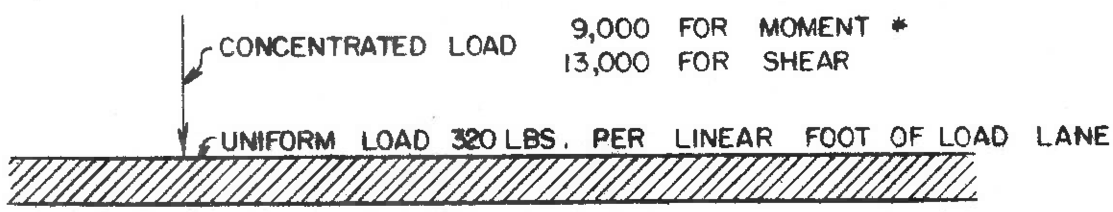
**For slab design, the center line of wheels shall be assumed to be 1 foot from face of curb. (See Art. 1.3.2(B))



H20-44 LOADING
HS 20-44 LOADING



H 15-44 LOADING
HS 15-44 LOADING

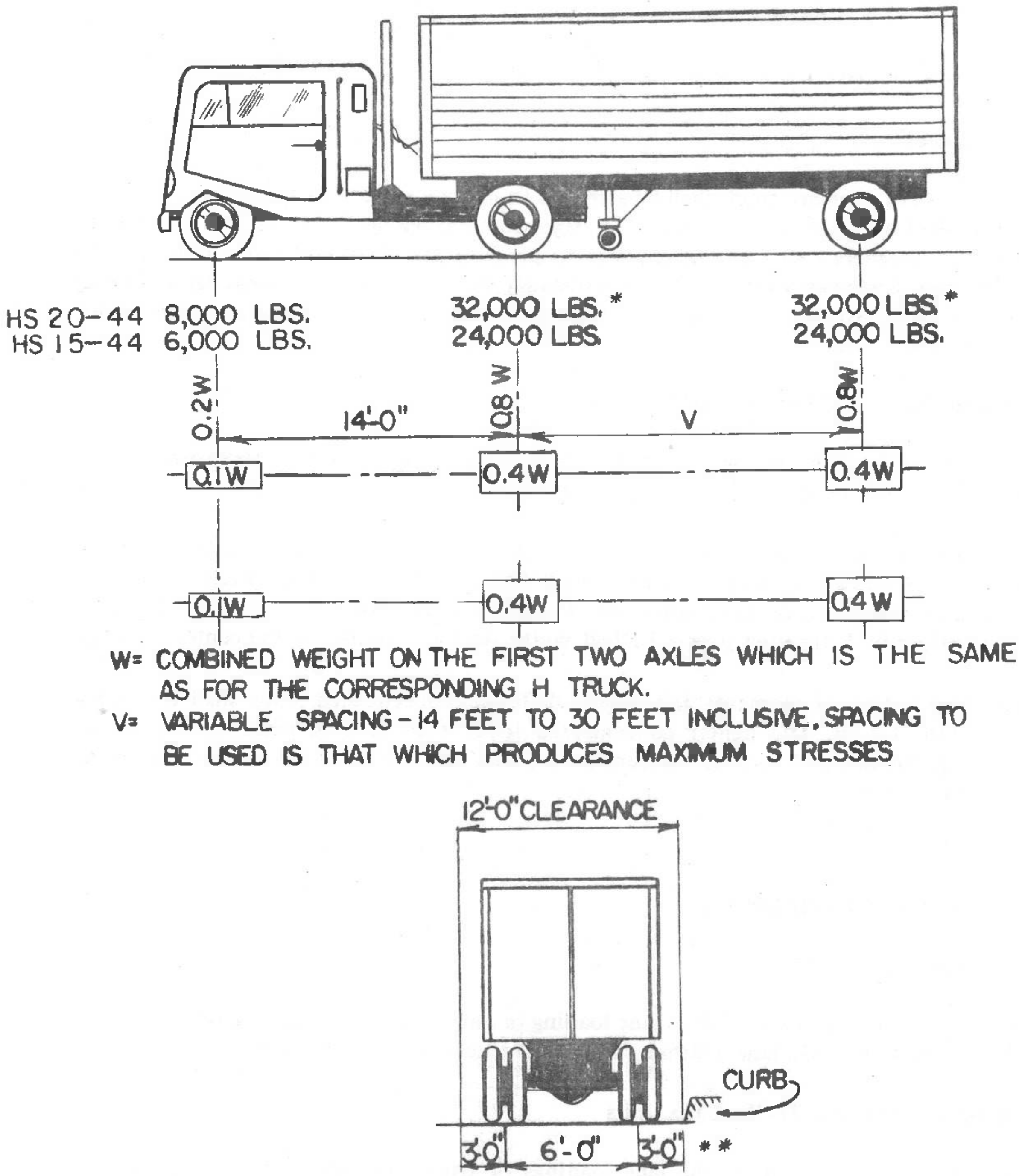


H 10-44 LOADING

H LANE AND HS LANE LOADINGS

FIGURE 1.2.5 B

*For the loading of continuous spans involving lane loading, refer to Art. 1.2.8(c) which provides for an additional concentrated load.



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))

1.2.6 – Traffic Lanes

The lane loading or standard truck shall be assumed to occupy a width of twelve feet.

These loads shall be placed in 12-foot wide design traffic lanes, spaced across the entire bridge roadway width, in numbers and positions required to produce the maximum stress in the member under consideration. Roadway width shall be the distance between curbs. fractional parts of design lanes shall not be used. Roadway widths from 20 to 24 feet shall have two design lanes each equal to one-half the roadway width.

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 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 Loading-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 table given in Article 1.3.7 shows 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 discontinuous, 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 maximum 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

$$P = \left(30 + \frac{3000}{L}\right)\left(\frac{55 - W}{50}\right)$$

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

For Railing Loads, see Article 1.1.9.

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:

$$I = \frac{50}{L + 125} \text{ in which}$$

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.

For computing truck load moments use the span length, except for cantilever arms use the length from moment center to the farthermost axle.

For shear due to truck loads use the length of the loaded portion of span from the point under consideration to the far reaction, except for cantilever arms use 30 per cent.

For continuous spans use the length of span under consideration for positive moment, and use the average of two adjacent loaded spans for negative moment.

For culverts with cover 0 ft.-0 in. to 1 ft.-0 in. incl. I = 30%

For culverts with cover 1 ft.-1 in. to 2 ft.-0 in. incl. I = 20%

For culverts with cover 2 ft.-1 in. to 2 ft.-11 in. incl. I = 10%

1.2.13 – LONGITUDINAL FORCES*****

Provision shall be made for the effect of a longitudinal force of five per cent of the live load in all lanes carrying traffic headed in the same direction. All lanes shall be considered as loaded for bridges likely to become one directional in the future. The load used, without impact, shall be the lane load plus the concentrated load for moment specified in Article 1.2.8, with reduction for multiple-loaded lanes as specified in Article 1.2.9. The center of gravity of the longitudinal force shall be assumed to be located 6 feet above the floor slab and transmitted to the substructure through the superstructure. The longitudinal force due to friction at expansion bearings shall also be provided for in the design. For sliding type bearings, this force may be taken as 15% of the dead load. For rocker type bearings, the force shall be based on a 15% friction coefficient on the pin, and shall be reduced in proportion to the radii of the pin and the rocker.

1.2.14 – WIND LOADS*****

The following wind load forces per square foot of exposed area shall be applied to all structures (see Article 1.2.22 for percentage of basic unit stress to be used under various combinations of loads and forces). The exposed area considered shall be the sum of the areas of all members, including floor system and railing, as seen in elevation at 90 degrees to the longitudinal axis of the structure. The forces and loads given herein are for a wind velocity of 100 miles per hour.

The wind loads may be reduced in intensity as indicated below. The reduction is based on a Map of Wind Speeds Throughout the United States, developed by the U.S. Weather Bureau in cooperation with the Bureau of Public Roads.

In Region 10, the area of Long Island south of a line from Rockaway Inlet through Port Jefferson, the wind loads may be reduced 10% up to 30 feet in height and no reduction over 30 feet.

In Regions 3, 4, 5, 6, the area of Region 7 within 20 miles of Lake Ontario or the St. Lawrence River and Region 10 except the area indicated above, the reduction may be 30% up to 30 feet; 24% over 30 feet to 50 feet; 12% over 50 feet to 100 feet and no reduction over 100 feet.

In Regions 1, 2, 8, 9 and 7 (except area shown above) the reduction may be 50% up to 30 feet; 46% over 30 feet to 50 feet; 38% over 50 feet to 100 feet; 26% over 100 feet to 300 feet and 18% over 300 feet.

All heights are measured from the average ground or water surface to the center of the superstructure in the span having the greatest clearance above the ground or water.

A. Superstructure Design

A moving uniformly distributed wind load of the following intensity shall be applied horizontally at right angles to the longitudinal axis of the structure in the design of the superstructure:

For trusses and arches . . . . 75 pounds per square foot
For girders and beams . . . . 50 pounds per square foot

The total force shall not be less than 300 pounds per linear foot in the plane of the loaded chord and 150 pounds per linear foot in the plane of the unloaded chord on truss spans, and not less than 300 pounds per linear foot on girder spans.

The above forces shall be used for Group II loading. For Group III loading there shall be added thereto a load of 100 pounds per linear foot applied at right angles to the longitudinal axis of the structure and 6 feet above the deck as a wind load on a moving live load. When a reinforced concrete floor slab or a steel grid deck is keyed to or attached to its supporting members, it may be assumed that the deck resists, within its plane, the shear resulting from the wind load on the moving live load.

B. Substructure Design

Forces transmitted to the substructure by the superstructure and forces applied directly to the substructure by wind loads shall be assumed to be as follows:

1. Forces From Superstructure*****

The transverse and longitudinal forces transmitted by the superstructure to the substructure for varying angles of wind direction shall be as set forth in the following table. The skew angle is measured from the perpendicular to the longitudinal axis. The assumed wind direction shall be that which produces the maximum stress in the substructure being designed. The transverse and longitudinal forces shall be applied simultaneously at the elevation of the center of gravity of the exposed area of the superstructure.

	Trusses		Girders	
Skew Angle of Wind (Degrees)	Lateral Load Per Sq. Ft. of Area (Pounds)	Longitudinal Load Per Sq. Ft. of Area (Pounds)	Lateral Load Per Sq. Ft. of Area (Pounds)	Longitudinal Load Per Sq. Ft. of Area (Pounds)
0	75	0	50	0
15	70	12	44	6
30	65	28	41	12
45	47	41	33	16
60	25	50	17	19

The loads listed above shall be used in Group II loading as given in Article 1.2.22.

For Group III loading, these loads may be reduced 70 per cent and there shall be added thereto, as a wind load on a moving live load, a load per linear foot as given in the following table:

Skew Angle of Wind (Degrees)	Lateral Load Per Lin. Ft. (Pounds)	Longitudinal Load Per Lin. Ft. (Pounds)
0	100	0
15	88	12
30	82	24
45	66	32
60	34	38

This load shall be applied at a point 6 feet above the deck.

For multiple stringer type superstructures having a concrete deck slab, the following wind loading may be used in lieu of the more precise loading specified above:

W (wind load on structure)
50 pounds per square foot, transverse;
12 pounds per square foot, longitudinal.
Both forces shall be applied simultaneously.

WL (wind load on live load)
100 pounds per linear foot, transverse;
40 pounds per linear foot, longitudinal.
Both forces shall be applied simultaneously.

2. Forces Applied Directly To The Substructure

The transverse and longitudinal forces to be applied directly to the substructure for a 100 mile per hour wind shall be calculated from an assumed wind force of 40 pounds per square foot. For wind directions assumed skewed to the substructure this force shall be resolved into components perpendicular to the end and front elevations of the substructure according to the functions of the skew angle. The component perpendicular to the end elevation shall act on the exposed substructure area as seen in end elevation and the component perpendicular to the front elevation shall act on the exposed substructure area as seen in front elevation. These loads shall be assumed to act on horizontal lines at the centers of gravity of the exposed areas and shall be applied simultaneously with the wind loads from the superstructure.

In lieu of the above loading a wind force of 32 pounds per square foot may be applied to the end elevation of the substructure with a simultaneous wind force of 8 pounds per square foot on the front elevation.

The above loads are for Group II loading and may be reduced 70 per cent for Group III loadings, as indicated in Article 1.2.22.

C. Overturning Forces

The effect of forces tending to overturn structures shall be calculated under Group II and Group III of Article 1.2.22, and there shall be added an upward force applied at the windward quarter point of the transverse superstructure width. This force shall be 20 pounds per square foot of deck and sidewalk plan area for Group II combination and 6 pounds per square foot for Group III combination. The wind direction shall be assumed to be at right angles to the longitudinal axis of the structure.

1.2.15 – THERMAL FORCES*****

Provisions shall be made for stresses or movements resulting from variations in temperature. The rise and fall in temperature shall be fixed for the locality in which the structure is to be constructed and shall be figured from an assumed temperature at the time of erection. Due consideration shall be given to the lag between air temperature and the interior temperature of massive concrete members or structures.

The range of temperature shall generally be as follows:

Metal Structures

Moderate climate, from 0 degree to 120 degree F.

Cold climate, from –30 degree to 120 degree F.

	Temperature rise	Temperature fall
Concrete Structures		
Moderate climate	30. degree F.	40 degree F.
Cold climate	35 degree F.	45 degree F.

Region 10 may be considered having a moderate climate and the remainder of the state having a cold climate.

1.2.16 – UPLIFT

Provision shall be made for adequate attachment of the superstructure to the substructure by engaging a mass of masonry equal to the largest force obtained under one of the following conditions:

a. 100% of the calculated uplift caused by any loading or combination of loadings in which the live plus impact loading is increased by 100%.
b. 150% of the calculated uplift at working load level.

Anchor bolts subject to tension or other elements of the structure stress under the above conditions shall be designed at 150% of the allowable basic stress.

1.2.17 – FORCE OF STREAM CURRENT, FLOATING ICE AND DRIFT

All piers and other portions of structures which are subject to the force of flowing water, floating ice, or drift shall be designed to resist the maximum stresses induced thereby.

The pressure of ice on piers shall be calculated at 400 pounds per square inch. The thickness of ice and height at which it applies shall be determined by investigation at the site of the structure.

The effect of flowing water on piers shall be calculated by the formula:

$P = KV^2$, where
P = pressure in pounds per square foot,
V = velocity of water in feet per second,
K = a constant, being 1 3/8 for square ends, 1/2 for angle ends where the angle is 30 degrees or less, and 2/3 for circular piers.

1.2.18 – BUOYANCY

Buoyancy shall be considered as it affects the design of either substructure, including piling, or the superstructure.

1.2.19 – EARTH PRESSURE

Structures which retain fills shall be proportioned to withstand pressure as given by Rankine's formula; provided, however, that no structure shall be designed for less than an equivalent fluid pressure of 30 pounds per cubic foot.

For rigid frames a maximum of one-half of the moment caused by earth pressure (lateral) may be used to reduce the positive moment in the beams, in the top slab, or in the top and bottom slab, as the case may be.

When highway traffic can come within a horizontal diatance from the top of the structure equal to one-half its height, the pressure shall have added to it a live load surcharge pressure equal to not less than 2 feet of earth.

Where an adequately designed reinforced concrete approach slab supported at one end by the bridge is provided, no live load surcharge need be considered.

All designs shall provide for the thorough drainage of the back-filling material by means of weep holes and crushed rock, pipe drains or gravel drains, or by perforated drains.

1.2.20 – EARTHQUAKE STRESSES

In regions where earthquakes may be anticipated, provision shall be made to accommodate lateral forces from earthquakes as follows:

$$EQ = CD$$

where

EQ = Lateral force applied horizontally in any direction at center of gravity of the weight of the structure.

D = Dead load of structure.

C = 0.02 for structures founded on spread footings on material rated as 4 tons or more per square foot.

C = 0.04 for structures founded on spread footings on material rated as less than 4 tons per square foot.

C = 0.06 for structures founded on piles.

Live load may be neglected.

1.2.21 – CENTRIFUGAL FORCES

Structures on curves shall be designed for a horizontal radial force equal to the following percentage of the live load, without impact, in all traffic lanes:

$$C = 0.00117\ S^2 D = \frac{6.68 S^2}{R}$$

where

C = the centrifugal force in percent of the live load, without impact.

S = the design speed, in miles per hour.

D = the degree of curve.

R = the radius of the curve, in feet.

The effects of superelevation shall be taken into account.

The centrifugal force shall be applied 6 feet above the roadway surface, measured along the center line of the roadway. The design speed shall be determined with regard to the amount of superelevation provided in the roadway. The traffic lanes shall be loaded in accordance with the provisions of Article 1.2.8.

Each design traffic lane shall be loaded with one standard truck (lane loading shall not be used in any case) placed in position for maximum loading.

When a reinforced concrete floor slab or a steel grid deck is keyed to or attached to its supporting members, it may be assumed that the deck resists, within its plane, the shear resulting from the centrifugal forces acting on the live load.

1.2.22–LOADING COMBINATIONS

The following Groups represent various combinations of loads and forces to which a structure may be subjected. Each component of the structure, or the foundation on which it rests, shall be proportioned to withstand safely all group combinations of these forces that are applicable to the particular site or type. Group loading combinations for Service Load Design and Load Factor Design are given by:

$$\begin{aligned}\text{Group (N)} = \; & \Upsilon\,[\beta_D \cdot D + \beta_L\,(L + I) + \beta_C CF + \beta_E E + \beta_B B + \beta_S SF + \\ & \beta_W W + \beta_{WL} WL + \beta_L \cdot LF + \beta_F F + \beta_R\,(R + S + T) + \\ & \beta_{EQ} EQ + \beta_{ICE} ICE]\end{aligned}$$

where

N = group number
Υ = load factor, see Table 1.2.22
β = coefficient, see Table 1.2.22

For service load design the percentage of the basic unit stress for the various groups is shown in Table 1.2.11.

See Articles 1.2.1 to 1.2.21 for loads and forces expressed in each group. The maximum section required shall be used.

For load factor design, the gamma and beta factors given in Table 1.2.22 are only intended for designing structural members by the load factor concept. The actual loads should not be increased by the factors shown in the table when designing for foundations (soil pressure, pile loads, etc.). The load factors are not intended to be used when checking for foundation stability (safety factors against over-turning, sliding, etc.) of a structure.

When structures (usually long span) are being proportioned by load factor design the gamma and beta factors specified for Load Factor Design represent usual conditions and should be increased, if in the engineer's judgment, the predictability of loads are different than anticipated by the specifications. If predictability of service load conditions or materials of construction are different than anticipated these should be accounted for in the appropriate service load analyses or unit stress increase percentages respectively.

D = Dead Load
L = Live Load
I = Live Load Impact
E = Earth Pressure
B = Buoyancy
W = Wind Load on Structure
WL = Wind Load on Live Load–100 lbs. per lin. ft. (1458N/m)
LF = Longitudinal Force from Live Load
CF = Centrifugal Force
F = Longitudinal force due to friction or shear resistance (elastomeric bearings)
R = Rib Shortening
S = Shrinkage
T = Temperature
EQ = Earthquake
SF = Stream Flow Pressure
ICE = Ice Pressure

For Working Stress Design

% (Column 15) Percentage of Basic Unit Stress

No increase in allowable unit stresses shall be permitted for members or connections carrying wind loads only.

β_E = See Culvert Loading specifications Art. 1.2.2(A)

β_E = 1.0 and 0.5 for lateral loads on rigid frames (check both loadings to see which one governs). See Art. 1.2.19.

TABLE 1.2.22

Table of Coefficients Υ and β

	Column No.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
			β FACTORS													
	Group	Υ	D	L+I	CF	E	B	SF	W	WL	LF	F	R+S+T	EQ	ICE	%
WORKING STRESS DESIGN	I	1.0	1	1	1	β_E	1	1	0	0	0	0	0	0	0	100
	II	1.0	1	0	0	1	1	1	1	0	0	0	0	0	0	125
	III	1.0	1	1	1	β_E	1	1	0.3	1	1	1	0	0	0	125
	IV	1.0	1	1	1	β_E	1	1	0	0	0	0	1	0	0	125
	V	1.0	1	0	0	1	1	1	1	0	0	0	1	0	0	140
	VI	1.0	1	1	1	β_E	1	1	0.3	1	1	1	1	0	0	140
	VII	1.0	1	0	0	1	1	1	0	0	0	0	0	1	0	133
	VIII	1.0	1	1	1	1	1	1	0	0	0	0	0	0	1	140
	IX	1.0	1	0	0	1	1	1	1	0	0	0	0	0	1	150
LOAD FACTOR DESIGN	I	1.3	β_D	1.67	1.0	β_E	1	1	0	0	0	0	0	0	0	
	IA	1.3		2.20	0	0	0	0	0	0	0	0	0	0	0	
	II	1.3		0	0	β_E	1	1	1	0	0	0	0	0	0	
	III	1.3		1	1		1	1	0.3	1	1	1	0	0	0	Not applicable
	IV	1.3		1	1		1	1	0	0	0	0	1	0	0	
	V	1.25		0	0		1	1	1	0	0	0	1	0	0	
	VI	1.25		1	1		1	1	0.3	1	1	1	1	0	0	
	VII	1.3		0	0		1	1	0	0	0	0	0	1	0	
	VIII	1.3		1	1		1	1	0	0	0	0	0	0	1	
	IX	1.20		0	0		1	1	1	0	0	0	0	0	1	

For Load Factor Design

For all loadings less than H20, provision shall be made for an infrequent heavy load by applying Group 1A loading, with the live load assumed to occupy a single lane without concurrent loading in any other lane

β_E = 1.3 for lateral earth pressure and 0.5 for checking positive moments in rigid frames

β_E = 1.0 for verticla earth pressure

β_D = 0.75 when checking member for minimum axial load and maximum moment or maximum eccentricity — for Column Design

β_D = 1.0 when checking member for maximum axial load and minimum moment — for Column Design

β_D = 1.0 for flexural and tension members

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 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:

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

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 4" thick or multiple layer floors over 5" thick	S/4.5	S/4.0
Strip 6" or more thick	S/5.0 If S exceeds 5' use footnote [2].	S/4.25 If S exceeds 6.5' use footnote [2].
Concrete:		
On Steel or Precast Concrete Stringers	S/7.0 If S exceeds 10' use footnote[2].	S/5.5 If S exceeds 14' use footnote [2].
On Adjacent Prestressed Concrete Box Beams	S/6.0	S/6.0
On Concrete T-Beams	S/6.5 If S exceeds 6' use footnote[2].	S/6.0 If S exceeds 10' use footnote [2].
On Timber Stringers	S/6.0 If S exceeds 6' use footnote[2].	S/5.0 If S exceeds 10' use footnote [2].
Concrete box girders See footnote[1]	S/8.0 If S exceeds 12' use footnote[2].	S/7.0 If S exceeds 16' use footnote [2].
On Steel Box Girders	(See Art. 1.7.103)	
On Prestressed Concrete Spread Box Beams	See Art. 1.6.24(A)	
Steel Grid:		
(less than 4" thick)	S/4.5	S/4.0
(4" or more)	S/6.0 If S exceeds 6.0' use footnote [2].	S/5.0 If S exceeds 10.5' use footnote [2].

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 as a simple beam. However, the provisions of Article 1.2.6 do not apply.

2. Outside Roadway Stringers and Beams

(a) Steel – Timber – Concrete T-Beams*****

The dead load considered as supported by the outside roadway stringer or beam shall be that portion of the floor slab carried by the stringer or beam, curbs, railings, wearing surface and parapets, if placed after the slab has cured, may be considered equally distributed to all roadway stringers or beams. Sidewalk liveload shall be distributed in the same manner as superimposed D.L.

The live load bending moment for outside roadway stringers or beams shall be determined by applying to the stringer or beam the reaction of the wheel load obtained by assuming the flooring to act as a simple span between stringers or beams.

When the outside beam or stringer supports the sidewalk live load as well as traffic live load and impact, the allowable stress in the beam or stringer may be increased 25 percent for the combination of dead load, sidewalk live load, traffic live load, and impact, providing the beam is of no less carrying capacity than would be required if there were no sidewalks. In no case shall an exterior stringer have less carrying capacity than an interior stringer.

In the case of a span with concrete floor supported by 4 or more steel stringers, the fraction of the wheel load shall not be less than:

$\frac{S}{5.5}$ Where S = 6 ft. or less

$\frac{S}{4.0 + 0.25S}$ Where S is more than 6 ft. and less than 14 ft.

When S is 14 ft. or more, use Note 2, Article 1.3.1 (B) (1)

S = distance in feet between outside and adjacent interior stringers.

(b) Concrete Box Girders

The dead load considered as supported by the exterior girder shall be determined in the same manner as for steel, timber, concrete T-Beams, as given in (a) above.

The wheel load distribution to the exterior girder shall be

$$\frac{W_e}{7}$$

W_e = Width of exterior girder. The width to be used in determining the wheel line distribution to the exterior girder shall be the top slab width as measured from the midpoint between girders to the outside edge of the slab. The cantilever dimension of any slab extending beyond the exterior girder shall preferably not exceed S/2.

3. Total Capacity of Stringers

The combined design load capacity of all the beams and stringers in a span shall not be less than required to support the total live and dead load in the span.

C. Bending Moment in Floor Beams (Transverse)

In calculating bending moments in floor beams no transverse distribution of the wheel loads shall be assumed.

If longitudinal stringers are omitted and the floor is supported directly on floor beams, the beams shall be designed for loads determined in accordance with the following Table:

Kind of Floor	Fraction of wheel load to each floor beam
Plank	S/4
Strip 4 inches in thickness, wood block on 4-inch plank subfloor or multi-thickness plank more than 5 inches thick	S/4.5
Strip 6 inches or more in thickness	S/5 (see note below)
Concrete	S/6 (see note below)
Steel Grid (less than 4 in. thick)	S/4.5
Steel Grid (4 inches or more)	S/6 (see note below)

S = spacing of beams in feet.

Note: If S exceeds denominator, the load on the beam shall be the reaction of the wheel loads assuming the flooring between beams to act as a simple beam.

D. Multibeam Precast Concrete Beams

A multi-beam bridge is constructed with precast reinforced or prestressed concrete beams which are placed side by side on the supports. The interaction between the beams is developed by continuous longitudinal shear keys and lateral bolts which may, or may not, be prestressed.

In calculating bending moments in multi-beam precast concrete bridges, conventional or prestressed, no longitudinal distribution of wheel load shall be assumed. The lateral distribution shall be determined as follows:

1. Load Fraction

The live load bending moment for each section shall be determined by applying to the beam the fraction of a wheel load (both front and rear) determined by the following relations:

$$\text{Load Fraction} = \frac{S}{D}$$

Where:

$$S = \frac{12 N_L + 9}{N_g}$$

$$D = 5 + \frac{N_L}{10} + \left(3 - \frac{2N_L}{7}\right)\left(1 - \frac{C}{3}\right)^2 \qquad C \leqslant 3$$

$$D = 5 + \frac{N_L}{10} \qquad C > 3$$

N_L = Total number of traffic lanes from Article 1.2.6
N_g = Number of longitudinal beams
C = K(W/L), a stiffness parameter
W = Overall width of bridge
L = Span length, feet

VALUES OF K TO BE USED IN C = K(W/L)

Bridge Type	Beam Type and Deck Material	K
Multi-beam	Nonvoided rectangular beams	0.7
	Rectangular beams with circular voids	0.8
	Box section beams	1.0
	Channel beams	2.2

1.3.2 – DISTRIBUTION OF LOADS AND DESIGN OF CONCRETE SLABS

A. Span Lengths (see also Article 1.5.3)

For simple spans the span length shall be the distance center to center of supports but not to exceed clear span plus thickness of slab.

The following effective span lengths shall be used in calculating distribution of loads and bending moments for slabs continuous over more than two supports:

Slabs monolithic with beams or walls (without haunches)
S = clear span.

Slabs supported on steel stringers, S = distance between edges of flanges plus 1/2 of stringer flange width.

Slabs supported on timber stringers, S = clear span plus 1/2 thickness of stringer.

B. Edge Distance of Wheel Load

In designing slabs the center line of wheel load (axle load/2) shall be assumed to be one foot from the face of the curb. If curbs or sidewalks are not used, the wheel load shall be assumed to be one foot from the face of the rail. Combined dead, live and impact stresses shall be not greater than the allowable stresses.

In designing sidewalks, slabs and supporting members, a wheel load shall be assumed to be one foot from the face of the rail. Combined dead, live and impact stresses for this loading shall be not greater than 150 percent of the allowable stresses. Wheel loads shall not be applied on sidewalks protected by a traffic barrier.

C. Bending Moment

Bending moment per foot width of slab shall be calculated according to methods given under Cases A and B, unless a more exact method is used.

In Cases A and B:

S	=	Effective span length, in feet, as defined under "span lengths" Articles 1.3.2(A) and 1.5.3
E	=	Width of slab in feet over which a wheel is distributed
P	=	Load on one rear wheel of truck (P15 or P20)
P15	=	12000 pounds for H15 loading
P20	=	16000 pounds for H20 loading

Case A—Main Reinforcement Perpendicular to Traffic (Spans 2 to 24 feet, inclusive)

The live load moment for simple spans shall be determined by the following formulas (impact not included):

HS20 Loading:

$$\frac{(S+2)}{32} \quad P20 = \text{Moment in ft.-pounds per ft. width of slab.}$$

HS15 Loading:

$$\frac{(S+2)}{32} \quad P15 = \text{Moment in ft. width of slab.}$$

In slabs continuous over three or more supports, a continuity factor of 0.8 shall be applied to the above formulas for both positive and negative moment.

Case B—Main Reinforcement Parallel to Traffic

Distribution of wheel loads, E = 4 + 0.06S, maximum 7.0 feet. Lane loads are distributed over a width of 2E. Longitudinally reinforced slabs shall be designed for the appropriate HS loading.

For simple spans, the maximum live load moment per foot width of slab, without impact, is closely approximated by the following formulas:

HS20 Loading:

Spans up to and including 50 feet: LLM = 900S foot-pounds
Spans 50 feet to 100 feet
LLM = 1000 (1.30S–20.0) foot-pounds

HS15 Loading:

Use 3/4 of the values obtained from the formulas for HS20 loading.

Moments in continuous spans shall be determined by suitable analysis using the truck or appropriate lane loading.

D. Edge Beams (Longitudinal)

Edge beams shall be provided for all slabs having main reinforcement parallel to traffic. The beam may consist of a slab section additionally reinforced, a beam integral with and deeper than the slab, or an integral reinforced section of slab and curb.

It shall be designed to resist a live load moment of 0.10 PS, where:

P = Wheel load in pounds P15 or P20
S = Span length, in feet

This formula gives the simple span moment. Values for continuous spans may be reduced 20 per cent unless a greater reduction results from a more exact analysis.

E. Distribution Reinforcement*****

Reinforcement shall be placed in the bottoms of all slabs transverse to the main steel reinforcement, to provide for the lateral distribution of the concentrated live loads, except that this specification will not apply on culverts or bridge slabs when the depth of fill over the slab exceeds two feet. The amount shall be the percentage of the main reinforcement steel required for positive moment as given by the following formulas:

For main reinforcement parallel to traffic:

$$\text{Percentage} = \frac{100}{\sqrt{S}} \quad \text{Maximum } 50\%$$

For main reinforcement perpendicular to traffic:

$$\text{Percentage} = \frac{220}{\sqrt{S}} \quad \text{Maximum } 67\%$$

where S = the effective span length, in feet.

For main reinforcement perpendicular to traffic the specified amount of distribution reinforcement shall be used in the middle half of the slab span. In the outer quarters of the slab span not less than 50 percent of the above amount shall be used.

Closed-box culverts on yielding foundations under fills exceeding two feet in depth, shall have the following longitudinal reinforcement placed in the tops of the top slabs and bottoms of the bottom slabs. The total amount of this steel (square inches) shall be computed by multiplying the area (square inches per foot) of steel required to develop the positive moment in the top slab by 35% and by the clear span of the opening of the box.

F. Shear and Bond Stress in Slabs

Slabs designed for bending moment in accordance with the foregoing shall be considered satisfactory in bond and shear.

G. Unsupported Edges, Transverse

The design assumptions of this article do not provide for the effect of loads near unsupported edges. Therefore, at the ends of the bridge and at intermediate points where the continuity of the slab is broken, the edges shall be supported by diaphragms or other suitable means. The diaphragms shall be designed to resist the full moment and shear produced by the wheel loads which can come on them.

H. Cantilever Slabs

1. Truck Loads

Under the following formulas for distribution of loads on cantilever slabs, the slab is designed to support the load independent of edge support along the end of the cantilever. The distribution given includes the effect of wheels on parallel elements.

Case A—Reinforcement Perpendicular to Traffic

Each wheel on the element perpendicular to traffic shall be distributed according to the following formula:

$$E = 0.8X + 3.75$$

Moment per foot of slab $= \frac{P}{E} X$ foot-pounds in which X = distance in feet from load to point of support.

Case B—Reinforcement Parallel to Traffic

The distribution for each wheel load on the element parallel to traffic shall be as follows:

$$E = .35X + 3.2 \text{ but shall not exceed } 7.0 \text{ feet.}$$

Moment per foot of slab $= \frac{P}{E} X$ foot-pounds.

2. Railing Loads

It shall be assumed that the effective length of slab resisting post loadings is equal to $E = 0.8X + 3.75$ feet ($0.8X + 1.143$ m) where no parapet is used and is equal to $E = 0.8X + 5.0$ feet ($0.8X + 1.524$ m) where a parapet is used, where X is the distance in feet (metres) from the center of the post to the point under investigation. Railing and wheel loads are not to be applied simultaneously. Railing loading shall be applied in accordance with Article 1.1.9.

I. Slabs Supported on Four Sides

In the case of slabs supported along four edges and reinforced in both directions, the proportion of the load carried by the short span of the slab shall be assumed as given by the following equations:

For load uniformly distributed, $P = \dfrac{B^4}{A^4 + B^4}$

For load concentrated at center, $P = \dfrac{A^3}{A^3 + B^3}$

where:

P = proportion of load carried by short span.
A = length of short span of slab.
B = length of long span of slab.

Where the length of the slab exceeds 1 1/2 times its width, the entire load shall be assumed to be carried by the transverse reinforcement.

The distribution width, E, for the load taken by either span shall be determined as provided for other slabs. Moments obtained shall be used in designing the center half of the short and long slabs. The reinforcement steel in the outer quarters of both short and long spans may be reduced 50 per cent. In the design of the supporting beams, consideration shall be given to the fact that the loads delivered to the supporting beams are not uniformly distributed along the beams.

J. Median Slabs

Raised median slabs shall be designed in accordance with the provisions of this article with truck loadings so placed as to produce maximum stresses. Combined dead, live and impact stress may be not greater than 150 percent of the allowable stresses. Flush median slabs shall be designed without any overstress.

1.3.3 – DISTRIBUTION OF WHEEL LOADS THROUGH EARTH FILLS*****

When the depth of fill is 2 feet or more, concentrated loads shall be considered as uniformly distributed over an area equal to 1 3/4 times the depth of the fill, including pavement, longitudinally and 6 feet transversely.

The shear produced by such loads shall be calculated as provided for dead loads.

For single spans the effect of live load may be neglected when the depth of fill is more than 8 feet and exceeds the span length; for multiple spans it may be neglected when the depth of fill exceeds the distance between faces of end supports or abutments. When the depth of fill is less than 2 feet the wheel load shall be distributed as in slabs with concentrated loads. When the calculated live load and impact moment in concrete slabs, based on distribution of the wheel load through fills, as herein outlined, exceeds the live load and impact moment calculated according to Article 1.3.2, then the latter moment shall be used.

1.3.4 – DISTRICUTION OF WHEEL LOADS ON TIMBER FLOORING

For the calculation of rending moments in timber flooring, each wheel load shall be distributed as follows:

A. Flooring Transverse

In direction of span:

Over width of tire (10 inches for H10, 15 inches for H15, and 20 inches for H20 loading).

Normal to direction of span:

Plank floor: width of plank.
Laminated Floor: 15 inches.
Splined or Doweled Floor: Not less than 5 1/2 inches thick: 4 times thickness.

For transferse flooring, the span shall be taken as the clear distance between stringers plus one-half the width of one stringer, but shall not exceed the clear span plus the floor thickness.

"For glued laminated panel decks using vertically laminated lumber with the panel placed in a transverse direction to the stringers, the determination of the deck thickness shall be based on the following equations for maximum unit primary moment and shear. The maximum shear is for a wheel position assumed to be 15 inches or less from the centerline of the support. The maximum moment is for a wheel position assumed to be centered between the supports."

Thus

$$M_x = P\left[(.51 \log_{10} s) - K\right]$$

$$R_x = .034P$$

$$t = \sqrt{\frac{6M_x}{F_b}}$$

or

$$t = \frac{3R_x}{2F_v}$$

whichever is greater

where

M_x = primary bending moment (in.-lb./in.)
R_x = primary shear (lb/in)
x = denotes direction perpendicular to longitudinal stringers
P = design wheel load (lb)
s = effective deck span (in)
t = deck thickness (in) based on moment or shear, which controls
K = design constant depending on design load as follows:

H10: K = 0.44
H15: K = 0.47
H20: K = 0.51

F_b = allowable bending stress, psi, based on load applied parallel to the wide face of the laminations. See Table 1.10.B.

F_v = allowable shear stress, psi, based on load applied parallel to the wide face of the laminations. See Table 1.10.1B.

The determination of the minimum size and spacing required of the steel dowels required to transfer the load between panels is based on the following equation:

$$n = \frac{1000}{\sigma PL}\left(\frac{\bar{R}_y}{R_D} + \frac{\bar{M}_y}{M_D}\right)$$

where

n = number of steel dowels required for given span, s

σ PL = proportional limit stress perpendicular to grain (for Douglas Fir Southern Pine, use 1000 psi)

R_y = total secondary shear transferred (lb) determined by the relationship

R_y = 6Ps/1000 for s $\leq$ 50 in.

or

$$\bar{R}_y = \frac{P}{25}(s-20)$$

M_y = total secondary moment transferred (in-lb) determined by the relationship

$$\bar{M}_y = \frac{Ps}{1600}(s-10)$$

or

$$\bar{M}_y = \frac{Ps}{20}\left(\frac{s-30}{s-10}\right)$$

$R_D M_D$ = shear and moment capacities as given in the following table:

Diameter of Dowel d	Shear Capacity R_D	Moment Capacity M_D	Steel Stress Coefficients C_R	Steel Stress Coefficients C_M	Total Dowel Length Required
In.	Lb.	In.-Lb.	$In.^2$	$In.^3$	In.
0.5	600	850	36.9	81.5	8.50
.625	800	1,340	22.3	41.7	10.00
.75	1,020	1,960	14.8	24.1	11.50
.875	1,260	2,720	10.5	15.2	13.00
1.0	1,520	3,630	7.75	10.2	14.50
1.125	1,790	4,680	5.94	7.15	15.50
1.25	2,100	5,950	4.69	5.22	17.00
1.375	2,420	7,360	3.78	3.92	18.00
1.5	2,770	8,990	3.11	3.02	19.50

In addition, the dowels shall be checked to insure that the allowable stress of the steel is not exceeded using the following equation:

$$\sigma = \frac{\frac{C_R}{R_y} + C_M \overline{M_y}}{n}$$

where

σ = allowable fiber stress of steel pins (psi) (see Table 1.7.1)
n, R_y, M_y = as previously defined
C_R, C_M = steel stress coefficients as given in preceding table.

B. Flooring Longitudinal

In direction of span:
Point Loading

Normal to direction of span:
Plank floor: width of plank
Laminated floor: width of wheel plus thickness of floor
Splined or doweled floor, not less than 5 1/2 inches thick: width of wheel plus twice thickness of floor

For longitudinal flooring the span shall be taken as the clear distance between floor beams plus one-half the width of one beam but shall not exceed the clear span plus the floor thickness.

C. Continuous Flooring

If the flooring is continuous over more than two spans the maximum bending moment shall be assumed as being 80 per cent of that obtained for a simple span.

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

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

*Continuous beam of 2 equal spans.

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.

The span lengths, edge distance of wheel load, edge beams (longitudinal) and unsupported edges (transverse) shall be as specified for concrete slabs, Article 1.3.2.

For H20 or HS20 loads use 32000 pound axle load.

B. Floors Filled with Concrete

The strength of the composite steel and concrete slab shall be determined by means of the "transformed area" method. The allowable stresses shall be as set forth in Articles 1.5.1 and 1.7.1.

C. When Investigating for Fatigue, use minimum cycles of maximum stress.

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

1.3.7 – MOMENTS, SHEARS AND REACTIONS*****

Maximum moments, shears, and reactions are given in the following Table for HS20 loadings. (Simple spans, one lane, impact not included).

Span	Moment	End shear and end Reaction	Span	Moment	End shear and end Reaction
1	8.0(b)	32.0(b)	39	432.1(b)	54.8(b)
2	16.0(b)	32.0(b)	40	449.8(b)	55.2(b)
3	24.0(b)	32.0(b)	42	485.3(b)	56.0(b)
4	32.0(b)	32.0(b)	44	520.9(b)	56.7(b)
5	40.0(b)	32.0(b)	46	556.5(b)	57.3(b)
6	48.0(b)	32.0(b)	48	592.1(b)	58.0(b)
7	56.0(b)	32.0(b)	50	627.9(b)	58.5(b)
8	64.0(b)	32.0(b)	52	663.6(b)	59.1(b)
9	72.0(b)	32.0(b)	54	699.3(b)	59.6(b)
10	80.0(b)	32.0(b)	56	735.1(b)	60.0(b)
11	88.0(b)	32.0(b)	58	770.8(b)	60.4(b)
12	96.0(b)	32.0(b)	60	806.5(b)	60.8(b)
13	104.0(b)	32.0(b)	62	842.4(b)	61.2(b)
14	112.0(b)	32.0(b)	64	878.1(b)	61.5(b)
15	120.0(b)	34.1(b)	66	914.0(b)	61.9(b)
16	128.0(b)	36.0(b)	68	949.7(b)	62.1(b)
17	136.0(b)	37.7(b)	70	985.6(b)	62.4(b)
18	144.0(b)	39.1(b)	75	1,075.1(b)	63.1(b)
19	152.0(b)	40.0(b)	80	1,164.9(b)	63.6(b)
20	160.0(b)	41.6(b)	85	1,254.7(b)	64.1(b)
21	168.0(b)	42.7(b)	90	1,344.4(b)	64.5(b)
22	176.0(b)	43.6(b)	95	1,434.1(b)	64.9(b)
23	184.0(b)	44.5(b)	100	1,524.0(b)	65.3(b)
24	192.7(b)	45.3(b)	110	1,703.6(b)	65.9(b)
25	207.4(b)	46.1(b)	120	1,883.3(b)	66.4(b)
26	222.2(b)	46.8(b)	130	2,063.1(b)	67.6
27	237.0(b)	47.4(b)	140	2,242.8(b)	70.8
28	252.0(b)	48.0(b)	150	2,475.1	74.0
29	267.0(b)	48.8(b)	160	2,768.0	77.2
30	282.1(b)	49.6(b)	170	3,077.1	80.4
31	297.3(b)	50.3(b)	180	3,402.1	83.6
32	312.5(b)	51.0(b)	190	3,743.1	86.8
33	327.8(b)	51.6(b)	200	4,100.0	90.0
34	343.5(b)	52.2(b)	220	4,862.0	96.4
35	361.2(b)	52.8(b)	240	5,688.0	102.8
36	378.9(b)	53.3(b)	260	6,578.0	109.2
37	396.6(b)	53.8(b)	280	7,532.0	115.6
38	414.3(b)	54.3(b)	300	8,550.0	122.0

(b) Maximum value determined by Standard Truck Loading. (One HS truck). Otherwise the Standard Lane Loading governs.

1.4.1 – ALLOWABLE STRESSES

Concrete, steel or timber substructures and retaining walls shall be designed for the unit stresses indicated in Section 5, Section 7 or Section 10.

1.4.2 – BEARING POWER OF FOUNDATION SOILS DETERMINATION OF BEARING POWER

When required by the Engineer, the bearing power of the soil in excavated foundation pits shall be determined by loading tests.

The following tabulation of the bearing power of broad basic groups of materials may be used as an aid to the judgment in the absence of more definite information:

	Safe Bearing Power Tons Per Square Foot	
Material	Min.	Max.
Alluvial soils	½	1
Clays	1	4
Sand, confined	1	4
Gravel	2	4
Cemented-sand and-gravel	5	10
Rock	5	...

Loading tests have a limited depth influence and may not disclose long-time consolidation.

When the consolidation of foundation soils causes the settlement of the backfill against an abutment or the settlement of the soil under an abutment which is placed on piles driven through a fill, the load transmitted may result in overloading the piles.

When the hydraulic gradient is increased as in excavating material from below the water table, foundation soils may be loosened by the upward flow of water. Such a condition should be guarded against.

Intrusion failures should be prevented by requiring a base course between rip rap and fine soils and by requiring proper gradation of drainage backfill behind abutments.

1.4.3 – ANGLES OF REPOSE

Earth, loam	30° to 45°
Dry sand	25° to 35°
Moist sand	30° to 45°
Wet sand	15° to 30°
Compact earth	35° to 40°
Gravel	30° to 40°
Cinders	25° to 40°
Coke	30° to 45°
Coal	25° to 35°

In the absence of exact data, which has been determined by field investigation and soil analysis, the angle of repose of the material shall be assumed to be the minimum given in the table.

1.4.4 – BEARING VALUE OF PILING

A. General

The design loads for piles shall not be greater than the minimum value which shall be determined for Case A, Case B and Case C; where Case A is the capacity of the pile as a structural member, Case B is the capacity of the pile to transfer its load to the ground and Case C is the capacity of the ground to support the load delivered to it by the pile or piles. The values assignable to each of the three cases shall be determined by making subsurface investigations or tests of sufficient extent to justify the assumed design values used for the particular condition of support under consideration.

In determining the bearing value of piles for use in designing, consideration shall be given to all information available relative to the subsurface conditions. Consideration shall also be given to:

1. The difference between the supporting capacity of a single pile and a group of piles.
2. The capacity of the underlying strata to support the load of the pile group.
3. The effect of driving additional piles and the effect of their loads on adjacent structure.
4. The possibility of scour and its effect.

B. Case A. Capacity of Pile as a Structural Member

1. Structural Columns

Piles shall be designed as structural columns. Timber piles shall be designed in accordance with Article 1.10.2, using the allowable unit stresses given in Article 1.10.1 for lumber and in the following table for round piles.

Species	Round Timber Piles Allowable unit working stress pounds per sq. in. compression parallel to grain for normal duration of loading
Ash, white	1200
Beech	1300
Birch	1300
Chestnut	900
Cypress, southern	1200
Cypress, tidewater red	1200
Douglas fir, coast type	1200
Douglas fir, inland	1100
Elm, rock	1300
Elm, soft	850
Gum, black and red	850
Hemlock, eastern	800
Hemlock, west coast	1000
Hickory	1650
Larch	1200
Maple, hard	1300
Oak, red and white	1100
Pecan	1650
Pine, lodgepole	800
Pine, Norway	850
Pine, southern	1200
Pine, southern, dense	1400
Poplar, yellow	800
Redwood	1100
Spruce, eastern	850
Tupelo	850

Concrete piles shall be designed in accordance with Article 1.5.1, steel piles in accordance with Article 1.7.1, and concrete-filled pipe piles in accordance with Article 1.5.1, except that the allowable unit stresses may be increased 20% provided the shell thickness is not less than 1/4 inch. The area of the shell may be included in determining the value of "P", (percentage of reinforcement). Where corrosion may be expected a suitable amount may be deducted from the shell thickness to allow for reduction in section by corrosion. The allowable stresses of Articles 1.5.1, 1.7.1 and 1.10.1 may be used in all cases where all of the stresses to which the piles may be subjected have been included. These stresses may be increased in accordance with Article 1.2.22. For trestle piles or other piles without lateral support designed for dead load and live load only and where temperature, traction, water pressure and other forces are not considered, the allowable unit stresses specified in Articles 1.5.1, 1.7.1 and 1.10.1 shall be decreased 20%.

2. Required Subsurface Investigations

Subsurface investigations shall be made which will determine the probable depth of scour or flotation of material and the condition of lateral support of the pile.

C. **Case B. Capacity of Pile to Transfer Load to the Ground**

1. Point-Bearing Piles

A pile shall be considered to be a point-bearing pile when placed or driven on or into a material which is capable of developing the pile load by direct bearing at the point with reasonable factor of safety.

The allowable load at the tip of the pile shall not exceed the following:

(a) For round timber piles, use values tabulated in Article 1.4.4(B) for allowable compression parallel to grain.

For sawn timber piles, use those values applicable to "wet condition" for allowable compression parallel to grain, in accordance with Article 1.10.1.

(b) For concrete piles, 0.33 in accordance with Article 1.5.1(B).

*(c) For concrete-filled piles, 0.40 in accordance with Article 1.5.1(B) applied to the total actual area of the concrete and steel.

(d) For steel H-piles and unfilled tubular steel piles, 9000 psi over the cross sectional area of the pile tip, not including the area of any pile tip reinforcement.

2. Friction Piles

A pile shall be considered to be a friction pile if its point does not rest on or in a material which is capable of developing the pile load by direct bearing at the point.

The load-carrying capacity of friction piles shall be determined by one or more of the following methods:

(a) Driving and loading test piles.

(b) Pile-driving experience in the vicinity. When piles are designed on the basis of experience in the vicinity, due consideration will be given to the variation in pile types and lengths, and in the variation of the soil strata. Where possible, the complete driving records of the piles in the vicinity shall be examined and compared to the driving records of the project piles.

*Note: The limitation in (C) and (D) govern except where the point bearing capacity of the piles is determined by loading test piles.

(c) Adequate tests of the soil strata through which the pile is to be driven. These tests should be projected and compared, if possible, to tests of similar material through which piles of known capacity have been driven.

3. Required Subsurface Investigations

(a) Point-bearing piles. Sufficient borings shall be made to determine the presence, position, and thickness of the material which is capable of developing point-bearing, and the log of borings shall show the nature of the overlying strata in order that the extent of lateral support may be determined. If the point-bearing stratum is of doubtful thickness and quality, the borings shall be made to such sufficient depth below this stratum that the capacity of a friction pile may be determined.

(b) Friction piles. Borings shall be made to an elevation well below the expected elevation of the pile tips and accurate logs of these borings shall be made. In those cases where the piles are to be designed on the basis of soil tests, undisturbed samples shall be taken on all strata which will have appreciable influence on the capacity of the pile.

(c) Combination point-bearing and friction piles. Piles shall be classified as either (1) point-bearing or (2) friction. Those cases where adequate strength is developed by both point-bearing and friction may be designed under either or these classifications.

D. Case C. Capacity of the Ground to Support the Load Delivered by the Pile

Preference shall be given to the determination of maximum loads on piles by test loading or by satisfactorv subgrade investigation.

The capacity of the ground to support the load delivered by the pile shall be determined from the results of the applicable subsurface investigations:

1. Point-Bearing Piles

Sufficient borings shall be made to determine the thickness and quality of the stratum in which the point bearing is developed. If that stratum is of sufficient thickness and is underlain by a firm material, no reduction will be made for group action of piles. In general, piles should not rest on a thin stratum of hard material which is underlain by a thick stratum of soft or yielding material, but where this condition cannot be avoided, group action should be considered and the design loads reduced accordingly.

2. Friction Piles

Borings shall be carried well below the tips of the piles in order to determine the characteristics of the underlying material. In most cases a study of those borings will suffice to determine whether or not the underlying soil will support the loads delivered to it, but in doubtful or special cases, especially large foundation areas and important footings the material should be investigated more thoroughly by soil mechanics methods.

A single row of piles shall not be considered as a group provided that they are not spaced closer center to center than 2 1/2 time the nominal diameter or dimension. In those cases where piles are driven in groups into plastic material, the design load shall be determined by the loading of a group of piles or definite allowance shall be made for the difference between the supporting capacity of a single pile and a group of piles. (refer to (G)).

E. Maximum Design Loads for Piles*****

In those cases where it is not feasible to make the required subsurface investigations or test loads, the maximum assumed design load for piles shall be given as in the table below:

Timber piles	20 tons
Cast-In-Place piles	35 tons
Steel Bearing piles	9000 pounds per. sq. ft. of tip (not including reinforcement)

F. Uplift*****

Friction piles may be considered to resist an intermittent but not sustained uplift equivalent to 40 per cent of the above loads providing proper provision is made for the anchorage at top and sufficient skin friction is developed and in no case shall it exceed the weight of material (buoyancy considered) surrounding the portion of the pile embedded in the ground.

Steel bearing piles may be assumed to resist an intermittent but not sustained uplift equivalent to 5 percent of the above load.

G. Group Pile Loading

Where the capacity of a group of friction piles driven into plastic material is not determined by test loading, the following Converse-Labarre formula is suggested to determine the reduction of a single pile load for a group pile load:

$$E = 1 - \phi \frac{(n-1)m + (m-1)n}{90mn}$$

where

- E = the efficiency or the decimal fraction of the single pile value to be used for each pile in the group.
- n = the number of piles in each row.
- m = the number of rows in each group.
- d = the average diameter of the pile.
- s = center to center spacing of piles.
- Tan ϕ = d/s
- ϕ is numerically equal to the angle expressed in degrees.

1.4.5 – PILES

A. General

In general, piling shall be used when footings cannot, at a reasonable expense, be founded on rock or other solid foundation material. At locations where unusual erosion may occur and the soil conditions permit the driving of piles, they, preferably, shall be used as a protection against scour, even though the safe bearing resistance of the natural soil is sufficient to support the structure without piling.

In general, the penetration for any pile shall not be less than 10 feet in hard material and not less than 1/3 the length of the pile nor less than 20 feet in soft material.

For foundation work, no piling shall be used to penetrate a very soft upper stratum overlying a hard stratum unless the piles penetrate the hard material a sufficient distance to rigidly fix the ends.

B. Limitation of Use

Untreated timber piles may be used for temporary construction, revetments, fenders and similar work, and in permanent construction under the following conditions:

1. For foundation piling when the cutoff is below permanent ground water level.
2. For trestle construction when it is economical to do so, though treated piles are preferable.
3. They shall not be used where they will, or may be, exposed to marine borers.

C. Design Loads

The design loads for piles shall be according to Article 1.4.4.

Piles shall be designed to carry the entire superimposed load, no allowance being made for the supporting value of the material between the piles.

The supporting power of piles shall be determined by the application of test loads or by the use of formulas.

D. Spacing, Clearances and Embedment*****

Footing areas shall be so proportioned that pile spacing shall be not less than 2 feet 6 inches center to center for timber piles and 3 feet 0 inches center to center for steel and cast-in-place concrete piles. When the tops of foundation piles are incorporated in a concrete footing, the minimum distance from the center of the pile to the nearest footing edge shall be 1 foot 6 inches but in no case shall the distance from the edge of the pile to the nearest edge of the footing be less than 9 inches.

The tops of cast-in-place concrete piles shall project 6 inches into the footing. The tops of steel and timber piles shall project 12 in. into the footing. Cap plates are not required for steel bearing piles.

Where a reinforced concrete beam is cast-in-place and used as a bent cap supported by piling, concrete cover at the sides of piles shall be a minimum of 6 inches. The piles shall project at least 6 inches, and preferable 9 inches, into the cap; provided, however, concrete piles may project a lesser distance into the cap if the projection of the pile reinforcement is sufficient to provide for adequate bond.

E. Batter Piles*****

When the lateral resistance to the soil surrounding the piles is inadequate to counteract the horizontal forces transmitted to the foundation or when increased rigidity of the entire structure is required, batter piles shall be used in the foundation.

The maximum batter shall be 1 horizontal to 3 vertical.

When the boring data indicates that it takes 6 or more blows per foot of a 300 pound hammer falling 18 inches or its equivalent energy to drive the casing, the amount of lateral resistance allowed per pile shall be 12,000 pounds for wooden piles, 15,000 pounds for cast-in-place piles and 20,000 pounds for steel bearing piles. When the casing is driven with fewer blows, smaller values shall be used. Sufficient batter piles shall be used so that when the horizontal components of the batter piles are added to the lateral resistance of the piles, as given above, the resultant shall not be less than the total lateral or horizontal force acting at the bottom of the footing.

F. Buoyancy

The effect of hydrostatic pressure shall be considered in the design as provided in Article 1.2.18.

G. Concrete Piles (Precast)

Precast concrete piles shall be of approved size and shape. If a square section is employed, the corners shall be chamfered at least one inch. Piles, preferably, shall be cast with a driving point and for hard driving, preferably shall be shod with a metal shoe of approved pattern. Piling may be either of uniform section or tapered. In general, tapered piling shall not be used for trestle construction except for that portion of the pile which lies below the ground line; nor shall tapered piles be used in any location where the piles are to act as columns. In general, concrete piles shall have a cross sectional area, measured above the taper, or not less than 140 square inches and when they are to be used in salt water they shall have a cross sectional area of not less than 220 square inches.

The diameter of tapered piles measured 2 feet from the point shall be not less than 8 inches. In all cases the diameter shall be considered as the least dimension through the center. The point in all cases, where steel points are not used, shall be not less than 6 inches in diameter and the pile shall be beveled, tapered or sloped uniformly from the point to 2 feet from the point.

Vertical reinforcement shall be provided consisting of not less than four bars spaced uniformly around the perimeter of the pile. It shall be at least 1 1/2 per cent of the total cross section measured above the taper, except that if more than four bars are used, the number may be reduced to four in the bottom 4 feet of the pile.

The full length of vertical steel shall be enclosed with spiral reinforcement or equivalent hoops.

The spiral reinforcement at the ends of the pile shall have a pitch of 3 inches, and gage of not less than No. 5 (U.S. Steel Wire Gage). In addition the top 6 inches of pile shall have five turns of spiral winding at one-inch pitch.

For the remainder of the pile the vertical steel shall be enclosed with spiral reinforcement No. 5 gage (U.S. Steel Wire Gage) with not more than 6-inch pitch, or with 1/4-inch round hoops spaced not more than 6-inch centers.

The reinforcement shall be placed at a clear distance from the face of the pile not less than 2 inches and when the piles are for use in salt water or alkali soils this clear distance shall be not less than 3 inches.

In computing stresses due to handling, the computed static loads shall be increased by 50 per cent as an allowance for impact and shock.

H. Concrete Piles (Cast-In-Place)*****

Cast-in-place concrete piles shall be, in general, cast in metal shells which shall remain permanently in place. However, other types of cast-in-place concrete piles, plain or reinforced, cased or uncased, may be used if, in the opinion of the Engineer, the soil conditions permit their use and if their design and the method of placing are satisfactory to him.

Cast-in-place concrete piles may be of either uniform section or tapered, or a combination thereof. The minimum size, measured at the butt, or above the taper, and embedment of reinforcement shall be as specified for precast piles, except that foundation piles may have a minimum butt cross-section area of 100 square inches. The minimum diameter at tip of pile shall be 8 inches.

Cast-in-place piling shall be reinforced when specified or shown on the plans. Cast-in-place foundation piling, carrying axial loads only and where the possibility of lateral forces being applied to the piles is insignificant, need not be reinforced when the soil provides adequate lateral support. Those portions of cast-in-place piling which are not supported laterally shall be designed as reinforced concrete columns in accordance with Article 1.5.9, and the reinforcing steel shall extend ten feet below the plane where the soil provides adequate lateral restraint. Where the shell is more than 0.12 inch in thickness and corrosion is not a factor, it may be considered as reinforcement.

Sufficient reinforcement shall be provided at the junction of the pile with the superstructure to make a suitable connection.

The metal shall be of sufficient thickness and strength so that the shell will hold its original form and show no harmful distortion after it and adjacent shells have been driven and the driving core, if any, has been withdrawn. The design of the shell shall be approved by the Engineer before any driving is done.

Those portions of cast-in-place piling using steel pipe as a shell and which are not supported laterally may be designed as steel columns, neglecting the enclosed concrete and deducting 1/16 inch of thickness from the outside of the pipe in computing the area of steel in the pipe.

I. Steel H-Piles

1. Thickness of Metal

Steel piles shall have a minimum thickness of web of .400 inch. Splice plates shall be not less than 3/8 inch thick.

2. Splices

Piles shall be spliced to develop the net section of pile. The flanges and web shall be either spliced by butt welding or with plates, welded, or bolted. The bolted splices shall only be used on projects where a small number of piling are required and where facilities for welding are not available.

Splices shall be detailed on the contract plans.

3. Caps

In general, caps are not required for steel piles embedded in concrete. Reference is made to Research Report No. 1, "Investigation of the Strength of the Connection between a Concrete Cap and the Embedded end of the Steel H-Pile"—Department of Highways, State of Ohio, for a discussion of this subject and for the results of tests pertinent to it.

4. Scour

If heavy scour is anticipated, consideration shall be given to design of the portion of the pile which would be exposed, as a column.

5. Lugs and Scabs*****

These devices may be used to increase the bearing power of the pile where necessary. They may consist of structural shapes, welded or bolted, of plates welded between the flanges.

6. Reinforcement*****

The tips of steel bearing piles shall be reinforced.

J. Unfilled Tubular Steel Pipes

1. Thickness of Metal

Piles shall have minimum wall thickness not less than indicated in the following table:

Outside Diameter	Less Than 14 inches	14 inches add over
	.25 inch	.375 inch

2. Splices

Piles shall be spliced to develop the full section of the pile. The piles shall be spliced either by butt welding or by the use of welded sleeves. Splices shall be detailed on the contract plans.

3. Driving

Tubular steel piles may be either closed or open ended. Closure plates should not extend beyond the perimeter of the pile.

4. Column Action

Where the piles are to be used as part of a bent structure or where heavy scour is anticipated that would expose a portion of the pile, the pile shall be investigated for column action.

The provisions of Article 1.4.5(K) shall apply to unfilled tubular steel piles.

K. Steel Pile and Steel Pile Shell Protection

Where conditions of exposure warrant, concrete encasement shall be used on steel piles and steel shells or 1/16 inch of thickness shall be deducted from all exposed surfaces in computing the area of steel in the piles or shells.

1.4.6 – FOOTINGS

A. Depth*****

The depths of footings shall be determined with respect to the character of the foundation materials and the possibility of undermining. Except where solid rock is encountered or in other special cases, the footings of all structures, other than culverts, which are exposed to the erosive action of stream currents, preferably, shall be founded at a depth of not less than 4 feet below the permanent bed of the stream. Stream piers and arch abutments, preferably, shall be founded at a depth of not less than 6 feet below stream bed, unless supported on piles. The above preferred minimum depths shall be increased as conditions may require.

Footings not exposed to the action of stream currents shall be founded on a firm foundation and below frost.

Footings for culverts shall be carried to an elevation sufficient to secure a firm foundation, or a heavy reinforced floor shall be used to distribute the pressure over the entire horizontal area of the structure. In any location liable to erosion, aprons or cut-off walls shall be used at both ends of the culvert and, where necessary, the entire floor area between the wing walls shall be paved. Baffle walls or struts across the unpaved bottom of a culvert barrel shall not be used where the stream bed is subject to erosion. When conditions require, culvert footings shall be reinforced longitudinally.

B. Anchorage*****

Footings on inclined smooth solid rock surfaces which are not restrained by an overburden of resistant material, shall be effectively anchored by means of rock bolts, dowels, keys or steps.

C. Distribution of Pressure

All footings shall be designed to keep the maximum soil pressures within safe bearing values. In order to prevent unequal settlement, footings shall be designed to keep the pressure as nearly uniform as practicable. In footings having unequal pressures and requiring piling, the spacing of the piles shall be such as to secure as nearly equal loads on each pile as may be practicable.

D. Spread Footings

Spread footings which act as cantilevers may be decreased in thickness from the junction of the footing slab with column or wall toward the edge of the footing, provided sufficient section is maintained at all points to provide the necessary resistance to diagonal tension and bending stresses. This decrease in section may be accomplished by sloping the upper surface of the footing or by means of vertical steps. Stepped footings shall be cast monolithically.

E. Internal Stresses in Spread Footings*****

Spread footings shall be considered as under the action of downward forces, due to the superimposed loads, resisted by an upward pressure exerted by the foundation materials and distributed over the area of the footings as determined by the eccentricity of the resultant of the downward forces. Where piles are used under footings, the upward reaction of the foundation shall be considered as a series of concentrated loads applied at the pile centers, each pile being assumed to carry its computed proportion of the total footing load.

When a single spread footing supports a column, pier or wall, this footing shall be assumed to act as a cantilever. When two or more piers or columns are placed upon a common footing, the footing slab shall be designed for the actual conditions of continuity and restraint.

The minimum thickness of spread footings shall be 2 feet or 1 foot 6 inches above the tops of the piles if piles are used.

Footings shall be designed for the bending stress, diagonal tension stress and bond at the critical section designated herein.

The critical section for bending shall be taken at the face of the column, pedestal or wall. In the case of columns other than square or rectangular, the critical section shall be taken at the side of the concentric square of equivalent area. For footings under masonry walls, where bond between the wall and footing is reduced to friction value, the critical section shall be taken as midway between the middle and the face of the wall. For footings under metallic column bases, the critical section shall be taken as midway between the face of the column and the edge of the metallic base. The load shall be considered as uniformly distributed over the column, pedestal or wall, or metallic column base.

The critical section for bond shall be taken at the same plane as for bending, and the shear used for computing bond shall be based on the same loading and section as for bending. Bond should also be investigated at planes where changes of section or of reinforcement occur.

The critical section for diagonal tension in footings on soil or rock shall be considered as the concentric vertical section through the footing at a distance "D" from each face of the column, pedestal, or wall; "D" being equal to the depth from the top of the section to the centroid of the longitudinal tension reinforcement.

The critical section for diagonal tension in footings supported on piles shall be considered as the concentric vertical section through the footing at a distance, D/2 from each face of the column, pedestal or wall, and any piles whose centers are at or outside this section shall be considered in computing the diagonal tension.

In sloped or stepped footings, stresses should be investigated at sections where the depth changes outside the critical section as defined above.

Bending need not be considered unless the projection of the footing is more than two-thirds of the depth.

F. Reinforcement*****

Footing slabs shall be reinforced for bending stresses and, where necessary, for diagonal tension. The computed stress in the bar shall be developed in bond.

The reinforcement for square footings shall consist of two or more bands of bars. The reinforcement necessary to resist the bending moment in each direction in the footing shall be determined as for a reinforced concrete beam; the effective depth of the footing shall be the depth from the top to the plane of the reinforcement. With piles the lower layer of bars in the footing shall be placed 2 inches above the piles. In cases where design requirements dictate, and if the pile pattern will permit, the bars may be located between piles. In this case, 3 inches of cover will be maintained. The required reinforcement shall be spaced uniformly across the footing, unless the footing width is greater than the side of the column or pedestal plus twice the effective depth of the footing, in which case the width over which the reinforcement is spread may equal the width of the column or pedestal plus twice the effective depth of the footing plus one-half the remaining width of the footing. In order that no considerable area of the footing shall remain unreinforced, additional bars shall be placed outside of the width specified, but such bars shall not be considered as effective in resisting the calculated bending moment. For the extra bars a spacing double that used for the reinforcement within the effective belt may be used.

G. Transfer of Stress from Vertical Reinforcement

The stresses in the vertical reinforcement of columns or walls shall be transferred to the footings by extending the reinforcement into them a sufficient distance to develop the strength of the bars in bond, or by means of dowels anchored in the footings and overlapping or fastened to the vertical bars in such manner as to develop their strength. If the dimensions of the footings are not sufficient to permit the use of straight bars, the bars may be hooked or otherwise mechanically anchored in the footings.

1.4.7 – ABUTMENTS

A. General

Abutments and their foundations shall be designed to support the following vertical loads:

1. Weight of Abutment
2. Reaction from Superstructure
3. Live load over any portion of superstructure or approach
4. Weight of Earth backfill

In addition, they shall be designed to withstand lateral forces from the following:

1. Earth Backfill
2. Live load on approach
3. Wind forces
4. Longitudinal forces from live load that may be transmitted by bearings.

The design shall be investigated for any combination of these forces which may produce the most severe conditions.

High abutments shall also be investigated for a construction condition where the superstructure is erected before the fill is placed in back of abutment and the construction condition where fill is placed in back of abutment before the superstructure is erected. Since these are temporary stresses use 150% of the design stresses. Passive earth pressure in front of walls shall not be used to counteract overturning or sliding except below the footing.

Abutments shall be designed to be safe against overturning about the toe of the footing, against sliding on the footing base and against crushing of foundation material or overloading of piles at the point of maximum pressure.

In computing stresses in abutments, the weight of filling material directly over an inclined or stepped rear face, or over a reinforced concrete spread footing extending back from the face wall, may be considered as part of the effective weight of the abutment. In the case of a spread footing, the rear projection shall be designed as a cantilever supported at the abutment stem and loaded with the full weight of the superimposed material, unless a more exact method is used.

The cross section of stone masonry or plain concrete abutments shall be proportioned to avoid the introduction of tensile stress in the material.

B. Reinforcement for Temperature

Faces of the abutments, except footings, not otherwise reinforced, shall be reinforced with a minimum of No. 5 bars at one foot horizontally and No. 5 bars at 2 feet vertically to resist the formation of temperature and shrinkage cracks.

C. Wing Walls

Wing walls shall be of sufficient length to retain the roadway embankment to the required extent and to furnish protection against erosion. For ordinary materials, in the absence of accurate data, the slope of the fill shall be assumed as two horizontal to one vertical and wing lengths computed on this basis.

Where expansion or contraction joints are not used reinforcement rods shall be spaced across the junction between all wing walls and abutments to thoroughly tie them together. Such bars shall be an extension of the face steel from one pour far enough through the joint to lap with the face steel in the other pour.

Keys between wingwalls and abutments shall be proportioned for the probable shear at this junction. If necessary, the wall thickness shall be increased to provide the necessary dimensions.

D. Drainage*****

The filling material behind abutments shall be effectively drained and weep holes shall be placed at a maximum spacing of 30 ft. with a minimum of two weep holes per abutment.

E. Approach Slabs*****

Approach slabs shall always be used. Where the highway is cement concrete, pressure relief type joints shall be used. Where the highway is asphaltic concrete, class A concrete for structures with a finish the same as the bridge slab shall be used.

1.4.8 – RETAINING WALLS

A. General*****

Retaining walls shall be designed to withstand earth pressure, including any live load surcharge, and the weight of the wall, in accordance with the general principles specified above for abutments.

B. Base or Footing Slabs

The rear projection or heel of base slabs shall be designed to support the entire weight of the superimposed materials, unless a more exact method is used.

The base slabs of cantilever walls shall be designed as cantilevers supported by the wall.

C. Vertical Walls

The vertical stems of cantilever walls shall be designed as cantilevers supported at the base.

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*****

Faces of abutments, except footings, not otherwise reinforced, shall be reinforced with a minimum of No. 5 bars at one foot horizontally and No. 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 90 foot intervals in all retaining walls and wingwalls more than 180 feet 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 a 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 theron; 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-Sold Piers*****

Exposed faces of abutments and walls, not otherwise reinforced, shall be reinforced with a minimum of No. 5 bars at 1 foot horizontally and No. 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.

Section 5 – REINFORCED CONCRETE DESIGN

1.5.1 – APPLICATION

The specifications of this section are intended for design of reinforced (nonprestressed) concrete bridge members and structures. Bridge members designed as prestressed concrete shall conform to Section 6.

1.5.2 – NOTATIONS AND DEFINITIONS

A. Notations

a = depth of equivalent rectangular stress block, defined in Article 1.5.31(A)(6).

a_b = depth of equivalent rectangular stress block for balanced conditions, in. See Article 1.5.33(B)(3).

A = effective tension area of concrete surrounding the main tension reinforcing bars and having the same centroid as that reinforcement, divided by the number of bars, sq. in. When the main reinforcement consists of several bar sizes the number of bars shall be computed as the total steel area of the largest bar used–Article 1.5.39.

A_b = area of an individual bar, sq. in.–Article 1.5.14.

A_c = area of core of spirally reinforced compression member measured to the outside diameter of the spiral. sq.in. Article 1.5.11(3).

A_g = gross area of section, sq. in.

A_s = area of tension reinforcement, sq. in.

A_s' = area of compression reinforcement, sq. in.

A_{sf} = area of reinforcement to develop compressive strength of overhanging flanges of I- and T- sections– Article 1.5.32(C).

A_{st} = total area of longitudinal reinforcement–Article 1.5.33(B)(1).

A_v = area of shear reinforcement within a distance s.

A_{vf} = area of shear-friction reinforcement, sq. in– Articles 1.5.29(D) and 1.5.35(D).

A_w = area of a deformed wire, sq. in.–Article 1.5.19(B).

A_1 = loaded area–Articles 1.5.26 and 1.5.36.

A_2 = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area–Articles 1.5.26 and 1.5.36.

b = width of compression face of member.

b_o = periphery of critical section for slabs and footings– Articles 1.5.29(F) and 1.5.35(F).

b_v = the width of the cross section being investigated for shear–Articles 1.5.29(E) and 1.5.35(E).

b_w = web width, or diameter of circular section. For tapered webs, the average width or 1.2 times the minimum width, whichever is smaller, in.–Articles 1.5.29(A) and 1.5.35(A).

c = distance from extreme compression fiber to neutral axis– Article 1.5.31(A)(6).

C_m = a factor relating the actual moment diagram to an equivalent uniform moment diagram–Article 1.5.34(B).

d = distance from extreme compression fiber to centroid of tension reinforcement, in.

d = distance from extreme compression fiber to centroid of compression reinforcement, in.

d = distance from centroid of gross section, neglecting the reinforcement, to centroid of tension reinforcement, in.

d_b = nominal diameter of bar or wire, in.

d_c = thickness of concrete cover measured from the extreme tension fiber to the center of the bar located closest thereto–Article 1.5.39.

e = eccentricity of design load parallel to axis measured from the centroid of the section. It may be calculated by conventional methods of frame analysis–Article 1.5.33(A)(2).

e_b = eccentricity of the balanced condition load-moment relationship. See Article 1.5.33(B)(3).

= M_b/P_b.

E_c = modulus of elasticity of concrete, psi. See Article 1.5.23(D).

EI = flexural stiffness of compression members–Article 1.5.34(B).

E_s = modulus of elasticity of steel, psi. See Article 1.5.23(D).

f_b = average bearing stress in concrete on loaded area–Articles 1.5.26(A) and 1.5.36.

f_c = extreme fiber compressive stress in concrete at service loads–Article 1.5.26(A).

f_c' = specified compressive strength of concrete, psi.

$\sqrt{} f_c'^{1/2}$ = square root of specified compressive strength of concrete, psi.

f_{ct} = average splitting tensile strength of lightweight aggregate concrete, psi.

f_h = tensile stress developed by a standard hook, psi–Article 1.5.17.

f_r = modulus of rupture of concrete, psi–Article 1.5.26(A).

f_s = tensile stress in reinforcement at service loads, psi.– Article 1.5.26(B).

f_s' = stress in compression reinforcement at balanced conditions. See Articles 1.5.32(D) and 1.5.33(B)(3).

f_t = extreme fiber tensile stress in concrete at service loads–Article 1.5.26(A).

f_y = specified yield strength of reinforcement, psi.

h = overall thickness of member, in.

h_f = compression flange thickness of I- and T- sections.

I_{cr} = moment of inertia of cracked section transformed to concrete–Article 1.5.23(G).

I_e = effective moment of inertia for computation of deflection— Article 1.5.23(G).

I_g = moment of inertia of gross concrete section about the centroidal axis, neglecting the reinforcement.

I_s = moment of inertia of rèinforcement about the centroidal axis of the member cross section.

k = effective length factor for compression members—Article 1.5.34(B)(3).

l_a = additional embedment length at support or at point of inflection, in.— Article 1.5.13(B)(3).

l_d = development length, in. See DEVELOPMENT AND SPLICES OF REINFORCEMENT.

l_c = equivalent embedment length, in. See DEVELOPMENT AND SPLICES OF REINFORCEMENT'

l_u = unsupported length of compression member—Article 1.5.34.

l_w = total lengths of wire extending beyond outermost cross wires, for each pair of spliced wires, in.—Articles 1.5.22(E).

M = computed moment capacity as defince in Article 1.5.13(B).

M_a = maximum moment in member at stage for which deflection is being computed—Article 1.5.23(G).

M_b = design moment strength of a section at simultaneous assumed ultimate strain of concrete and yielding of tension reinforcement (balanced conditions)—Article 1.5.33(B)(3).

M_c = moment to be used for design of compression member—Article 1.5.34(B).

M_{cr} = cracking moment. See Article 1.5.23(G).

M_t = theoretical moment strength of a section.

M_u = design moment strength of a section, M_u = o$M_t \geqslant$ applied design moment.

M_{ux} = design moment strength of a section in the direction of the x axis—Article 1.5.33(C).

M_{uy} = design moment strength of a section in the direction of the y axis—Article 1.5.33(C).

M_x = applied design moment component in the direction of the x axis—Article 1.5.33(C).

M_y = applied design moment component in the direction of the y axis—Article 1.5.33(C).

M_1 = value of smaller design end moment on compression member calculated from a conventional elastic analysis, positive if member is bent in single curvature, negative if bent in double curvature—Article 1.5.34(B).

M_2 = value of larger design end moment on compression member calculated from a conventional elastic analysis, always positive—Article 1.5.34(B).

n = number of pairs of cross wires in splice—Article 1.5.22(E).

n = modular ratio = E_s/E_c—Article 1.5.27.

n_w = number of cross wires in anchorage zone of welded deformed wire fabric—Article 1.5.19(B).

$N(N_u)$ = axial design load normal to the cross section occurring simultaneously with $V(V_u)$ to be taken as positive for compression, negative for tension, and to include the effects of tension due to shrinkage and creep—Articles 1.5.29(B). amd 1.5.35(B).

P_b = axial design load strength of a section at simultaneous assumed ultimate strain of concrete and yielding of tension reinforcement (balanced conditions)—Article 1.5.33(B)(3).

P_c = critical load. See Article 1.5.34(B).

P_o = axial design load strength of a section in pure compression—Article 1.5.33(B)(1).

P_t = theoretical axial load strength.

P_u = axial design load strength of a section under combined flexure and axial load, $P_u = oP_t \geqslant$ applied axial design load.

P_{ux} = axial design load strength corresponding to M_{ux} with bending considered in the direction of the x axis only — 1.5.33(C).

P_{uy} = axial design load strength corresponding to M_{uy} with bending considered in the direction of the y axis only—Article 1.5.33(C).

P_{uxy} = axial design load strength with biaxial loading— Article 1.5.33(C).

r = radius of gyration of the cross section of a compression member—Article 1.5.34(B)(2).

s = tie spacing, in.—Article 1.5.22(C)(1).

s = shear reinforcement spacing in a direction parallel to the longitudinal reinforcement, in.

s_w = spacing of deformed wires, in.—Article 1.5.19(B).

S = span length as defined in Article 1.5.23(F).ft.

v_c = permissible shear stress carried by concrete—Articles 1.5.29(B) and 1.5.35(B).

v_{dh} = design horizontal shear stress at any cross section — Article 1.5.29(E) and 1.5.35(E).

v_h = permissible horizontal shear stress—Articles 1.5.29(E) and 1.5.35(E).

$v(v_u)$ = total applied design shear stress at section—Articles 1.5.29(A) and 1.5.35(A).

$V(V_u)$ = total applied design shear stress at section— Articles 1.5.29(A) and 1.5.35(A).

w = weight of concrete, lb. per cu. ft.

y_t = distance from the centroidal axis of gross section, neglecting the reinforcement, to the extreme fiber in tension—Article 1.5.23(G).

z = a quantity limiting distribution of flexural reinforcement— 1.5.39.

α(alpha) = angle between inclined shear reinforcement and longitudinal axis of member.

βb(beta) = ratio of area of bars cut off to total area of bars at the section. See Article 1.5.13(A)(6).

βd = the ratio of maximum design dead load moment to maximum design total load moment, always positive—Article 1.5.34(B).

β_1 = a factor defined in Article 1.5.31(A)(6).

δ(delta) = moment magnification factor for compression members. See Article 1.5.34(B).

μ(mu) = coefficient of friction. See Articles 1.5.29(D) and 1.5.35(D).

ξ(xi) = constant for standard hook—Article 1.5.17.

ρ(rho) = tension reinforcement ration = A_s/bd.

= compression reinforcement ration = A_s'/bd.

ρb = reinforcement ration producing balanced conditions. See Article 1.5.32(A).

ρmin = minimum tension reinforcement ratio. See Article 1.5.7.

ρs = ratio of volume of spiral reinforcement to total volume of core (out-to-out of spirals) of a spirally reinforced compression member. Article 1.5.11(B).

ρw = reinforcement ratio used in Eq. (5-2) and Eq. (6-21).

= A_s/b_wd.

ϕ(phi) = capacity reduction factor. See Article 1.5.30(A).

B. Definitions

The following terms are defined for general use in Section 5. Specialized definitions appear in individual Articles.

Compressive strength of concrete (f_c')—Specified compressive strength of concrete in pounds per square inch (psi). Wherever this quantity is under a radical sign, the square root of the numerical value only is intended, and the resultant is in pounds per square inch (psi).

Deformed reinforcement—Deformed reinforcing bars, deformed wire, welded wire fabric, and welded deformed wire fabric.

Design load—All applicable loads and forces or their related internal moments and forces used to proportion members. For design by SERVICE LOAD DESIGN, design load refers to loads without load factor—LOAD FACTOR DESIGN, design load refers to loads multiplied by appropriate load factors.

Development length—The length of embedded reinforcement required to develop the design strength of the reinforcement at a critical section.

Effective area of reinforcement—The area obtained by multiplying the right cross-sectional area of the reinforcement by the cosine of the angle between its direction and the direction for which the effectiveness is to be determined.

Embedment length—The length of embedded reinforcement provided beyond a critical section.

Embedment length, equivalent (l_e)—The length of embedded reinforcement which can develop the same stress as that which can be developed by a hook or mechanical anchorage.

End anchorage–Length of reinforcement, or a mechanical anchor, or a hook, or combination thereof, beyond the point of nominal zero stress in the reinforcement.

Plain reinforcement–Reinforcement that does not conform to the definition of deformed reinforcement.

Service Load–Loads without load factors.

Spiral–Continuously wound reinforcement in the form of a cylindrical helix.

Splitting tensile strength (f_{ct})–The tensile strength of concrete determined by splitting test made in accordance with "Specifications for Lightweight Aggregates for Structural Concrete" AASHTO M 195 (ASTM C330).

Stirrups or ties–Lateral reinforcement formed of individual units, open or closed, or of continuously wound reinforcement. The term "stirrups" is usually applied to lateral reinforcement in horizontal members and the term "ties" to those in vertical members.

Yield strength or yield points (f_y)–Specified minimum yield strength or yield point of reinforcement in pounds per square inch.

Concrete, structural lightweight–A concrete containing lightweight aggregate having an air-dry unit weight as determined by "Method of Test for Unit Weight of Structural Lightweight Concrete" (ASTM C567), not exceeding 115 pcf. In this specification, a lightweight concrete without natural sand is termed "all-lightweight concrete" and lightweight concrete in which all fine aggregate consists of normal weight sand is termed "sand-lightweight concrete."

1.5.3 – MATERIALS

A. Concrete

1. The compressive strength of the concrete "f'_c", for which each part of the structure is designed, shall be shown on the plans.
2. The specified compressive strength of the concrete, "f'_c" shall be the basis for acceptance. The requirements for "f'_c" shall be based on tests of cylinders made and tested in accordance with the methods as prescribed in Division II, Section 4.

B. Reinforcement

1. The yield strength or grade of reinforcement used in design shall be shown on the plans.
2. Reinforcement to be welded shall be indicated on the plans and the welding procedure to be used shall be specified.
3. Designs shall not be based on a yield strength, "f_y", in excess of 60,000 psi.
4. Only deformed reinforcement as defined in Article 1.5.2(B) shall be used except that plain bars or plain wire may be used as spirals and ties.
5. Reinforcement shall conform to the specifications listed in Division II, Section 5, except that, for reinforcing bars, the yield strength shall correspond to that determined by tests on full sized bars.

1.5.4 – HOOKS AND BENDS

A. Hooks

The term "standard hooks" as used herein shall mean either:

1. A 180-deg. turn plus an extension of at least four bar diameters but not less than 2½ in. at the free end of the bar, or
2. A 90-deg. turn plus an extension of at least 12 bar diameters at the free end of the bar, or
3. For stirrup and tie anchorage only, either a 90-deg. or a 135-deg. turn plus an extension of at least six bar diameters but not less than 2½ in. at the free end of the bar.

B. Minimum Bend Diameter

1. The diameter of bend measured on the inside of the bar, other than for stirrups and ties, shall not be less than the values of Table 1.5.4 except that for Grade 40 bars in sizes No. 3 to No. 11, inclusive, with turns not exceeding 180-deg., the minimum diameter shall be five bar diameters.
2. Inside diameter of bend for stirrups and ties shall not be less than four bar diameters for sizes No. 5 and smaller, and five bar diameters for sizes No. 6 to No. 8 inclusive.
3. Inside diameter of bend in welded wire fabric, plain or deformed, for stirrups and ties shall not be less than four wire diameters for deformed wire larger than D6 and two wire diameters for all other wires. Bends with inside diameter of less than eight wire diameters shall not be less than four wire diameters from the nearest welded intersection.

TABLE 1.5.4–Minimum Diameters of Bend

Bar Size	Minimum Diameter
No. 3 through No. 8	6 bar diameters
No. 9, No. 10, and No. 11	8 bar diameters
No. 14 and No. 18	10 bar diameters

1.5.5 – SPACING OF REINFORCEMENT

A. For cast-in-place concrete the clear distance between parallel bars in a layer shall not be less than one and one-half times the nominal diameter of the bars, one and one-half times the maximum size of the coarse aggregate, nor 1½ in.

B: For precast concrete (manufactured under plant control conditions) the clear distance between parallel bars in a layer shall be not less than the nominal diameter of the bars, one and one-third times the maximum size of the coarse aggregate, nor 1 in.

C. Where positive or negative reinforcement is placed in two or more layers, the bars in the upper layers shall be placed directly above those in the bottom layer with the clear distance between layers not less than 1 in.
D. The clear distance limitation between bars shall also apply to the clear distance between a contact lap splice and adjacent splices or bars.
E. Groups of parallel reinforcing bars bundled in contact to act as a unit shall be limited to four in any one bundle. Bars larger than No. 11 shall be limited to two in any one bundle in beams. Bundled bars shall be located within stirrups or ties. Individual bars in a bundle cut off within the span of a member shall terminate at different points with at least 40 bar diameters stagger. Where spacing limitations are based on bar size, a unit of bundled bars shall be treated as a single bar of a diameter derived from the equivalent total area.
F. In walls and slabs the principal reinforcement shall be spaced not farther apart than one and one-half times the wall or slab thickness, nor more than 18 in.

1.5.6 – CONCRETE PROTECTION FOR REINFORCEMENT

A. The following minimum concrete cover shall be provided for reinforcement:

	Minimum Cover, in.
Concrete cast against and permanently exposed to earth	3
Concrete exposed to earth or weather	
Principal reinforcement	2
Stirrups, ties and spirals	1½
Concrete bridge slabs	
Top reinforcement	2
Bottom reinforcement	1
Concrete not exposed to weather or in contact with the ground	
Principal reinforcement	1½
Stirrups, ties and spirals	1

B. For bar bundles, minimum concrete cover shall be equal to the lesser of the equivalent diameter of the bundle or 2 in., but not less than that given in paragraph A above.
C. In corrosive or marine environments or other severe exposure conditions, the amount of concrete protection shall be suitably increased, and the denseness and nonporosity of the protecting concrete shall be considered, or other protection shall be provided.
D. Exposed reinforcing bars, inserts, and plates intended for bonding with future extensions shall be protected from corrosion.

1.5.7 – MINIMUM REINFORCEMENT OF FLEXURAL MEMBERS

A. At any section of a flexural member, except walls and slabs, where reinforcement is required by analysis, the ratio "ρ" provided shall be not less than that given by

$$\rho\ min = \frac{\sqrt{f'_c}}{.57\, f_y \left(1 - \frac{cover}{d}\right)} \qquad (2\text{-}1)$$

In T-girders, where the web is in tension, the ratio "ρ" shall be computed for this purpose using the width of the web.

B. Alternately, a minimum reinforcement ratio, "ρ min", may be computed such that the reinforcement provided shall be adequate to develop a moment capacity at least 1.2 times the cracking moment calculated on the basis of the modulus of rupture specified in Article 1.5.25(A)(1).

C. The requirements of paragraph A or B above may be waived if the area of reinforcement provided at a section, is at least one-third greater than that required by analysis.

D. In walls and slabs, the principal reinforcement in the direction of the span shall provide a ratio of reinforcement area to gross concrete area at least equal to 0.002.

1.5.8 – DISTRIBUTION OF REINFORCEMENT IN FLEXURAL MEMBERS

A. Principal tension reinforcement shall be well distributed in the zones of maximum tension. Where girder flanges are in tension, the tension reinforcement shall be distributed over an effective slab width as defined in Article 1.5.23(J)(2) or 1.5.23(K)(2), or a width equal to one-tenth of the girder span, whichever is smaller. If the effective slab width exceeds one-tenth of the span, additional longitudinal reinforcement shall be provided in the outer portions of the slab. For LOAD FACTOR DESIGN, the additional requirements of Article 1.5.39 shall also apply.

B. If the depth of the side face of a member exceeds 2 ft. longitudinal reinforcement having a total area at least equal to 10 percent of the principal tension reinforcement shall be placed near the side faces of the member and distributed in the zone of flexural tension. The spacing of such reinforcement shall not exceed 12 in. or the width of the web, whichever is less. Such reinforcement may be included in computing the flexural capacity only if a stress and strain compatibility analysis is made to determine stresses in the individual bars.

1.5.9 – LATERAL TIE IN BEAMS

Compression reinforcement in beams shall be enclosed by ties or stirrups, at least No. 3 in size for longitudinal bars No. 10 or smaller, and at least No. 4 in size for No. 11, No. 14, No. 18 and bundled longitudinal bars, or by welded wire fabric of equivalent area. The spacing of the ties shall not exceed 16 longitudinal bar diameters. Such stirrups or ties shall be used throughout the distance where the compression reinforcement is required.

1.5.10 – SHEAR REINFORCEMENT–GENERAL REQUIREMENTS

A. Minimum Shear Reinforcement

1. A minimum area of shear reinforcement shall be provided in all flexural members, except slabs and footings, where the design shear stress is greater than one-half the permissible shear stress, "v_c" carried by the concrete.
2. Where shear reinforcement is required by paragraph 1, or by analysis, the area provided shall be not less than

$$A_v = \frac{50\, b_w s}{f_y} \tag{2-2}$$

where "b_w" and "s" are in inches.
3. Minimum shear reinforcement requirements may be waived if it is shown by test that the required ultimate flexural and shear capacity can be developed when shear reinforcement is omitted.

B. Types of Shear Reinforcement

1. Shear reinforcement may consist of:

(a) Stirrups perpendicular to the axis of the member or making an angle of 45-deg. or more with the longitudinal tension reinforcement.
(b) Welded wire fabric with wires located perpendicular to the axis of the member.
(c) Longitudinal bars with a bent portion making an angle of 30-deg. or more with the longitudinal tension bars.
(d) Combinations of stirrups and bent bars.
(e) Spirals.

2. Shear reinforcement shall be anchored at both ends in accordance with the requirements of Article 1.5.21.

C. Spacing of Shear Reinforcement

Where shear reinforcement is required and is placed perpendicular to the axis of the member, it shall be spaced not further apart than 0.50d, but not more than 24 in. Inclined stirrups and bent bars shall be so spaced that every 45-deg. line, extending toward the reaction from the middepth of the member, 0.50d, to the longitudinal tension bars, shall be crossed by at least one line of shear reinforcement.

1.5.11 – LIMITS FOR REINFORCEMENT OF COMPRESSION MEMBERS

A. Longitudinal Reinforcement

1. Longitudinal reinforcement for compression members shall be not less than 0.01 nor more than 0.08 times the gross area, "A_g", of the section. The minimum number of longitudinal reinforcing bars shall be six for bars in a circular arrangement and four for bars in a rectangular arrangement. The minimum size of bar shall be No. 5.

2. In a compression member which has a larger cross section than required by considerations of loading, a reduced effective area, A_g, may be used for determining minimum longitudinal reinforcement, provided that in no case shall less longitudinal reinforcement be used than that required by the minimum section designed with one percent of longitudinal reinforcement.

B. Lateral Reinforcement

1. Spiral reinforcement for compression members shall consist of evenly spaced continuous spirals held firmly in place by attachment to the longitudinal reinforcement and true to line by vertical spacers. The spirals shall be of such size as to permit handling and placing without being distorted from the designed dimensions. Spiral reinforcement may be hot rolled or cold drawn material, plain or deformed, with a minimum diameter of 3/8 in. Anchorage or spiral reinforcement shall be provided by one and one-half extra turns of spiral bar or wire at each end of the spiral unit. Splices when necessary in spiral bars or wires shall be tension lap splices of 48 diameter minimum but not less than 12 in., or welds. The clear spacing between spirals shall not exceed 3 in. nor be less than 1 in. or 1 1/3 times the maximum size of coarse aggregate used. The reinforcing spiral shall extend from the footing or other support to the level of the lowest horizontal reinforcement in the members supported above.

The ratio of spiral reinforcement, "ρs", shall not be less than the value given by;

$$\rho s = 0.45 \left(\frac{A_g}{A_c} - 1 \right) \frac{f'c}{fy} \qquad (2\text{-}3)$$

where f_y is the specified yield strength of spiral reinforcement but not more than 60,000 psi.

2. All bars for tied compression members shall be enclosed by lateral ties, at least No. 3 in size for longitudinal bars No. 10 or smaller, and at least No. 4 in size for No. 11, No. 14, No. 18 and bundled longitudinal bars. The spacing of the ties shall not exceed the least dimension of the member or 12 inches, except that this spacing may be increased in compression members which have a larger cross section than required by conditions of loading. When bars larger than No. 10 are bundled more than two in any one bundle, the tie spacing shall be reduced to one-half that specified above. The ties shall be so arranged that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie having an included angle of not more than 135-deg. and no bar shall be farther than 6 in. clear on either side from such a laterally supported bar. Ties shall be located vertically not more than half a tie spacing above the footing or other support and shall be spaced as provided herein to not more than half a tie spacing below the lowest horizontal reinforcement in the members supported above. Welded wire fabric of equivalent area may be used. Where the bars are located around the periphery of a circle, a complete circular tie may be used.

3. Lateral reinforcement requirements for compression members may be waived where tests or structural analysis show adequate strength and feasibility of construction.

1.5.12 – SHRINKAGE AND TEMPERATURE REINFORCEMENT

Reinforcement for shrinkage and temperature stresses shall be provided near exposed surfaces of walls and slabs not otherwise reinforced. The total area of reinforcement provided shall be at least 1/8 square inch per foot and be spaced not farther apart than three times the wall or slab thickness nor 18 in.

DEVELOPMENT AND SPLICES OF REINFORCEMENT

1.5.13 – DEVELOPMENT REQUIREMENTS

A. General

1. The calculated tension or compression in the reinforcement at each section shall be developed on each side of that section by embedment length or end anchorage or a combination thereof. For bars in tension, hooks may be used in developing the bars.
2. Tension reinforcement may be anchored by bending it across the web and making it continuous with the reinforcement on the opposite face of the member, or anchoring it there.
3. The critical sections for development of reinforcement in flexural members are at points of maximum stress and at points within the span where adjacent reinforcement terminates, or is bent. The provisions of Article 1.5.13(B)(3) must also be satisfied.
4. Reinforcement shall extend beyond the point at which it is no longer required to resist flexure for a distance equal to the effective depth of the member, 15 bar diameters, or 1/20 of the clear span whichever is greater, except at supports of simple spans and at the free end of cantilevers.
5. Continuing reinforcement shall have an embedment length not less than the development length "l_d" beyond the point where bent or terminated tension reinforcement is no longer required to resist flexure.
6. Flexural reinforcement located within the width of a member used to compute the shear strength shall not be terminated in a tension zone unless one of the following conditions is satisfied:

   a. The shear at the cutoff point does not exceed two-thirds that permitted, including the shear strength of furnished shear reinforcement.
   b. Stirrup area in excess of that required for shear is provided along each terminated bar over a distance from the termination point equal to three-fourths the effective depth of the member. The excess stirrups shall be proportioned such that their $(A_v/b_ws)f_y$ is not less than 60 psi. The resulting spacings shall not exceed $d/8\beta_b$ where β_b is the ratio of the area of bars cut off to the total area of bars at the section.
   c. For #11 and smaller bars, the continuing bars provide double the area required for flexure at the cutoff point and the shear does not exceed three-fourths that permitted.

B. Positive Moment Reinforcement

1. At least one-third the positive moment reinforcement in simple members and one-fourth the positive moment reinforcement in continuous members shall extend along the same face of the member into the support. In beams, such reinforcement shall extend into the support at least 6 inches.
2. When a flexural member is part of the lateral load resisting system, the positive reinforcement required to be extended into the support by paragraph (1) above shall be anchored to develop the full "f_y" in tension at the face of the support.

3. At simple supports and at points of inflection, positive moment tension reinforcement shall be limited to a diameter such that "l_d" computed for "f_y" by Article 1.5.14 does not exceed

$$\frac{M}{V} + l_a$$

where "M" is the computed moment capacity assuming all positive moment tension reinforcement at the section to be fully stressed. "V" is the maximum applied design shear at the section, "l_a" at a support shall be the sum of the embedment length beyond the center of the support and the equivalent embedment length of any furnished hook or mechanical anchorage, "l_a" at a point of inflection shall be limited to the effective depth of the member of "$12d_b$", whichever is greater. The value "M/V" in the development length limitation may be increased 30 percent when the ends of the reinforcement are confined by a compressive reaction.

C. Negative Moment Reinforcement

1. Tension reinforcement in a continuous, restrained, or cantilever member, or in any member of a rigid frame, shall be anchored in or through the supporting member by embedment length, hooks, or mechanical anchorage.
2. Negative moment reinforcement shall have an embedment length into the span as required by Articles 1.5.13(A)(1) and 1.5.13(A)(4).
3. At least one-third the total reinforcement provided for negative moment at the support shall have an embedment length beyond the point of inflection not less than the effective depth of the member, 12 bar diameters, or one-sixteenth of the clear span, whichever is greater.

D. Special Members

Adequate end anchorage shall be provided for tension reinforcement in flexural members where reinforcement stress is not directly proportional to moment, such as: sloped, stepped, or tapered footings; brackets; deep beams; or members in which the tension reinforcement is not parallel to the compression face.

1.5.14 – DEVELOPMENT LENGTH OF DEFORMED BARS AND DEFORMED WIRED IN TENSION

The development length "l_d", in inches, of deformed bars and deformed wire in tension shall be computed as the product of the basic development length of (1) and the applicable modification factor or factors of (2), (3), and (4), but "l_d" shall be not less than that specified in (5).

1. The basic development length shall be:

For #11 or smaller bars	$\frac{0.04 A_b f_y}{\sqrt{f'_c}}$ (1)
but not less than	$0.0004 d_b f_y$ (2)
For #14 bars	$\frac{0.085 f_y}{\sqrt{f'_c}}$ (3)
For #18 bars	$\frac{0.11 f_y}{\sqrt{f'_c}}$ (3)
For deformed wire	$\frac{0.03 d_b f_y}{\sqrt{f'_c}}$

2. The basic development length shall be mulitiplied by a factor of 1.4 for top reinforcement.[4]

3. When lightweight aggregate concrete is used, the basic development lengths in (1) shall be multiplied by 1.33 for "all-lightweight" concrete and 1.18 for "sand-lightweight" concrete with linear interpolation when partial sand replacement is used, or the basic development length may be multiplied by:

$$\frac{6.7\sqrt{f'c}}{f_{ct}}$$

but not less than 1.0, when "f_{ct}" is specified. The factors of (2) and (4) shall also be applied.

4. The basic development length, may be multiplied by the applicable factor or factors for:

Reinforcement being developed in the length under consideration and spaced laterally at least 6 in. on center and at least 3 in. from the side face of the member 0.8

Where anchorage or development for "f_y" is not specifically required, reinforcement in flexural members in excess of that required . . . . (A_s required) / (A_s provided)

Bars enclosed within a spiral which is not less than ¼ in. diameter and not more than 4 in. pitch 0.75

5. The development length, "l_d", shall be taken as not less than 12 in. except in the computation of lap splices by Article 1.5.22(B) and anchorage of shear reinforcement by Article 1.5.21.

1.5.15 – DEVELOPMENT LENGTH OF DEFORMED BARS IN COMPRESSION

The development length "l_d" for bars in compression shall be computed as:

$$\frac{0.02 f_y d_b}{\sqrt{f'_c}}$$

but shall not be less than $0.0003 f_y d_b$ or 8 in. Where excess bar area is provided, the "l_d" length may be reduced by the ratio of required area to area provided. The development length may be reduced 25 percent when the reinforcement is enclosed by spirals not less than ¼ in. in diameter and not more than 4 in. pitch.

1 The constant carries the unit of 1/in.
2 The constant carries the unit of in. ^{2}/lb.
3 The constant carries the unit of in.

4 Top reinforcement is horizontal reinforcement so placed that more than 12 in. of concrete is cast in the member below the bar.

1.5.16 – DEVELOPMENT LENGTH OF BUNDLED BARS

The development length of each bar of bundled bars shall be that for the individual bar, increased by 20 percent for a three-bar bundle, and 33 percent for a four-bar bundle.

1.5.17 – STANDARD HOOKS

A. Standard hooks shall be considered to develop a tensile stress in bar reinforcement $f_h = \xi \sqrt{f'_c}$ where "ξ" is not greater than the values in Table 1.5.17. The value of "ξ" may be increased 30 percent where enclosure is provided perpendicular to the plane of the hook. Enclosure may consist of external concrete or internal closed ties, spirals or stirrups.

B. An equivalent embedment length "l_e" shall be computed using the provisions of Article 1.5.14(1) by substituting "f_h" for "f_y" and "l_e" for "l_d".

C. Hooks shall not be considered effective in adding to the compressive resistance of reinforcement.

TABLE 1.5.17 – ξ Values

Bar Size	f_y= 60 ksi		f_y = 40 ksi
	Top bars	Other bars	All bars
#3 to #5	540	540	360
#6	450	540	360
#7 to #9	360	540	360
#10	360	480	360
#11	360	420	360
#14	330	330	330
#18	220	220	220

1.5.18 – COMBINATION DEVELOPMENT LENGTH

Development length, "l_d", may consist of a combination of the equivalent embedment length of a hook or mechanical anchorage plus additional embedment length of the reinforcement measured from the point of tangency of the hook.

1.5.19 – DEVELOPMENT OF WELDED WIRE FABRIC

A. The yield strength of smooth longitudinal wires of welded fabric shall be considered developed by embedding at least two cross wires with the closer one at least 2 in. from the point of critical section. An embedment of one cross wire at least 2 in. from the point of critical section may be considered to develop half the yield strength.

B. The development length, "l_d", of welded deformed wire fabric may be computed as a deformed wire in Article 1.5.14(1), (2), (3), and (4) by substituting ($f_y - 20{,}000n_w$)* for "f_y", where "n_w" is the number of cross wires within the development length which are at least 2 in. from the critical section. The minimum development length shall be

$$\frac{250 A_w}{s_w}$$

where "A_w" is the area of one tension wire and "s_w" is the spacing of wires.

1.5.20 – MECHANICAL ANCHORAGE

Any mechanical device capable of developing the strength of the reinforcement without damage to the concrete may be used as anchorage.

1.5.21 – ANCHORAGE OF SHEAR REINFORCEMENT

A. Shear reinforcement shall extend to a distance "d" from the extreme compression fiber and shall be carried as close to the compression and tension surfaces of the member as cover rquirements and the proximity of other reinforcement permit. Shear reinforcement shall be anchored at both ends for its design yield strength.

B. The ends of single leg, single U-, or multiple U-stirrups shall be anchored by one of the following means:

1. A standard hook plus an effective embedment of "$0.5l_d$". The effective embedment of a stirrup leg shall be taken as the distance between the middepth of the member "d/2" and the start of the hook (point of tangency).
2. Embedment above or below the middepth, "d/2" of the member on the compression side for a full development lenght "l_d" but not less than 24 bar diameters or, for deformed bars or deformed wire, 12 in.
3. Bending around the longitudinal reinforcement through a least 180-deg. Hooking or bending stirrups around the longitudinal reinforcement shall be considered effective anchorage only when the stirrups make an angle of at least 45-deg. with the longitudinal bars.
4. For each leg of welded plain wire fabric forming single U-stirrups, either:
   a. Two longitudinal wires running at a 2 in. spacing along the beam at the top of the U.
   b. One longitudinal wire not more than "d/4" from the compression face and a second wire closer to the compression face and spaced at least 2 in. from the first. The second wire may be beyond a bend or on a bend which has an inside diameter of at least 8 wire diameters.

C. Pairs of U-stirrups or ties so placed as to form a closed unit shall be considered properly spliced when the laps are "$1.7l_d$".

*The 20,000 has units of psi.

D. Between the anchored ends, each bend in the continuous portion of a transverse single U- or multiple U-stirrup shall enclose a longitudinal bar.

E. Longitudinal bars bent to act as shear reinforcement shall, in a region of tension, be continuous with the longtudinal reinforcement and in a compression zone shall be anchored, above or below the middepth "d/2" as specified for development length in Article 1.5.14 for that part of the stress in the reinforcement needed to satisfy Eq. (5-6) or Eq. (6-25).

1.5.22 – SPLICES IN REINFORCEMENT

A. General

1. Splices of reinforcement shall be made only as shown on the design drawings or as specified, or as authorized by the engineer. Except as provided herein, all welding shall conform to Recommended Practices for Welding Reinforcing Steel, Metal Inserts and Connections in Concrete Constructions (AWS D12.1).
2. Lap splices shall not be used for bars larger than No. 11.
3. Lap splices of bundled bars shall be based on the lap splice length required for individual bars of the same size as the bars spliced and such individual splices within the bundle shall not overlap each other. The length of lap, as prescribed in Article 1.5.22(B) or (C) shall be increased 20 percent for a three-bar bundle and 33 percent for a four-bar bundle.
4. Bars spliced by noncontact lap splices in flexural members shall not be spaced transversely farther apart than one-fifth the required length of lap nor 6 in.
5. Welded splices or other positive connections may be used. A full welded splice is one in which the bars are butted and welded to develop in tension at least 125 percent of the specified yield specified yield strength of the bar.

A full positive connection is one in which the bars are connected to develop in tension or compression, as required, at least 125 percent of the specified yield strength of the bar.

B. Splices in Tension

1. Classification of tension lap splices–The minimum length of lap for tension lap splices shall be that given in this Article, but not less than 12 inches. "l_d" is the tensile development length for the full "f_y" as given in Article 1.5.14(1), (2), (3) and (4).

Class A splices	$1.0l_d$
Class B splices	$1.3l_d$
Class C splices	$1.7l_d$
Class D splices	$2.0l_d$

The bars in a Class D splice shall be enclosed within a spiral meeting the requirements of Article 1.5.14(4) but no reduction in required development length shall be allowed for the effect of the spiral. In a Class D splice the ends of bars larger than No. 4 shall be hooked 180-deg.

2. Splices in tension tie members—Where feasible, splices shall be staggered and made with full welded or full positive connections as given in Article 1.5.22(A)(5). If lap splices are used, they shall meet the requirements of a Class D splice (lap of "$2.0l_d$").

3. Tension splices in other members—

a. In regions of high tensile stress—Splices in regions where the tensile reinforcement provided is equal to or less than twice that required for strength shall meet the following requirements:

If no more than one-half the bars are lap spliced within a required lap length, splices shall meet the requirements for Class B splices (lap of "$1.3l_d$").

If more than one-half of the bars are lap spliced within a required lap length, splices shall meet the requirements for Class C splices (lap of "$1.7l_d$").

If welded splices or positive connections are used they shall meet the requirements of Article 1.5.22(A)(5).

b. In regions of low tensile stress—Splices in regions where the tensile reinforcement provided is more than twice that required for strength shall meet the following requirements:

If no more than three-quarters of the bars are lap spliced within a required lap length, splices shall meet the requirements for Class A splices (lap of "$1.0l_d$").

If more than three-quarters of the bars are lap spliced within a required lap length, splices shall meet the requirements for Class B splices (lap of "$1.3l_d$").

If welded splices or positive connections are used, the requirements of Article 1.5.22(A)(5) may be waived if the splices are staggered at least 24 in. and in such a manner as to develop at every section at least twice the calculated tensile force at the section and in no case less than 20,000 psi on the total sectional area of all bars used. In computing the capacity developed at each section, spliced bars shall be rated at the specified splice strength. Unspliced bars shall be rated at the amount of anchorage provided on either side of the section.

C. Splices in Compression

1. Lap splices in compression—

a. The minimum length of lap for compression lap splices shall be "$0.0005f_yd_b$," in inches, but not less than 12 in. When the specified concrete strengths are less than 3000 psi, the lap shall be increased by one-third.

b. In tied compression members where ties throughout the lap length have an effective area of at least "0.0015hs", 0.83 of the lap length specified in paragraph (a) above may be used, but the lap length shall be not less than 12 in. Tie legs perpendicular to dimension h shall be used in determining the effective area.

c. Within the spiral of spiral compression members, 0.75 of the lap length specified in paragraph (a) above may be used, but the lap length shall not be less than 12 in.

2. End bearing—In bars required for compression only, the compressive stress may be transmitted by bearing of square cut ends held in concentric contact by a suitable device. Ends shall terminate in flat surfaces within 1½-deg. of right angles to the axis of the bars and shall be fitted within 3-deg. of full bearing after assembly. End bearing splices shall not be used except in members containing closed ties, closed stirrups, or spirals.

3. Welded splices or positive connections—Welded splices or positive connections used in compression shall meet the requirements of Article 1.5.22(A)(5).

D. Splices of Welded Plain Wire Fabric

1. In regions of high tensile stress—lap splices of wires in regions where the tensile reinforcement provided is equal to or less than twice that required for strength shall have outermost cross wires of each fabric sheet overlapped not less than the spacing of the cross wires plus 2 in.

2. In regions of low tensile stress—lap splices of wires in regions where the tensile reinforcement provided is more than twice that required for strength shall have outermost cross wires of each fabric sheet overlapped not less than 2 in.

E. Splices of Deformed Wire and Welded Deformed Wire Fabric

The minimum splice length for deformed wire reinforcement shall be

$$\frac{0.045 d_b f_y}{\sqrt{f'_c}} \quad \text{but not less than 12 in.}$$

The minimum splice length for welded deformed wire fabric reinforcement shall be

$$0.045\, d_b \left(\frac{f_y - 20{,}000n}{\sqrt{f'_c}}\right)$$

but the outermost cross wires in the splice shall be lapped at least

$$\frac{A_w}{s_w}\left[360 - \left(\frac{8 l_w}{d_b}\right)\right]$$

where "n" is the number of pairs of cross wires in the splice and "l_w" is the total lengths of wire extending beyond the outermost cross wires for each pair of spliced wires.

1.5.23 – ANALYSIS METHODS

A. General

1. All members of continuous and rigid frame structures shall be designed for the maximum effects of the loads specified in Section 2 as determined by the theory of elastic analysis.
2. Consideration shall be given to the effects of forces due to shrinkage, temperature changes, creep, and unequal settlement of supports

B. Expansion and Contraction

1. In general, provision for temperature changes shall be made in simple spans when the span length exceeds 40 feet.
2. In continuous bridges, provision shall be made in the design to resist thermal stresses induced or means shall be provided for movement caused by temperature changes.
3. Movements not otherwise provided for shall be provided by rockers, sliding plates, elastomeric pads or other means.

C. Stiffness

1. Any reasonable assumptions may be adopted for computing the relative flexural and torsional stiffnesses of continuous and rigid frame members. The assumptions made shall be consistent throughout the analysis.
2. The effect of haunches shall be considered both in determining bending moments and in design of members.

D. Modulus of Elasticity

1. The modulus of elasticity, "E_c" for concrete may be taken as $w^{1.5} 33 \sqrt{f'_c}$, in psi, for values of w between 90 and 155 lb. per cu. ft. For normal weight concrete (w = 145 pcf), "E_c" may be considered as $\sqrt{57{,}000 f'_c}$
2. The modulus of elasticity of nonprestressed steel reinforcements may be taken as 29,000,000 psi.

E. Thermal and Shrinkage Coefficients

1. The thermal coefficient for normal weight concrete may be taken as 0.000006 per deg. F.
2. The shrinkage coefficient for normal weight concrete may be taken as 0.0002.
3. Thermal and shrinkage coefficients for lightweight concrete shall be determined for the type of lightweight aggregate used.

F. Span Length

1. The span length of members that are not built integrally with their supports shall be considered the clear span plus the depth of the member but need not exceed the distance between centers of supports.
2. In analysis of continuous and rigid frame members, center-to-center distances shall be used in the determination of moments. Moments at faces of support may be used for member design. When fillets making an angle of 45-degrees or more with the axis of a continuous or restrained member are built momolithic with the member and support, face of support shall be considered at a section where the combined depth of the member and fillet is at least one and one-half times the thickness of the member. No portion of a fillet shall be considered as adding to the effective depth.
3. The effective span length of slabs shall be as specified in Article 1.3.2(A).

G. Computation of Deflections

1. Where deflections are to be computed, they shall be based on the cross sectional properties of the entire superstructure section except railings, curbs, sidewalks or any element not placed monolithically with the superstructure section before falsework removal.
2. Computation of live load deflection may be based on the assumption that the superstructure flexural members act together and have equal deflection. The live loading shall consist of all traffic lanes fully loaded, with reduction in load intensity allowed, as specified in Article 1.2.9. The live loading shall be considered uniformly distributed to all longitudinal flexural members.
3. Computation of immediate deflection–Deflections which occur immediately on application of load shall be computed by the usual methods or formulas for elastic deflections. Unless values are obtained by a more comprehensive analysis, deflections shall be computed taking the modulus of elasticity for concrete as specified in Article 1.5.23(D)(1) for normal weight or lightweight concrete and taking the effective moment of inertia as follows, but not greater than "I_g".

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \qquad (4\text{-}1)$$

where

$$M_{cr} = \frac{f_r I_g}{Y_t}$$

and "fr" = modulus of rupture of concrete specified in Article 1.5.26(A)(1).

For continuous spans, the effective moment of inertia may be taken as the average of the values obtained from Eq. (4-1) for the critical positive and negative moment sections.

4. Computation of long-time deflection–Unless values are obtained by a more comprehensive analysis, the additional longtime deflection for both normal weight and lightweight concrete flexural members shall be obtained by multiplying the immediate

deflection caused by the sustained load considered, computed in accordance with paragraph (3) above, by the factor

$$\left[2 - 1.2 \left(\frac{A'_s}{A_s}\right)\right] \geq 0.6$$

H. Bearings

Bearing devices shall be designed in accordance with Articles 1.7.49 through 1.7.56, or Section 12, Elastomeric Bearings. Bearing stresses in concrete shall not exceed the values given in Article 1.5.26 or 1.5.36.

I. Composite Concrete Flexural Members

1. Application

Composite flexural members consist of concrete elements constructed in separate placements but so interconnected that the elements respond to loads as a unit.

2. General Considerations

a. The total depth of the composite member or portions thereof may be used in resisting the shear and the bending moment. The individual elements shall be investigated for all critical stages of loading.

b. If the specified strength, unit weight, or other properties of the various components are different, the properties of the individual components, or the most critical values, shall be used in design.

c. In calculating flexural strength of a composite member by load factor design, no distinction shall be made between shored and unshored members.

d. All elements shall be designed to support all loads introduced prior to the full development of the design strength of the composite member.

e. Reinforcement shall be provided as necessary to prevent separation of the components.

3. Shoring

When used, shoring shall not be removed until the supported elements have developed the design properties required to support all loads and limit deflections and cracking at the time of shoring removal.

4. Vertical Shear

a. When the total depth of the composite member is assumed to resist the vertical shear, the design shall be in accordance with the requirements of Article 1.5.29 or Article 1.5.35 as for a monolithically cast member of the same cross-sectional shape.

b. Shear reinforcement shall be fully anchored in accordance with Article 1.5.21. Extended and anchored shear reinforcement may be included as ties for horizontal shear.

5. Horizontal Shear

In a composite member, full transfer of the shear forces shall be assured at the interfaces of the separate components. Design for horizontal shear shall be in accordance with the requirements of Article 1.5.29(E) or Article 1.5.35(E).

J. T-Girder Construction

1. In T-girder construction, the girder web and slab shall be built integrally or otherwise effectively bonded together. Full transfer of shear forces shall be assured at the interface of web and slab. Where applicable, the design requirements of Article 1.5.23(I) for composite concrete members shall apply.

2. Compression flange width:

   a. The effective slab width as a T-girder flange shall not exceed one-fourth of the span length of the girder, and its overhanging width on either side of the girder shall not exceed six times the thickness of the slab nor one-half the clear distance to the next girder.

   b. For girders having a slab on one side only, the effective overhanging flange width shall not exceed 1/12 of the span length of the girder, nor six times the thickness of the slab, nor one-half the clear distance to the next girder.

   c. Isolated T-girders in which the flange is used to provide additional compression area shall have a flange thickness not less than one-half the width of the girder web and a total flange width not more than four times the width of the girder web.

3. Slab thickness

The thickness of the slab shall be designed in accordance with Article 1.3.2(C) Case A, but shall not be less than the minimum specified in Table 1.5.40

4. Diaphragms

Diaphragms shall be placed between the girders at span ends, and within the spans at intervals not exceeding 40 feet. Diaphragms may be omitted where tests or structural analysis show adequate strength.

K. Box-Girder Construction

1. In box-girder construction, the girder web and top and bottom slab shall be built integrally or otherwise effectively bonded together. Full transfer of shear forces shall be assured at the interfaces of the girder web with the top and bottom slab. Design shall be in accordance with the requirements of Article 1.5.23(I). When required by design, changes in girder web thickness shall be tapered for a minimum distance of 12 times the difference in web thickness.

2. Compression flange width

The entire slab width shall be assumed to be effective in compression.

3. Top and bottom slab thickness

a. The thickness of the top slab shall be designed in accordance with Article 1.3.2(C) Case A, but shall not be less than the minimum specified in Table 1.5.40.

b. The thickness of the bottom slab shall be not less than 1/16 of the clear span between girder webs or 5½ in., whichever is greater, except that the thickness need not be greater than the top slab unless required by design.

4. Top and bottom slab reinforcement

a. Minimum distributed reinforcement of 0.4 percent of the flange area shall be placed in the bottom slab parallel to the girder span. A single layer of reinforcement may be provided. The spacing of such reinforcement shall not exceed 18 in.

b. Minimum distributed reinforcement of 0.5 percent of the cross-sectional area of the slab, based on the least slab thickness, shall be placed in the bottom slab transverse to the girder span. Such reinforcement shall be distributed over both surfaces with a maximum spacing of 18 in. All transverse reinforcement in the bottom slab shall extend to the exterior face of the outside girder web in each group and be anchored by a standard 90-deg. hook.

c. At least 1/3 of the bottom layer of the transverse reinforcement in the top slab shall extend to the exterior face of the outside girder web in each group and be anchored by a standard 90-deg. hook. If the slab extends beyond the last girder web, such reinforcement shall extend into the slab overhang and shall have an anchorage beyond the exterior face of the girder web not less than that provided by a standard hook.

5. Diaphrahms

Diaphragms or spreaders shall be placed between the girders at span ends, and within the spans at intervals not exceeding 60 feet. Diaphragms may be omitted where tests or structural analysis show adequate strength. Diaphragm spacing for curved girders shall be given special consideration.

L. Concrete Arch Construction

1. Shape of Arch Rings

Arch rings shall be selected as to shape in such manner that the axis of the ring shall conform, as nearly as practicable, to either the equilibrium polygon for ful dead load or to the equilibrium polygon for full dead plus one-half live load over the full span, whichever produces the smallest bending stresses under combined loads.

2. Spandrel Walls

When the spandrel walls or filled spandrel arches exceed 8 feet in height above the extrados they shall be designed as vertical slabs supported by transverse diaphragm walls or deep counterforts. Vertical cantilever walls over 8 feet in height, or counterforts having a back slope of less than 45-degrees with the vertical, shall not be used due to indeterminate stresses set up in the arch ring by torsion.

3. Expansion Joints

Vertical expansion joints shall be placed in the spandrel walls of arches to provide for movement due to temperature change and arch deflection. Such joints shall be placed at the ends of spans and at intermediate points, preferably not more than 50 feet apart.

4. Reinforcement

Arch ribs shall be reinforced with a complete double line of longitudinal reinforcement consisting of an intradosal system and an extradosal system connected by a series of stirrups or ties.

For barrel arches, a system of transverse reinforcement, fully anchored to the longitudinal reinforcement, shall be used in both intrados and extrados. The transverse reinforcement shall be proportioned to resist the bending stresses due to any overturning action of the spandrel wall.

For rib arches, the longitudinal rib reinforcement shall be enclosed by lateral hoops or ties as for compression members.

5. Waterproofing

Preferably, the top of the arch ring and the interior faces of the spandrel walls of all filled spandrel arches shall be waterproofed with a membrane waterproofing constructed in accordance with the requirements specified in Division II, Section 17.

6. Drainage of Spandrel Fill

The fills of filled spandrel arches shall be effectively drained by a system of tile drains or French drains laid along the intersection of the spandrel walls and arch rings and discharging through suitable outlets in the piers and abutments. The location and details of the drainage outlets shall be such as to eliminate, as far as possible, the discoloration by drainage water of the exposed concrete surfaces.

1.5.24 – DESIGN METHODS

A. The design of reinforced concrete members shall be made either with reference to service loads and allowable service load stresses as provided in SERVICE LOAD DESIGN or alternately, with reference to load factors and strengths as provided in LOAD FACTOR DESIGN.

B. All applicable provisions of this specification shall apply to both methods of design, except Articles 1.2.4 and 1.2.16 shall not apply for design by LOAD FACTOR DESIGN.

C. The strength and serviceability requirements of LOAD FACTOR DESIGN may be assumed to be satisfied for design by SERVICE LOAD DESIGN if the service load stresses are limited to the values given in Article 1.5.26.

D. Members proportioned by LOAD FACTOR DESIGN will sustain without damage at least the following overload:

Members designed for Group 1 loading = $D + 5/3\ (L + I)$
Members designed for Group 1A loading = $D + 2.2(L + I)$

SERVICE LOAD DESIGN

1.5.25 – GENERAL REQUIREMENTS

A. For reinforced concrete members designed with reference to service loads and allowable stresses, the service load stresses shall not exceed the values given in Article 1.5.26.

B. Development and splices of reinforcement shall be as required under "DEVELOPMENT AND SPLICES OF REINFORCEMENT."

1.5.26 – ALLOWABLE SERVICE LOAD STRESSES

A. Concrete

For service load design, the stresses in concrete shall not exceed the following:

1. Flexure

Extreme fiber stress in compression, f_c . . . . $0.40f'_c$
Extreme fiber stress in tension for plain concrete, f_t . . . . $0.21f_r$
Modulus of rupture, f_r from tests, or if data are not available:

Normal weight concrete . . . . $7.5\sqrt{f'_c}$
"Sand-lightweight" concrete . . . . $6.3\sqrt{f'_c}$
"All-lightweight" concrete . . . . $5.5\sqrt{f'_c}$

2. Shear*

Beams: Shear carried by concrete, v_c . . . . $0.95\sqrt{f'_c}$
Maximum shear carried by concrete plus shear reinforcement, v . . . . $v_c + 4\sqrt{f'_c}$
Slabs and Footings (peripheral shear):**
Shear carried by concrete, v_c . . . . $1.8\sqrt{f'_c}$
Maximum shear carried by concrete plus shear reinforcement, v . . . . $3\sqrt{f'_c}$

3. Bearing on loaded area, f_b . . . . $0.30f'_c$

a. When the supporting surface is wider on all sides than the loaded area, the allowable bearing stress on the loaded area may be increased by $(A_2/A_1)^{1/2}$, but not more than 2.

b. When the supporting surface is sloped or stepped, "A_2" may be taken as the area of the lower base of the largest frustrum of a right pyramid or cone contained wholly within the support and having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal.

*For more detailed analysis of permissible shear stress, v_c, carried by the concrete, and shear values for lightweight aggregate concrete – see Article 1.5.29(B).
**See Article 1.5.26(F).

c. When the loaded area is subjected to high edge stresses due to deflection or eccentric loading, the allowable bearing stress on the loaded area shall be multiplied by a factor of 0.75. The requirements of (a) and (b) shall also apply.

B. Reinforcement

For service load design, the tensile stress in the reinforcement, "f_s" shall not exceed the following:

Grade 40 or Grade 50 reinforcement . . . . 20,000 psi
Grade 60 reinforcement . . . . 24,000 psi

Fatigue Stress Limit – The range between a maximum and minimum stress in straight reinforcement casued by live load plus impact shall not exceed 21,000 psi †. Bends in primary reinforcement shall be avoided in regions of high stress range.

1.5.27 – FLEXURE

For investigation of service load stresses, the straight-line theory of stress and strain in flexure shall be used and the following assumptions shall be made:

a. A section plane before bending remains plane after bending; strains vary as the distance from the neutral axis.

b. The stress-strain relation of concrete is a straight line under service loads within the allowable service load stresses. Stresses vary as the distance from the neutral axis except, for deep flexural members with overall depth span ratios greater than 2/5 for continuous spans and 4/5 for simple spans, a nonlinear distribution of stress should be considered.

c. The steel takes all the tension due to flexure.

d. The modular ratio, $n = E_s/E_c$, may be taken as the nearest whole number (but not less than 6). Except in calculations for deflections, the value of "n" for lightweight concrete shall be assumed to be the same as for normal weight concrete of the same strength.

e. In doubly reinforced flexural members, an effective modular ratio of $2E_s/E_c$ shall be used to transform the compression reinforcement for stress computations. The compressive stress in such reinforcement shall not be greater than the allowable tensile stress.

1.5.28 – COMPRESSION MEMBERS WITH OR WITHOUT FLEXURE

The combined axial load and moment capacity of compression members shall be taken as 35 percent of that computed in accordance with the provisions of Article 1.5.33. Slenderness effects shall be included according to the requirements of Article 1.5.34. The term "P_u" in Eq. (6-15) shall be replaced by 2.5 times the axial design load. In using the provisions of Articles 1.5.33 and 1.5.34 ϕ shall be taken as 1.0.

†Applicable primarily to concrete deck slabs and short span slab bridges where the dead load to total load ratio is less than approximately 0.125.

1.5.29 – SHEAR

A. Shear Stress

1. The design shear stress, v, shall be computed by:

$$v = \frac{V}{b_w d} \qquad (5\text{-}1)$$

where "b_w" shall be taken as the width of web and "d" shall be taken as the distance from the extreme compression fiber to the centroid of the longitudinal tension reinforcement. For a circular section, "b_w" shall be taken as the diameter and "d" need not be taken less than the distance from the extreme compression fiber to the centroid of the longitudinal reinforcement in the opposite half of the member.

2. When the reaction in the direction of the applied shear introduces compression into the end region of the member, sections located less than a distance "d" from the face of the support may be designed for the same shear, "v", as that computed at a distance "d".

3. The shear stress carried by the concrete, "v_c", shall be calculated according to Article 1.5.29(B). When "v" exceeds "v_c", shear reinforcement shall be provided according to Article 1.5.29(C). Whenever applicable, the effects of torsion* shall be added.

B. Permissible Shear Stress

1. The shear stress carried by the concrete, v_c, shall not exceed $0.95\sqrt{f'_c}$ unless a more detailed analysis is made in accordance with (2) or (3). For members subjected to axial tension, "v_c" shall not exceed the value given in (4). For lightweight concrete, the provisions of (5) shall apply.

2. The shear stress carried by the concrete, "v_c" may be computed by:

$$v_c = 0.9\sqrt{f'_c} + 1100 p_w \left(\frac{Vd}{M}\right) \qquad (5\text{-}2)$$

but "v_c" shall not exceed $1.6\sqrt{f'_c}$. The quantity Vd/M shall not be taken greater than 1.0, where "M" is the applied design moment occurring simultaneously with "V" at the section considered.

3. For members subjected to axial compression, "v_c" may be computed by:

$$v_c = 0.9\left[1 + 0.0007\left(\frac{N}{A_g}\right)\right]\sqrt{f'_c} \qquad (5\text{-}3)$$

The quantity N/A_g shall be expressed in psi.

*The design criteria for combined torsion and shear given in "Building Code Requirements for Reinforced Concrete – ACI 318-71" may be used.

4. For members subjected to significant axial tension, shear reinforcement shall be designed to carry the total shear, unless a more detailed analysis is made using

$$v_c = 0.9\left[1 + 0.004\left(\frac{N}{A_g}\right)\right]\sqrt{f'_c} \tag{5-4}$$

where "N" is negative for tension. The quantity N/A_g shall be expressed in psi.

5. The provisions for shear stress, "v_c" carried by the concrete apply to normal weight concrete. When lightweight aggregate concretes are used, one of the following modifications shall apply:

a. When "f_{cr}" is specified, the shear stress "v_c" shall be modified by substituting:

$$\frac{f_{ct}}{6.7} \text{ for } \sqrt{f'_c}$$

but the value of

$\frac{f_{ct}}{6.7}$ used shall not exceed $\sqrt{f'_c}$

b. When "f_{ct}" is not specified, the shear stress, "v_c", shall be multiplied by 0.75 for "all-lightweight" concrete, and 0.85 for "sand-lightweight" concrete. Linear interpolation may be used when partial sand replacement is used.

C. Design of Shear Reinforcement

1. Shear reinforcement shall conform to the general requirements of Article 1.5.10. When shear reinforcement perpendicular to the axis of the member is used, the required area shall be computed by:

$$A_v = \frac{(v - v_c)b_w s}{f_s} \tag{5-5}$$

2. When inclined stirrups or bent bars are used as shear reinforcement the following provisions apply:

a. When inclined stirrups are used, the required area shall be computed by:

$$A_v = \frac{(v - v_c)b_w s}{f_s(\sin a + \cos a)} \tag{5-6}$$

b. When shear reinforcement consists of a single bar or a single group of parallel bars, all bent up at the same distance from the support, the required area shall be computed by:

$$A_v = \frac{(v - v_c)b_w d}{f_s \sin a} \tag{5-7}$$

in which $(v - v_c)$ shall not exceed $1.5\sqrt{f'_c}$

c. When shear reinforcement consists of a series of parallel bent-up bars or groups of parallel bent-up bars at different distances from the support, the required areas shall be computed by (a).

d. Only the center three-fourth of the inclined portion of any longitudinal bar that is bent shall be considered effective for shear reinforcement.

3. Where more than one type of shear reinforcement is used to reinforce the same portion of the member, the required area shall be computed as the sum for the various types separately. In such computations, v_c shall be included only once.

4. When $(v - v_c)$ exceeds $2\sqrt{f'_c}$ the maximum spacings given in Article 1.5.10(C) shall be reduced by one-half.

5. The value of $(v - v_c)$ shall not exceed $4\sqrt{f'_c}$.

D. Shear-Friction

1. These provisions apply where it is inappropriate to consider shear as a measure of diagonal tension, and particularly in design of reinforcing details for precast concrete structural elements.

2. A crack shall be assumed to occur along the shear path. Relative displacement shall be considered resisted by friction maintained by shear-friction reinforcement across the crack. This reinforcement shall be approximately perpendicular to the assumed crack.

3. The shear stress, "v", shall not exceed "$0.09f'_c$", nor 360 psi.

4. The required area of reinforcement, "A_{vf}", shall be computed by:

$$A_{vf} = \frac{V}{f_s \mu} \tag{5-8}$$

The coefficient of friction, "μ" shall be 1.4 for concrete cast monolithically, 1.0 for concrete placed against hardened concrete, and 0.7 for concrete placed against as-rolled structural steel.

5. Direct tension across the assumed crack shall be provided by additional reinforcement.

6. The shear-friction reinforcement shall be well distributed across the assumed crack and shall be adequately anchored on both sides by embedment, hooks, or welding to special devices.

7. When shear is transferred between concrete placed against hardened concrete, the interface shall be rough, clean, and free of laitance with a full amplitude of approximately ¼ in. When shear is transferred between as-rolled steel and concrete, the steel shall be clean and without paint.

E. Horizontal Shear Design for Composite Concrete Flexural Members.

1. In a composite member, full transfer of the shear forces shall be assured at the interfaces of the separate components.

2. Full transfer of horizontal shear forces may be assumed when all of the following are satisfied: (a) the contact surfaces are clean and intentionally roughened. (b) minimum ties are provided in accordance with paragraph (6). (c) web members are designed to resist the entire vertical shear, and (d) all shear reinforcement is anchored into all intersecting components. When all of the above are not satisfied, horizontal shear shall be fully investigated.

3. The horizontal shear stress, "v_{dh}," may be computed at any cross section as:

$$v_{dh} = \frac{V}{b_v d} \tag{5-9}$$

in which d is for the entire composite section. Alternatively, in any segment not exceeding one-tenth of the span, the actual change in compressive or tensile force to be transferred may be computed, and provisions made to transfer that force as horizontal shear to the supporting element.

4. The horizontal shear may be transferred at contact surfaces using the permissible horizontal shear stress, "v_h," stated below.

   a. When ties are not provided, but the contact surfaces are clean and intentionally roughened, permissible "v_h" = 36 psi.

   b. When the minimum tie requirements of paragraph (6) are provided and the contact surfaces are clean but not intentionally roughened, permissible "v_h" = 36 psi.

   c. When the minimum tie requirements of paragraph (6) are provided and the contact surfaces are clean and intentionally roughened, permissible "v_h" = 160 psi.

   d. When "v_{dh}" exceeds 160 psi, design for horizontal shear shall be made in accordance with Article 1.3.29(D).

5. When tension exists perpendicular to any surface shear transfer by contact may be assumed only when the minimum tie requirements of paragraph (6) are satisfied.

6. Ties for horizontal shear

   a. When vertical bars or extended stirrups are used to transfer horizontal shear, the tie area shall not be less than that required by Article 1.5.10(A)(2) and the spacing shall not exceed four times the least dimension of the supported element nor 24 in.

   b. Ties for horizontal shear may consist of single bars, multiple leg stirrups, or the vertical legs of welded wire fabric. All ties shall be adequately anchored into the components by embedment of hooks.

7. Measure of roughness

   Internal roughness may be assumed only when the contact surface is roughened, clean, and free of laitance. Roughness shall have a full amplitude of approximately ¼ in.

F. Special Provisions for Slabs and Footings

1. The shear capacity of slabs and footings in the vicinity of concentrated loads or reactions shall be governed by the more severe of two conditions:

   a. The slab or footing acting as a wide beam, with a critical section extending in a plane across the entire width and located at a distance d from the face of the concentrated load or reaction area. For this condition, the slab of footing shall be designed in accordance with Articles 1.5.29(A) through (C).

   b. Two-way action for the slab or footing, with a critical section perpendicular to the plane of the slab and located so that its periphery is a minimum and approaches not closer than d/2 to the periphery of the concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with paragraphs (2) and (3).

2. The peripheral shear stress shall be computed by:

$$v = \frac{V}{b_o d} \tag{5-10}$$

in which "V" and "b_o" are taken at the critical section defined in (b). The peripheral shear stress, v, shall not exceed the the shear stress carried by the concrete, "v_c" = $1.8\sqrt{f_c'}$) unless shear reinforcement is provided in accordance with (3), in which case "v" shall not exceed $3\sqrt{f_c'}$.

3. Shear reinforcement consisting of bars or wires anchored in accordance with Article 1.5.21 may be provided. For design of such shear reinforcement, shear stresses shall be investigated at the critical section defined in (b) and at successive sections more distant from the support; and the shear stress, "v_c," carried by the concrete at any section shall not exceed $0.9\sqrt{f_c'}$. Where v exceeds v_c, the shear reinforcement shall be provided according to Article 1.5.29(C).

LOAD FACTOR DESIGN

1.5.30 – STRENGTH REQUIREMENTS

A. Strength

For reinforced concrete members designed with reference to load factors and strengths, the design strength of a member or cross section shall be taken as the theoretical strength computed in accordance with the requirements and assumptions of LOAD FACTOR DESIGN modified by a capacity reduction factor, ϕ.* The following values of ϕ shall be used:

For flexure . . . . $\phi = 0.90$
For shear . . . . $\phi = 0.85$
For spirally reinforced compression members with or without flexure . . . . $\phi = 0.75$
For tied compression members with or without flexure . . . . $\phi = 0.70$
For bearing on concrete . . . . $\phi = 0.70$

The value of "ϕ" may be increased linearly from the value for compression members to the value for flexure as the axial design load strength, "P_u", decreases from "$0.10f_c'A_g$" or "P_b," whichever is smaller, to zero.

Development and splices of reinforcement specified in DEVELOPMENT AND SPLICES OR REINFORCEMENT do not require a "ϕ" factor.

B. Required Strength

The required strength provided shall be at least equal to the structural effects of the load groups covered in Sec. 1.2.22, which represent various combinations of loads and forces to which a structure may be subjected. Each part of such structure shall be proportioned for the group loads that are applicable, and the maximum design required shall be used.

1.5.31 – DESIGN ASSUMPTIONS

A. The strength design of members for flexure and axial loads shall be based on the assumptious given in this article, and on satisfaction of the applicable conditions fo equilibrium and compatibility of strains.

1. Strain in the reinforcing steel and concrete shall be assumed directly proportional to the distance from the neutral axis.
2. The maximum usable strain at the extreme concrete compression fiber shall be assumed equal to 0.003.
3. Stress in reinforcement below the specified yield strength, "f_y", for the grade of steel used shall be taken as "E_s" times the steel strain. For strains greater than that corresponding to "f_y", the stress in the reinforcement shall be considered independent of strain and equal to "f_y".

*The coefficient ϕ provides for the possibility that small adverse variations in material strengths, workmanship and dimensions, while individually within acceptable tolerances and limits of good practice, may combine to result in undercapacity.

4. Tensile strength of the concrete shall be neglected in flexural calculations of reinforced concrete.
5. The relationship between the concrete compressive stress distribution and the concrete strain may be assumed to a rectangle, trapezoid, parabola, or any other shape which results in prediction of strength in substantial agreement with the results of comprehensive tests.
6. The requirements of Article 1.5.31(A)(5) may be considered satisfied by an equivalent rectangular concrete stress distribution which is defined as follows: A concrete stress of "$0.85f_c$'" shall be assumed uniformly distributed over an equivalent compression zone bounded by the edges of the cross section and a straight line located parallel to the neutral axis at a distance "$a = \beta_1 c$" from the fiber of maximum compressive strain. The distance "c" from the fiber of maximum strain to the neutral axis is measured in a direction perpendicular to that axis. The fraction "β_1 shall be taken as 0.85 for strengths "f_c" up to 4000 psi and shall be reduced continuously at a rate of 0.05 for each 1000 psi of strength in excess of 4000 psi.

1.5.32 – FLEXURE

A. Maximum Reinforcement of Flexural Members

1. For flexural members, the reinforcement ratio "ρ" provided shall not exceed 0.75 of that ratio, "ρ_b" which would produce balanced conditions for the seciton under flexure.
2. Balanced conditions exist at a cross section when the tension reinforcement reaches its specified yield strength, "f_y", just as the concrete in compression reaches its assumed ultimate strain of 0.003.

B. Rectangular Sections With.Tension Reinforcement Only

1. For rectangular sections, the design moment strength may be computed by:

$$M_u = \phi \left[A_s f_y d \left(1 - 0.6 \rho \frac{f_y}{f'_c}\right)\right] \tag{6-1}$$

$$= \phi \left[A_s f_y \left(d - \frac{a}{2}\right)\right] \tag{6-2}$$

where

$$a = \frac{A_s f_y}{0.85 f'_c b}$$

2. The balanced reinforcement ration, "ρ_b" for rectangular sections with tension reinforcement only is given by:

$$\rho_b = \left(\frac{0.85\beta_1 f'_c}{f_y}\right)\left(\frac{87{,}000}{87{,}000 + f_y}\right) \tag{6-3}$$

C. **I- and T-Sections With Tension Reinforcement Only**

1. When the compression flange thickness is equal to or greater than the depth of the equivalent rectangular stress block, "a", the design moment strength may be computed by the equations given in (B)(1);

2. When the compression flange thickness is less than "a", the design moment strength may be computed by:

$$M_u = \phi\left[(A_s - A_{sf}) f_y \left(d - \frac{a}{2}\right) + A_{sf} f_y (d - 0.5h_f)\right] \tag{6-4}$$

where

$$A_{sf} = \frac{0.85 f'_c (b - b_w) h_f}{f_y}$$

$$a = \frac{(A_s - A_{sf}) f_y}{0.85 f'_c b_w}$$

3. The balanced reinforcement ratio, b, for I- and T-sections with tension reinforcement only is given by:

$$\rho_b = \frac{b_w}{b}\left[\left(\frac{0.85\beta_1 f'_c}{f_y}\right)\left(\frac{87{,}000}{87{,}000 + f_y}\right) + \rho_f\right] \tag{6-5}$$

where

$$\rho_f = \frac{A_{sf}}{b_w d}$$

4. For T-girder and box-girder construction defined by Articles 1.5.23(J) and 1.5.23(K), the width of the compression face "b", shall be equal to the effective slab width.

D. Rectangular Sections With Compression Reinforcement

1. For rectangular sections, the design moment strength may be computed by:

$$M_u = \phi\left[(A_s - A'_s) f_y \left(d - \frac{a}{2}\right) + A'_s f_y (d - d')\right] \tag{6-6}$$

where

$$a = \frac{(A_s - A'_s) f_y}{0.85 f'_c b}$$

and the following condition shall be satisfied:

$$\frac{(A_s - A'_s)}{bd} \geqq 0.85\beta_1 \left(\frac{f'_c d'}{f_y d}\right)\left(\frac{87{,}000}{87{,}000 - f_y}\right) \tag{6-7}$$

2. When the value of $(A_s - A_s')/bd$ is less than the value given by Eq. (6-7), so that the stress in the compression reinforcement is less than the yield strength, "f_y", or when effects of compression reinforcement are neglected, the moment strength may be computed by the equations in (B)(1), except when a general analysis is made based on stress and strain compatibility using the assumptions given in Article 1.5.31.
3. The balanced reinforcement ratio, " $_b$," for rectangular sections with compression reinforcement is given by:

$$\rho_b = \left(\frac{0.85\beta_1 f'_c}{f_y}\right)\left(\frac{87{,}000}{87{,}000 + f_y}\right) + \rho'\left(\frac{f'_s}{f_y}\right) \tag{6-8}$$

where

f'_s = stress in compression reinforcement

$$= 87{,}000\left[1 - \left(\frac{d'}{d}\right)\left(\frac{87{,}000 + f_y}{87{,}000}\right)\right] \leqq f_y$$

E. Other Cross Sections

For other cross sections the design moment strength, "$M_u = \phi M_t$," shall be computed by a general analysis based on stress and strain compatibility using the assumptions given Article 1.5.31. The requirements of Article 1.5.32(A) shall also be satisfied.

1.5.33 – COMPRESSION MEMBERS WITH OR WITHOUT FLEXURE

A. General Requirements

1. The design of cross sections subject to combined flexure and axial load shall be based on stress and strain compatibility using the assumptions given in Article 1.5.31. Slenderness effects shall be included according to the requirements of Article 1.5.34.
2. All members subjected to a compression load shall be designed for an eccentricity, "e" equal to the greater of

   a. that corresponding to the maximum design moment which accompanies this compression load, or
   b. 0.05h for spirally reinforced compression members, or "0.10h" for tied compression members, about either axis, or
   c. 1 in. about either axis.

B. Compression Member Strengths

The following provisions may be used as a guide to define the range of the load-moment interaction relationship for members subjected to combined flexure and axial load.

1. Pure Compression

The axial design load strength in pure compression, "P_o", may be computed by:

$$P_o = \phi\left[0.85 f'_c (A_s - A_{st}) + A_{st} f_y\right] \tag{6-9}$$

Concentric loading is a hypothetical loading condition since all members subjected to a compression load shall be designed for eccentricities not less than the value given in Article 1.5.33(A)(2).

2. Pure Flexure

The assumptions given in Article 1.5.31, or the applicable equations for flexure given in Article 1.5.32 may be used to compute the design moment strength, "M_u", in pure flexure.

3. Balanced Conditions

Balanced conditions for a cross section are defined in Article 1.5.32(A)(2). For a rectangular section with reinforcement in one or two faces and located at approximately the same distance from the axis of bending, the balanced load, "P_b", and balanced moment, "M_b", may be computed by:

$$P_b = \phi \left[0.85 f'_c \, b a_b + A'_s f'_s - A_s f_y \right] \tag{6-10}$$

and

$$M_b = P_b e_b = \phi \left[0.85 f'_c \, b a_b \left(d - d'' - \frac{a_b}{2} \right) + A'_s f'_s (d - d' - d'') + A_s f_y d'' \right] \tag{6-11}$$

where

$$a_b = \left(\frac{87{,}000}{87{,}000 + f_y} \right) B_1 d$$

and

$$f'_s = 87{,}000 \left[\left(1 - \left(\frac{d'}{d} \right) \left(\frac{87{,}000 + f_y}{87{,}000} \right) \right) \right] \leqq f_y$$

4. Combined Flexure and Axial Load

The design strength under combined flexure and axial load shall be based on stress and strain compatibility using the assumptions given in Article 1.5.31. The strength of a cross section is controlled by tension when the axial design load strength, "P_u", is less than "P_b" (or "e" is greater than "e_b"). The strength of a cross section is controlled by compression when the axial design load strength, "P_u", is greater than "P_b" (or "e" is less than "e_b").

The combined axial load and moment strength shall be multiplied by the appropriate capacity reduction factor, "ϕ" for spirally reinforced or tied compression members as given in Article 1.5.30(A). The value of "ϕ" may be increased linearly from the value for compression members to the value for flexure as the axial design load strength, "P_u", decreased from "$0.10 f'_c A_g$ or "P_b", whichever is smaller, to zero.

C. Biaxial Loading

In lieu of a general section analysis based on stress and strain compatibility for a loading condition of biaxial bending, the design strength of non-circular members subjected to biaxial bending may be computed by the following approximate expressions:

$$P_{uxy} = \frac{1}{\left(\frac{1}{P_{ux}}\right)+\left(\frac{1}{P_{uy}}\right)+\left(\frac{1}{P_o}\right)} \qquad (6\text{-}12)$$

when the applied axial design load,

$$P_u \geqq 0.1 f'_c A_g$$

or

$$\frac{M_x}{M_{ux}} + \frac{M_y}{M_{uy}} \leqq 1 \qquad (6\text{-}13)$$

when the applied axial design load,

$$P_u < 0.1 f'_c A_s$$

1.5.34 – SLENDERNESS EFFECTS IN COMPRESSION MEMBERS

A. General Requirements

1. The design of compression members shall be based on forces and moments determined from an analysis of the structure. Such an analysis shall take into account the influence of axial loads and variable moment of inertia on member stiffness and fixed-end moments, the effect of deflections on the moments and forces, and the effects of the duration of the loads.
2. In lieu of the procedure described in paragraph (1), the design of compression members may be based on the approximate procedure given in Article 1.5.34(B).

B. Approximate Evaluation of Slenderness Effects

1. The unsupported length, "l_u", of a compression member shall be taken as the clear distance between slabs, girders, or other members capable of providing lateral support for the compression member. Where haunches are present, the unsupported length shall be measured to the lower extremity of the haunch in the plane considered.
2. The radius of gyration, "r", may be taken equal to 0.30 times the overall dimension in the direction in which stability is being considered for rectangular compression members, and 0.25 times the diameter for circular compression members. For other shapes, "r" may be computed for the gross concrete section.

3. For compression members braced against sidesway, the effective length factor, "k", shall be taken as 1.0, unless an analysis shows that a lower value may be used. For compression members not braced against sidesway, the effective length factor, "k", shall be determined with due consideration of cracking and reinforcement on relative stiffness, and shall be greater than 1.0.

4. For compression members braced against sidesway, the effects of slenderness may be neglected when, "kl_u/r" is less than "$34-12M_1/M_2$". For compression members not braced against sidesway, the effects of slenderness may be neglected when "kl_u/r" is less than 22. For all compression members with "kl_u/r" greater than 100, an analysis as defined in Article 1.5.34(A)(1) shall be made. "M_1" = value of smaller design end moment on compression member calculated from a conventional elastic analysis, positive if member is bent in single curvature, negative if bent in double curvature. "M_2" = value of larger design end moment on compression member calculated from a conventional elastic analysis, always positive.

5. Compression members shall be designed using the applied axial design load, "P_u" from a conventional elastic analysis and a magnified moment, "M_c" defined by:

$$M_c = \sigma M_z \qquad (6\text{-}14)$$

where

$$\sigma = \frac{C_m}{1 - \dfrac{P_u}{\phi P_c}} \geq 1.0 \qquad (6\text{-}15)$$

and

$$P_c = \frac{\pi^2 EI}{(K l_u)^2} \qquad (6\text{-}16)$$

In lieu of a more precise calculation, EI may be taken either as

$$EI = \frac{\dfrac{E_c I_g}{5} + E_s I_s}{1 + \beta_d} \qquad (6\text{-}17)$$

or conservatively

$$EI = \frac{\dfrac{E_c I_g}{2.5}}{1 + \beta_d} \qquad (6\text{-}18)$$

where "β_d" is the ratio of maximum design dead load moment to maximum design total load moment, always positive. For members braced against sidesway and without transverse loads between supports, "C_m" may be taken as

$$C_m = 0.6 + 0.4 \frac{M_1}{M_2} \qquad (6\text{-}19)$$

but not less than 0.4.

For all other cases "C_m" shall be taken as 1.0.

6. When design of compression members is governed by the minimum eccentricities specified in Article 1.5.33(A)(2), "M_2" in Eq; (6-14) shall be based on the specified minimum eccentricity with conditions of curvature determined by either of the following:

a. When the actual computed eccentricities are less than the specified minimum, the computed end moments may be used to evaluate the conditions of curvature.

b. If computations show that there is no eccentricity at both ends of the member, conditions of curvature shall be based on a ratio of M_1/M_2 equal to one.

7. When compression members are subject to bending about both principal axis, the moment about each axis shall be amplified by δ computed from the corresponding conditions of restraint about that axis.

8. In structures which are not braced against sidesway, the flexural members shall be designed for the total magnified end moments of the compression members at the joint.

9. When a group of compression members on one level comprise a bent, or when they are connected integrally to the same superstructure, and collectively resist the sidesway of the structure, the value of δ shall be computed for the member group. "P_u" and "P_c" shall be taken as the summation of "P_u" and "P_c" for all members in the group. In designing each member in the group, δ shall be taken as the larger of (a) the value computed for the group as a whole, or (b) the value computed for the individual compression member assuming its ends to be braced against sidesway.

1.5.35 – SHEAR

A. Shear Strength

1. The design shear stress, "v_u", shall be computed by:

$$v_u = \frac{V_u}{\phi b_w d} \tag{6-20}$$

where "b_w" shall be taken as the width of web and "d" shall be taken as the distance from the extreme compression fiber to the centroid of the longitudinal tension reinforcement.

For a circular section, "b_w" shall be taken as the diameter and "d" need not be taken less than the distance from the extreme compression fiber to the centroid of the longitudinal reinforcement in the opposite half of the member.

2. When the reaction in the direction of the applied shear introduces compression into the end region of the member, sections located less than a distance "d" from the face of the support may be designed for the same shear, "v_u" as that computed at a distance "d".

3. The shear stress carried by the concrete, "v_c", shall be calculated according to Article 1.5.35(B). When "v_u" exceeds "v_c", shear reinforcement shall be provided according to Article 1.5.35(C). Whenever applicable, the effects of torsion* shall be added.

*The design criteria for combined torsion and shear given in "Building Code Requirements for Reinforced Concrete – ACI 318-71" may be used.

B. Permissible Shear Stress

1. The shear stress carried by the concrete, "v_c", shall not exceed "$2\sqrt{f'_c}$" unless a more detailed analysis is made in accordance with (2) or (3). For members subjected to axial tension, "v_c" shall not exceed the value given in (4). For lightweight concrete, the provisions of (5) shall apply.

2. The shear stress carried by the concrete, "v_c" may be computed by:

$$v_c = 1.9\sqrt{f'_c} + 2500\rho_u \frac{V_u d}{M_u} \qquad (6\text{-}21)$$

but "v_c" shall not exceed "$3.5\sqrt{f'_c}$". The quantity "$V_u d/M_u$" shall not be taken greater than 1.0, where "M_u" is the applied design moment occurring simultaneously with "V_u" at the section considered.

3. For members subjected to axial compression, "v_c" may be computed by:

$$v_c = 2\left(1 + 0.0005\frac{N_u}{A_g}\right)\sqrt{f'_c} \qquad (6\text{-}22)$$

The quantity N_u/A_g shall be expressed in psi.

4. For members subjected to significant axial tension, shear reinforcement shall be designed to carry the total shear, unless a more detailed analysis is made using

$$v_c = 2\left(1 + 0.0002\frac{N_u}{A_s}\right)\sqrt{f'_c} \qquad (6\text{-}23)$$

where "N_u" is negative for tension. The quantity N_u/A_g shall be expressed in psi.

5. The provisions for shear stress, "v_c", carried by the concrete apply to normal weight concrete. When lightweight aggregate concretes are used, one of the following modifications shall apply:

a. When "f_{ct}" is specified, the shear stress "v_c" shall be modified by substituting

$$\frac{f_{ct}}{6.7} \text{ for } \sqrt{f'_c}$$

but the value of

$$\frac{f_{ct}}{6.7} \text{ used shall not exceed } \sqrt{f'_c}$$

b. When "f_{ct}" is not specified, the shear stress, "v_c", shall be multiplied by 0.75 for "all-lightweight" concrete, and 0.85 for "sand-lightweight" concrete. Linear interpolation may be used when partial sand replacement is used.

C. Design of Shear Reinforcement

1. Shear reinforcement shall conform to the general requirements of Article 1.5.10. When shear reinforcement perpendicular to the axis of the member is used, the required area shall be computed by:

$$A_v = \frac{(V_u - V_c)\, b_w\, s}{f_y} \tag{6-24}$$

2. When inclined stirrups or bent bars are used as shear reinforcement the following provisions apply:

   a. When inclined stirrups are used, the required area shall be computed by:

$$A_v = \frac{(V_u - V_c)\, b_w\, s}{f_y\,(\sin c + \cos a)} \tag{6-25}$$

   b. When shear reinforcement consists of a single bar or a single group of parallel bars, all bent up at the same distance from the support, the required area shall be computed by:

$$A_v = \frac{(V_u - V_c)\, b_w\, d}{f_y \sin a} \tag{6-26}$$

   in which $(v_u - v_c)$ shall not exceed $3\sqrt{f'_c}$.

   c. When shear reinforcement consists of a series of parallel bent-up bars or groups of parallel bent-up bars at different distances from the support, the required area shall be computed by (a).

   d. Only the center three-fourths of the inclined portion of any longitudinal bar that is bent shall be considered effective for shear reinforcement.

3. Where more than one type of shear reinforcement is used to reinforce the same portion of the member, the required area shall be computed as the sum for the various types separately. In such computations, v_c shall be included only once.
4. When $(v_u - v_c)$ exceeds $4\sqrt{f'_c}$, the maximum spacings given in Article 1.5.10(C) shall be reduced by one-half.
5. The value of $(v_u - v_c)$ shall not exceed $8\sqrt{f'_c}$.

D. Shear-Friction

1. These provisions apply where it is inappropriate to consider shear as a measure of diagonal tension, and particularly in design of reinforcing details for precast concrete structural elements.
2. A crack shall be assumed to occur along the shear path. Relative displacement shall be considered resisted by friction maintained by shear-friction reinforcement across the crack. This reinforcement shall be approximately perpendicular to the assumed crack.
3. The shear stress, "v_u", shall not exceed $0.2f'_c$ nor 800 psi.
4. The required area of reinforcement, "A_{vf}", shall be computed by

$$A_{vf} = \frac{V_u}{\phi f_y \mu} \tag{6-27}$$

The coefficient of friction, "μ", shall be 1.4 for concrete cast monolithically, 1.0 for concrete placed against hardened concrete, and 0.7 for concrete placed against as-rolled structural steel.

5. Direct tension across the assumed crack shall be provided by additional reinforcement.
6. The shear-friction reinforcement shall be well distributed across the assumed crack and shall be adequately anchored on both sides by embedment, hooks, or welding to special devices.
7. When shear is transferred between concrete placed against hardened concrete, the interface shall be rough, clean, and free of laitance with a full amplitude of approximately ¼ in. When shear is transferred between as-rolled steel and concrete, the steel shall be clean and without paint.

E. Horizontal Shear Design for Composite Concrete Flexural Members

1. In a composite member, full transfer of the shear forces shall be assured at the interfaces of the separate components.
2. Full transfer of horizontal shear forces may be assumed when all of the following are satisfied: (a) the contact surfaces are clean and intentionally roughened, (b) minimum ties are provided in accordance with paragraph (6), (c) web members are designed to resist the entire vertical shear, and (d) all shear reinforcement is anchored into all intersecting components.

   When all of the above are not satisfied, horizontal shear shall be fully investigated.
3. The horizontal shear stress, "v_{dh}" may be computed at any cross section as

$$v_{dh} = \frac{V_u}{\phi b_v d} \tag{6-28}$$

in which "d" is for the entire composite section. Alternatively, in any segment not exceeding one-tenth of the span, the actual change in compressive or tensile force to be transferred may be computed, and provisions made to transfer that force as horizontal shear to the supporting element.

4. The horizontal shear may be transferred at contact surfaces using the permissible horizontal shear stress, "v_h", stated below.

a. When ties are not provided, but the contact surfaces are clean and intentionally roughened, permissible "$v_h = 80$ psi."

b. When the minimum tie requirements of paragraph (6) are provided and the contact surfaces are clean but not intentionally roughened, permissible "$v_h = 80$ psi."

c. When the minimum tie requirements of paragraph (6) are provided and the contact surfaces are clean and intentionally roughened, permissible "$v_h = 350$ psi.".

d. When "v_{dh}" exceeds 350 psi, design for horizontal shear shall be made in accordance with Article 1.5.35(D).

5. When tension exists perpendicular to any surface, shear transfer by contact may be assumed only when the minimum tie requirements of paragraph (6) are satisfied.

6. Ties for horizontal shear

a. When the vertical bars or extended stirrups are used to transfer horizontal shear, the tie area shall not be less than that required by Article 1.5.10(A)(2) and the spacing shall not exceed four times the least dimension of the supported element nor 24 in.

b. Ties for horizontal shear may consist of single bars, multiple leg stirrups, or the vertical legs of welded wire fabric. All ties shall be adequately anchored into the components by embedment or hooks.

7. Measure of roughness

Internal roughness may be assumed only when the contact surface is roughened, clean, and free of laitance. Roughness shall have a full amplitude of approximately ¼ in.

F. Special Provisions for Slabs and Footings

1. The shear capacity of slabs and footings in the vicinity of concentrated loads or reactions shall be governed by the more severe of two conditions:

a. The slab or footing acting as a wide beam, with a critical section extending in a plane across the entire width and located at a distance "d" from the face of the concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with Articles 1.5.35(A) through (C).

b. Two-way action for the slab or footing, with a critical section perpendicular to the plane of the slab and located so that its periphery is a minimum and approaches no closer than "d/2" to the periphery of the concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with paragraphs (2) and (3).

2. The peripheral shear stress shall be computed by:

$$v_u = \frac{V_u}{\phi b_o d} \qquad (6\text{-}29)$$

in which "V_u" and b_o are taken at the critical section defined in (b). The peripheral shear stress, "v_u" shall not exceed the shear stress carried by the concrete, $v_c = 4\sqrt{f'_c}$ unless shear reinforcement is provided in accordance with (3), in which case "v_u" shall not exceed $6\sqrt{f'_c}$.

3. Shear reinforcement consisting of bars or wires anchored in accordance with Article 1.5.21 may be provided. For design of such shear reinforcement, shear stresses shall be investigated at the critical section defined in (b) and at successive sections more distant from the support; and the shear stress, "v_c" carried by the concrete at any section shall not exceed $2\sqrt{f'_c}$. Where "v_u" exceeds "v_c", the shear reinforcement shall be provided according to Article 1.5.35(C).

1.5.36 – PERMISSIBLE BEARING STRESS

A. Bearing stress in concrete on loaded area, "f_b", shall not exceed "$0.85\phi f'_c$, except as provided below.

B. When the supporting surface is wider on all sides than the loaded area, the permissible bearing stress on the loaded area may be multiplied by

$$\sqrt{\frac{A_2}{A_1}} \quad \text{but not more than 2.}$$

C. When the supporting surface is sloped or stepped, "A_2" may be taken as the area of the lower base of the largest frustrum of a right pyramid or cone contained wholly within the support and having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal.

D. When the loaded area is subjected to high edge stresses due to deflection or eccentric loading, the permissible bearing stress on the loaded area shall be multiplied by a factor of 0.75. The requirements of (B) and (C) shall also apply.

1.5.37 – SERVICEABILITY REQUIREMENTS

A. Application

For flexural members designed with reference to load factors and strengths by LOAD FACTOR DESIGN, stresses at service load shall be limited to satisfy the requirements for fatigue in Article 1.5.38, and the requirements for distribution of reinforcement in Article 1.5.39. The requirements for deflection control in Article 1.5.40 shall also apply.

B. **Service Load Stresses**

For investigation of service load stresses to satisfy the requirements of Articles 1.5.38 and 1.5.39, the straight-line theory of stress and strain in flexure shall be used and the assumptions given in Article 1.5.27 shall apply.

1.5.38 – FATIGUE STRESS LIMITS

A. **Concrete**

The maximum compressive stress in the concrete shall not exceed "$0.5f'_c$ at sections where stress reversals occur caused by live load plus impact at service load. This stress limit shall not apply to concrete deck slabs.

B. **Reinforcement**

The range between a maximum and minimum stress in straight reinforcement caused by live load plus impact at service load shall not exceed 21,000 psi*. Bends in primary reinforcement shall be avoided in regions of high stress range.

1.5.39 – DISTRIBUTION OF FLEXURAL REINFORCEMENT

Tension reinforcement shall be well distributed in the zones of maximum tension. When the design yield strength, "f_y", for tension reinforcement exceeds 40,000 psi, cross sections of maximum positive and negative moment shall be so proportioned that the calculated stress in the reinforcement at service load, "f_s", in kips per sq. in., does not exceed the value computed by:

$$f_s = \frac{z}{\sqrt[3]{d_c A}} \qquad (6\text{-}30)$$

but f_s shall not be greater than $0.6f_y$, where

A = effective tension area of concrete surrounding the main tension reinforcing bars and having the same centroid as that reinforcement, divided by the number of bars, sq. in. When the main reinforcement consists of several bar sizes the number of bars shall be computed as the total steel area divided by the area of the largest bar used.

d_c = thickness of concrete cover measured from the extreme tension fiber to the center of the bar located closest thereto, in.

The quantity "z" in Eq. (6-30) shall not exceed 170 kips per in. for members in moderate exposure conditions and 130 kips per in. for members in severe exposure conditions. Where members are exposed to very aggressive exposure or corrosive environments, such as deicer chemicals, the denseness and nonporosity of the protecting concrete should be considered, or other protection, such as a waterproof protecting system, should be provided in addition to satisfying Eq. (6-30).

*Applicable primarily to concrete deck slabs and short span bridges where the dead lead to total load ratio is less than approximately 0.25.

1.5.40 – CONTROL OF DEFLECTIONS

A. General

Flexural members of bridge structures shall be designed to have adequate stiffness to limit deflections or any deformations which may adversely affect the strength or serviceability of the structure at service load.

B. Superstructure Depth Limitations

The minimum thicknesses stipulated in Table 1.5.40 are recommended unless computation of deflection indicates that lesses thickness may be used without adverse effects.

TABLE 1.5.40 – Recommended Minimum Thickness for Constant Depth Members*

Superstructure Type	Minimum Thickness** (in feet)
Bridge slabs with main reinforcement parallel or perpendicular to traffic	$\frac{S + 10}{30}$ but less than 0.542
T-Girders	$\frac{S + 9}{18}$
Box-Girders	$\frac{S + 10}{20}$

*When variable depth members are used, table values may be adjusted to account for change in relative stiffness of positive and negative moment sections.
**Recommended values for continuous spans; simple spans should have about 10 percent greater thickness.
S = span length as defined in Article 1.5.23(F), in feet.

Section 6 — PRESTRESSED CONCRETE

1.6.1 — GENERAL

The specifications of this section are intended for design of prestressed concrete bridge members. Members designed as reinforced concrete, except for a percentage of tensile steel stressed to improve service behavior, shall conform to the applicable specifications of Section 5.

Exceptionally long span or unusual structures require detailed consideration of effects which under this section may have been assigned arbitrary values.

1.6.2 — NOTATION

A_s	=	area of non-prestressed tension reinforcement.
A'_s	=	area of compression reinforcement.
A^*_s	=	area of prestressing steel.
A_{sf}	=	steel area required to develop the ultimate compressive strength of the overhanging portions of the flange.
A_{sr}	=	steel area required to develop the ultimate compressive strength of the web of a flanged section.
A_v	=	area of web reinforcement.
b	=	width of flange of flanged member or width of rectangular member.
b'	=	width of a web of flanged member.
CR_c	=	loss of prestress due to creep of concrete.
CR_s	=	loss of prestress due to relaxation of prestressing steel.
CR_{sp}	=	loss of prestress due to relaxation of post-tensioning steel.
D	=	effect of dead load. (Article 1.6.5)
D	=	nominal diameter of prestressing steel.
d	=	distance from extreme compressive fiber to centroid of the prestressing force.
ES	=	elastic shortening loss.
e	=	base of naperian logarithms.
f_{cd}	=	average concrete compressive stress at the c.g. of the prestressing steel under full dead load.
f_{cr}	=	average concrete stress at the c.g. of the prestressing steel at time of release.
f'_c	=	compressive strength of concrete at 28 days.
f'_{ci}	=	compressive strength of concrete at time of initial prestress.
Δf_s	=	total prestress loss, excluding friction.
f_{se}	=	effective steel prestress after losses.
f^*_{su}	=	average stress in prestressing steel at ultimate load.
f'_s	=	ultimate strength of prestressing steel.
f_{sy}	=	yield strength of non-prestressed conventional reinforcement in tension.
f'_y	=	yield strength of non-prestressed conventional reinforcement in compression.
f^*_y	=	yield point stress of prestressing steel.
I	=	moment of inertia about the centroid of the cross section.
I	=	impact load. (Article 1.6.5)

j = ratio of distance between centroid of compression and centroid of tension to the depth d.
K = friction wobble coefficient per foot of prestressing steel.
L = effect of design live load. (Article 1.6.5)
L = length of prestressing steel element from jack end to point x.
M_u = ultimate flexural strength.
p = As/bd, ratio of non-prestressed tension reinforcement.
p* = A_s^*/bd, ratio of prestressing steel.
P' = A_s'/bd, ratio of compression reinforcement.
Q = statical movement of cross sectional area, above or below the level being investigated for shear, about the centroid.
SH = concrete shrinkage loss.
s = longitudinal spacing of the web reinforcement.
t = average thickness of the flange of a flanged member.
T_o = steel stress at jacking end.
T_x = steel stress at any point x.
v = ultimate horizontal shear stress.
V_c = shear carried by concrete.
V_u = shear due to ultimate load and effect of prestressing.
μ = friction curvature coefficient.
α = total angular change of prestressing steel profile in radians from jacking end to point x.

1.6.3 – DESIGN THEORY

Members shall meet the ultimate strength and allowable stress requirements as specified.

Design shall be based on ultimate strength and behavior at service conditions for all load stages that may be critical during the life of the structure from the time of prestressing.

1.6.4 – BASIC ASSUMPTIONS

The following assumptions are made for design purposes:

1. Strains vary linearly over the depth of the member throughout the entire load range.
2. Before cracking, stress is linearly proportional to strain.
3. After cracking, tension in the concrete is neglected.

1.6.5–LOAD FACTORS

The computed ultimate capacity shall not be less than the largest value from load factor design in Section 1.2.22.

The following capacity reduction factors shall be used:

For factory produced precast prestressed concrete members ϕ = 1.0
For post-tensioned cast in place concrete members ϕ = 0.95
For shear ϕ = 0.90

1.6.6 – ALLOWABLE STRESSES*****

The design of precast prestressed members ordinarily shall be based on $f'_c = 5000$ psi. An increase to 6000 psi is permissible where, in the Engineer's judgment, it is reasonable to expect that this strength will be obtained consistently. Still higher concrete strengths may be considered on an individual area basis. In such cases, the engineer shall satisfy himself completely that the controls over materials and fabrication procedures will provide the required strengths, the provisions of this chapter are equally applicable to prestressed concrete structures or components designed with lower concrete strengths.

A. Prestressing Steel

Temporary stress before loss due to creep and shrinkage . . . . $.0.07f'_s$
Stress at service load (*) after losses . . . . $0.08f^*_y$

(Overstressing to $0.75f'_s$ for pretensioned members and 0.80 for post-tensioned members may be permitted for short periods of time provided that the stress, after transfer to the concrete in pretensioning or seating of anchorage in post-tensioning, does not exceed $0.70f'_s$)

B. Concrete

1. Temporary stresses before losses due to creep and shrinkage

Compression

Pretensioned members . . . . $0.60f'_{ci}$
Post-tensioned members . . . . $0.55f'_{ci}$

Tension

Precompressed Tensile Zone. (An area that is initially under compression during prestressing, but may be in tension under load). No temporary allowable stresses are specified. See Article 1.6.6(B)(2) for allowable stresses after losses.

Other Areas

In tension areas with no pretensioned strands, grouted post-tensioned strands or nonprestressed reinforcement.

$$200 \text{ psi or } 3\sqrt{f'_{ci}}$$

Where the calculated tensile stress exceeds this value, bonded reinforcement shall be provided to resist the total tension force in the concrete computed on the assumption of an uncracked section. The maximum tensile stress shall not exceed

$$7.5\sqrt{f'_{ci}}$$

*Service load consists of all loads contained in Section 1.2.1 but does not include overload provisions.

2. Stress at service load after losses have occurred:

Compression . . . . $0.40f'_c$

Tension in the precompressed tensile zone

(a) For members with bonded reinforcement . . . . $6\sqrt{f'_c}$

For severe corrosive exposure conditions, such as coastal areas . . $3\sqrt{f'_c}$

(b) For members without bonded reinforcement 0

Tension in Other Areas

Tension in other areas is limited by the allowable temporary stresses specified in Article 1.6.6(B)(1).

3. Cracking stress (refer to Article 1.6.10)

Modulus of rupture from tests or if not available:

For normal weight concrete . . . . $7.5\sqrt{f'_c}$

For sand-lightweight concrete . . . . $.6.3\sqrt{f'_c}$

For all other lightweight concrete . . . . $5.5\sqrt{f'_c}$

4. Anchorage Bearing Stress:

Post-tensioned anchorage at service load. . . . . 3000 psi

(But not to exceed $0.9f'_{ci}$)

1.6.7 – LOSS OF PRESTRESS

A. Friction Losses

Friction losses in post-tensioned steel shall be based on experimentally determined wobble and curvature coefficients, and shall be verified during stressing operations. The values of coefficients assumed for design, and the acceptable ranges of jacking forces and steel elongation shall be shown on the plans. These friction losses shall be calculated as follows:

$$T_0 = T_x e^{(KL + \mu \alpha)}$$

When ($KL + \mu\alpha$) is not greater than 0.3, the following equation may be used:

$$T_0 = T_x (1 + KL + \mu \alpha)$$

The following values for K and μ may be used when experimental data for the materials used are not available:

Type of Steel	Type of Duct	K	μ
Wire or Ungalvanized Strand	Bright Metal Sheathing	0.0020	0.30
	Galvanized Metal Sheathing	0.0015	0.25
	Greased or Asphalt-Coated and Wrapped	0.0020	0.30
	Galvanized Rigid	0.0002	0.25
High-Strength Bars	Bright Metal Sheathing	0.0003	0.20
	Galvanized Metal Sheathing	0.0002	0.15

Friction losses occur prior to anchoring but should be estimated for design and checked during stressing operations. Rigid ducts shall have sufficient strength to maintain their correct alignment without visible wobble during placement of concrete. Rigid ducts may be fabricated with either welded or interlocked seams. Galvanizing of the welded seam will not be required.

B. Prestress Losses

1. Loss of prestress due to all causes, excluding friction, may be determined by the following method.* The method is based on normal weight concrete and one of the following types of prestressing steel: 250 or 270 ksi, (1724 or 1862 MPa), seven-wire, stress-relieved strand; 240 ksi (1655 MPa) stress-relieved wires; or 145 to 160 ksi (1000 to 1103 MPa) smooth or deformed bars. For data regarding the properties and effects of lighweight aggregate concrete and low-relaxation tendons, refer to documented tests or see authorized suppliers.

TOTAL LOSS

$$\Delta f_s = SH + ES + CR_c + CR_s$$

where Δf_s = total loss excluding friction in psi. (MPa)
SH = loss due to concrete shrinkage in psi. (MPa)
ES = loss due to elastic shortening in psi. (MPa)
CR_c = loss due to creep of concrete in psi. (MPa)
CR_s = loss due to relaxation of prestressing steel in psi. (MPa)

(a) SHRINKAGE

Pretensioned Members

$$SH = 17{,}000 - 150\ RH$$
$$= (117.21 - 1.034\ RH) \text{ in MPa}$$

Post-tensioned Members

$$SH = 0.80\ (17{,}000 - 150\ RH)$$
$$= 0.80\ \ 117.21 - 1.034\ RH \text{ in MPa}$$

where RH = average annual ambient relative humidity in percent
(See Figure 1.6.7)

*Should more exact prestress losses be desired, data representing the materials to be used, the methods of curing, the ambient service condition and any pertinent structural details should be determined for use in accordance with a method of calculating prestress losses that is supported by appropriate research data.

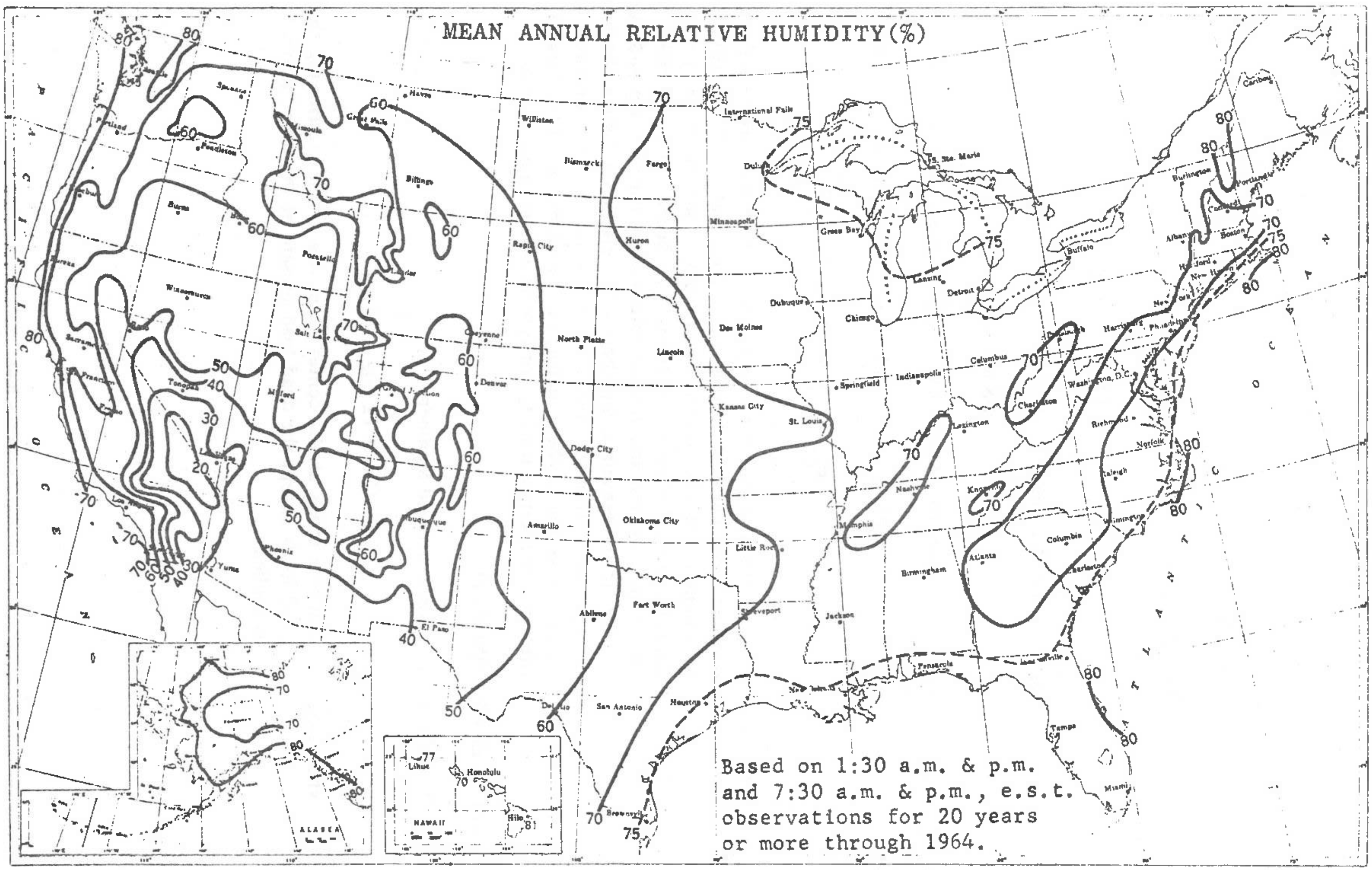

MEAN ANNUAL RELATIVE HUMIDITY (%)
Based on 1:30 a.m. & p.m.
and 7:30 a.m. & p.m., e.s.t.
observations for 20 years
or more through 1964.
ALASKA
HAWAII

Figure 1.6.7

(b) ELASTIC SHORTENING

Pretensioned Members

$$ES = \frac{E_s}{E_{ci}} f_{cir}$$

Post-tensioned Members**

$$ES = 0.5 \frac{E_s}{E_{ci}} f_{cir}$$

where E_s = modulus of elasticity of steel which can be assumed to be 28 x 10^6 psi (.193 x 10^6 MPa).

E_{ci} = modulus of elasticity of concrete at transfer of stress (which can be calculated from:

$$E_{ci} = 33 \sqrt[3]{w^2} \sqrt{f'_{ci}}$$

f_{cir} = concrete stress at the center of gravity of the prestressing steel due to prestressing force and dead load of beam immediately after transfer. f_{cir} shall be computed at the section or sections of maximum moment. (At this stage, the intitial stress in the tendon has been reduced by elastic shortening of the concrete and tendon relaxation during placing and curing the concrete for pretensioned members, or by elastic shortening of the concrete and tendon friction for post-tensioned members. The reductions to initial tendon stress due to these factors can be estimated, or the reduced tendon stress can be taken as 0.63 f_s' for typical pretensioned members.)

(c) CREEP OF CONCRETE

Pretensioned and Post-tensioned Members

$$CR_c = 12\ f_{cir} - 7\ f_{cds}$$

where f_{cds} = concrete stress at the center of gravity of the prestressing steel due to all dead loads except the dead load present at the time the prestressing force is applied.

**Certain tensioning procedures may alter the elastic shortening losses.

(d) RELAXATION OF PRESTRESSING STEEL***

Stress Relieved Strand

$$CR_s = 20{,}000 + 6250\ (\%\ Guts^* - 70)\ /5. - 0.4\ ES - 0.2\ (SH + CR_c)$$
$$= 137.9 + 43\ (\%\ Guts^* - 70)\ /5. - .00275\ ES - .0014\ (SH + CR_c) \text{ in MPa}$$

Stabilized Strand

$$CR_s = 5270 + 1480\ (\%\ Guts^* - 70)\ /5. - 0.4\ ES - 0.2\ (SH + CR_c)$$
$$= 36.4 + 10.2\ (\%\ Guts^* - 70)\ /5. - .00275\ ES - .0014\ (SH + CR_c) \text{ in MPa}$$

*% Guts is ratio of Jacking Force to guaranteed ultimate tensile strength.

Post-tensioned Members

250 to 270 ksi Strand (1724 to 1862 MPa)

$$CR_s = 20{,}000 - 0.3\ FR - 0.4\ ES - 0.2\ (SH + CR_c)$$
$$= (137.9 - .0021\ FR - .00275\ ES - .0014\ (SH + CR_c) \text{ in MPa}$$

240 ksi Wire (1655 MPa)

$$CR_s = 18{,}000 - 0.3\ FR - 0.4\ ES - 0.2\ (SH + CR_c)$$
$$= (124.10 - .0021\ FR - .00275\ ES - .0014\ (SH + CR_c) \text{ in MPa}$$

145 to 160 ksi Bars (1000 to 1103 MPa)

$$CR_s = 3000\ (20.68 \text{ MPa})$$

where FR = friction loss stress reduction in psi (MPa) below the level of 0.70 f_s' at the point under consideration, computed according to Article 1.6.7(A).

ES, SH, and CR_c = appropriate values as determined for either pretensioned or post-tensioned members.

2. In lieu of the preceding method, the following estimates of total losses may be used for prestressed members or structures of usual design. These loss values are based on use of normal weight concrete, normal prestress levels, and average exposure conditions. For exceptionally long spans, or for unusual designs, the method in Section 1.6.7(B)(1) or a more exact method shall be used.

***The relaxation losses are based on an initial stress of 0.70 f_s'.

TYPE OF PRESTRESSING STEEL	TOTAL LOSS	
	$f_c' = 4{,}000$ psi (27.58 MPa)	$f_c' = 5{,}000$ psi (34.47 MPa)
PRETENSIONING		
Strand	–	45,000 psi (310.26 MPa)
POST-TENSIONING****		
Wire or Strand	32,000 psi (220.63 MPa)	33,000 psi (227.53 MPa)
Bars	22,000 psi (151.68 MPa)	23,000 psi (158.58 MPa)

****Losses due to friction are excluded. Friction losses should be computed according to Article 1.6.7(A).

COMMENTARY ART. 1.6.7(B)

Introduction

The subject revision to Section 1.6.7(B)—Prestress Losses in the 1973 AASHTO Specifications for Highway Bridges is based largely on consideration of research results (1,2)* made available subsequent to the adoption of the current specification, and on a proposed revision of the specifications with respect to losses in post-tensioned bridges presented at the 1972 AASHTO regional Bridge Committee meetings (3).

Precise determination of prestress losses in a given situation is very complex and requires detailed information on the materials to be used, the methods of curing, the ambient exposure conditions, and other detailed construction information that usually is not available to designers. For large and/or special structures such as precast or cast-in-place segmental cantilever bridges it may be appropriate or necessary to obtain this information, in order to maintain control of the geometry of the bridge during construction. For most structures, precise calculation of losses is not necessary, and it is possible to calculate reasonably accurate and satisfactory approximations of prestress losses for general design use.

The necessary level of precision in calculation of losses is subject ot both technical and subjective evaluation. It has long been recognized that the value used for loss of prestress does not affect the ultimate strength of a bridge, but is important with respect to serviceability characteristics such as deflection, camber, stresses, and cracking (4). The value used for prestress loss also has an influence on economy in that higher values of losses require somewhat more

*Numbers in parenthesis refer to references listed at the end of the commentary.

prestressing steel. However, it has been shown that both the serviceability and economy of prestressed bridges are relatively insensitive to sizeable variations in the value assumed for prestress losses in typical designs (5). None-the-less, there is disagreement between agencies and designers as to what degree of precision is necessary in the calculation of losses, and there is some ambiguity in the research results on this subject, even when attempts are made to relate the research to a common base.

As a result of the above considerations, the subject revision of the AASHTO prestress loss specifications provides for three optional methods of loss calculation as follows:

1) The lump sum loss values in Section 1.6.7(B)(2).
2) The more precise and detailed provisions of Section 1.6.7(B)(1).
3) The use of still more exact procedures for loss calculation under the provisions of the footnote on page one of the proposed revision.

A detailed commentary of considerations related to the first two approaches is presented below. Due to the broad nature of calculations and procedures that might be involved in calculating losses under the provisions of the footnote as noted under the third option above, the discussion of this approach is not included in the commentary.

SECTION 1.6.7(B)(1)

As compared to the existing specifications, the revision adds consideration of three types of tendons used for post-tensioned structures: 250 or 270 ksi (1724 or 1862 MPa), seven-wire, stress-relieved strand; 240 ksi (1655 MPa), stress-relieved wires; or 145 to 160 ksi (1000 to 1103 MPa) smooth or deformed bars.

The total loss is specified as the direct addition of losses due to shrinkage, elastic shortening, concrete creep, and steel relaxation. Addition of these losses in this manner is an approximation because the losses are to some extent inter-related. As will be discussed below under Section 1.6.7(B)(1)(d), Relaxation of Prestressing Steel, the inter-relationship of losses is considered in the formulas given for relaxation of prestressing steel.

The loss format of addition of four loss components is similar to the procedure outlined in the "Criteria for Prestressed Concrete Bridges" published by the U.S. Department of Commerce, Bureau of Public Roads in 1954 (6). However, the values related to each of the four components in the loss equation have been modified to more accurately reflect the research that has occurred in each area since 1954.

SECTION 1.6.7(B)(1)(a) – SHRINKAGE

The two equations for loss due to concrete shrinkage give similar shrinkage losses to those included in the table in the current specifications. However, the equations have the advantage of eliminating the abrupt jump in shrinkage loss between the three humidity ranges. A map of the United States showing the average annual ambient relative humidity is presented in Fig. 1 (7). This map is to be used as the basis for determining average annual ambient relative humidity for use in calculating shrinkage loss.

The ultimate shrinkage strain of small specimens (8, 9) was assumed to be 550 x 10^{-6} at 50 percent relative humidity. This is an average value for shrinkage strain, some concretes may inherently shrink somewhat more and others somewhat less. Research indicates that shrinkage strains are reduced 10 to 40 percent by steam curing (10).

Research (11) has shown that shrinkage is related to the size or thickness of the member. This factor is usually expressed as the Volume/Surface ratio of the member. In development of the subject equations an average Volume/Surface ratio of 4 inches (.102 m) was assumed, and the ultimate shrinkage strain was reduced by a factor of 0.77. The volume/surface ratio is effectively one-half the average member thickness and it therefore represents the distance from the drying surface to the center of the member.

Shrinkage loss is also a function of the average ambient relative humidity of the environment. The amount of shrinkage reduces as the humidity increases. In some cases concrete stored under water has been observed to expand slightly. In developing the existing AASHTO specifications, humidity shrinkage factors were assumed as follows (12):

Average Ambient Relative Humidity Percent	Humidity Shrinkage Coefficient
100-75	0.3
75-25	1.0
25-0	1.3

Based on the above assumptions and on assumed modulus of elasticity for steel of 28 x 10^6 psi (.193 x 10^6 MPa), shrinkage losses were calculated for the table in the existing specifications as the product of the four interior columns in the table following:

Humidity	Shrinkage Strain	Steel Modulus-psi (MPa)	Volume Surface Factor	Humidity Factor	Shrinkage Loss-psi (MPa)
100-75	550×10^{-6}	28×10^6 $(.193 \times 10^6)$	0.77	.3	3,540 (24.41)
75-25	550×10^{-6}	28×10^6 $(.193 \times 10^6)$	0.77	1.0	11,800 (81.36)
25-0	550×10^{-6}	28×10^6 $(.193 \times 10^6)$	0.77	1.3	15,250 (105.15)

These shrinkage loss values were rounded to 5,000 10,000 and 15,000 for use in the existing (1973 edition) AASHTO specifications.

Multiplication of the shrinkage loss equation for post-tensioned design by the factor 0.8 is in consideration of the fact that post-tensioning is not accomplished until the concrete has reached an age of at least seven days at which time 20 percent of the shrinkage is assumed to have occurred. This is a very conservative assumption on time of stressing relative to concrete age for most bridges. A general curve relating shrinkage (or creep) to time is presented in Fig. C1(13). Use of some other reduction factor in the equation for shrinkage loss for post-tensioned applications might be considered if the designer has knowledge that the elapsed time from concrete placement to stressing will be significantly greater than seven days.

In the discussion relative to the subject revision of the loss specification, some expressed the view that the basic ultimate shrinkage strain of 550×10^{-6} was too low, while others suggested that it was too high, and still others felt the value to be satisfactory. This value was retained, and the tabular values in the existing specification were replaced by the two linear equations for reasons described above.

1.6.7(B)(1)(b) Elastic Shortening

The table of losses due to elastic shortening in the current AASHTO Specification has been deleted in the revision, and the formula for elastic shortening loss has been generalized by including the ratio of the modulus of elasticity of steel to the modulus of concrete at release. The formula in the current specifications uses a value of 7 for the ratio of the moduli of elasticity.

A significant revision is in the definition of the concrete stress to be used in calculating elastic shortening loss. In the current specifications, this value is given the nomenclature "f_{cr} where f_{cr} = average concrete stress at the center of gravity of the prestressing steel at time of release. The reason for using the average value of concrete stress (along the length of the beam) was to avoid the necessity of precise calculation of stresses at each point in a beam before a value of the elastic shortening loss could be obtained. It was intended that the average stress value could be estimated or obtained

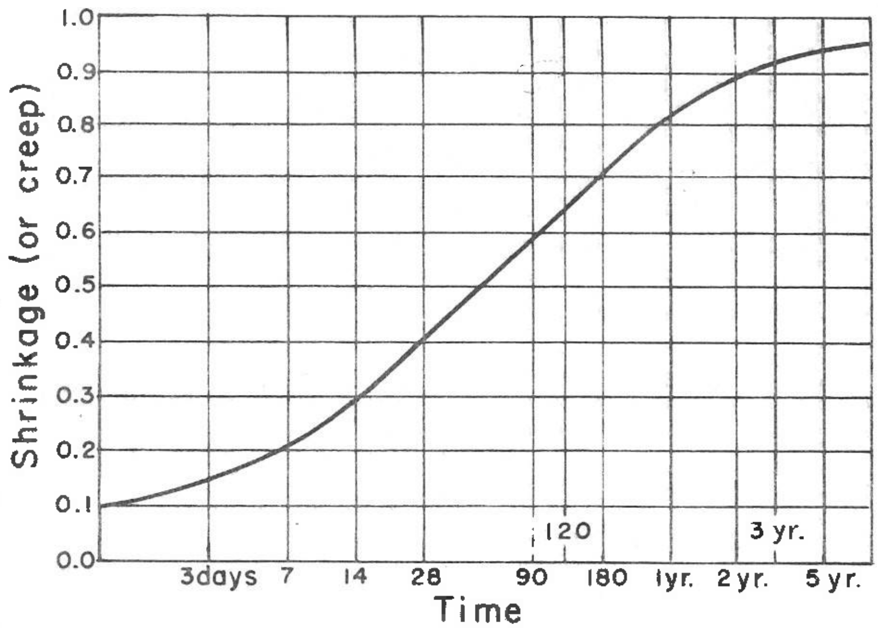


FIG. C1—Shrinkage or creep vs. time

from the table values so that it would not be necessary for designers to use a trial and error process of design iterations in order to obtain a value for the elastic shortening loss. Use of average stress along the center of gravity of the steel is a conservative approximation since the average stresses and related elastic shortening is greater than the stress and elastic shortening at the points of maximum moment.

The current specifications were criticized for use of average concrete stresses in calculating both elastic shortening and concrete creep losses. It was pointed out that this process was theoretically incorrect and that the actual elastic shortening loss (and creep loss) is dependent on the concrete stress at the section under consideration. In development of the proposed revision, it was agreed that the definition of stress for use in elastic shortening loss calculations would be more precise if related to the section under consideration, and it was acknowledged that this increase in precision might require a trial and error design process. To minimize the amount of additional design calculation, the final specification wording related the concrete stress, f_{cir}, for use in the elastic shortening formula to "...the section or sections of maximum moment," rather than the section under consideration. For a simple span precast design, this change results in a requirement of loss calculation at one point only, whereas relating the definition of f_{cir} to the section under consideration would have required a separate loss calculation at each point where stresses were investigated. For simple span pre-tensioned designs, it should be understood that the intent of the proposed revision as written is that elastic shortening loss (and concrete creep loss) is to be calculated only at the point of maximum moment, and that this value is to be used as the loss value throughout the length of the beam. For continuous post-tensioned bridges, the intent of this portion of the specification is that the loss value be calculated at each controlling point of maximum positive or negative moment along the length of the structure.

It should be noted that the stress in the tendon for pretensioned products for use in calculating f_{cir} should be reduced from the initial stress value by an amount to allow for elastic shortening of the concrete and for steel relaxation prior to release of the tendon force to the member.* Recognizing that this further complicates the iterative process for the designer, an approximate value of the reduced tendon stress of 0.63 f_s' is provided. This approximation related to tendons stressed initially to 0.70 f_s'. Note that initial overstress is permitted so that the initial tendon stress after transfer to concrete is not greater than 0.70 f_s'; this requires more accurate estimates for initial losses to ensure the correct tendon stress at transfer. It is more complicated to obtain the reduced tendon force for calculation of f_{cir} for post-tensioned applications because the reduction depends on both elastic shortening and tendon friction. In addition, the use of initial tendon stresses in excess of $0.70f_s'$ is quite common in post-tensioned applications to compensate for anchor seating loss. If the designer is willing to sacrifice some accuracy for convenience in calculation; a value of 0.65 f_s' might be used in calculating f_{cir} for post-tensioned designs. For structures utilizing long tendons where the friction losses become sizeable, this approximation would become quite conservative.

*See Figures C2 and C3 for steel relaxation losses for constant strain specimens of stress relieved and stabilized 270K(1.2MN) strand.

For structures or elements where the concrete strength is determined on the basis of the nominal minimums in the specifications, and where full utilization of the allowable compressive stresses does not occur during the life of the structure, the lump sum loss values will be somewhat conservative. Such conditions often occur when an over-size section is used for a short span due to the esthetic consideration of matching the depth of section used on a longer adjacent span.

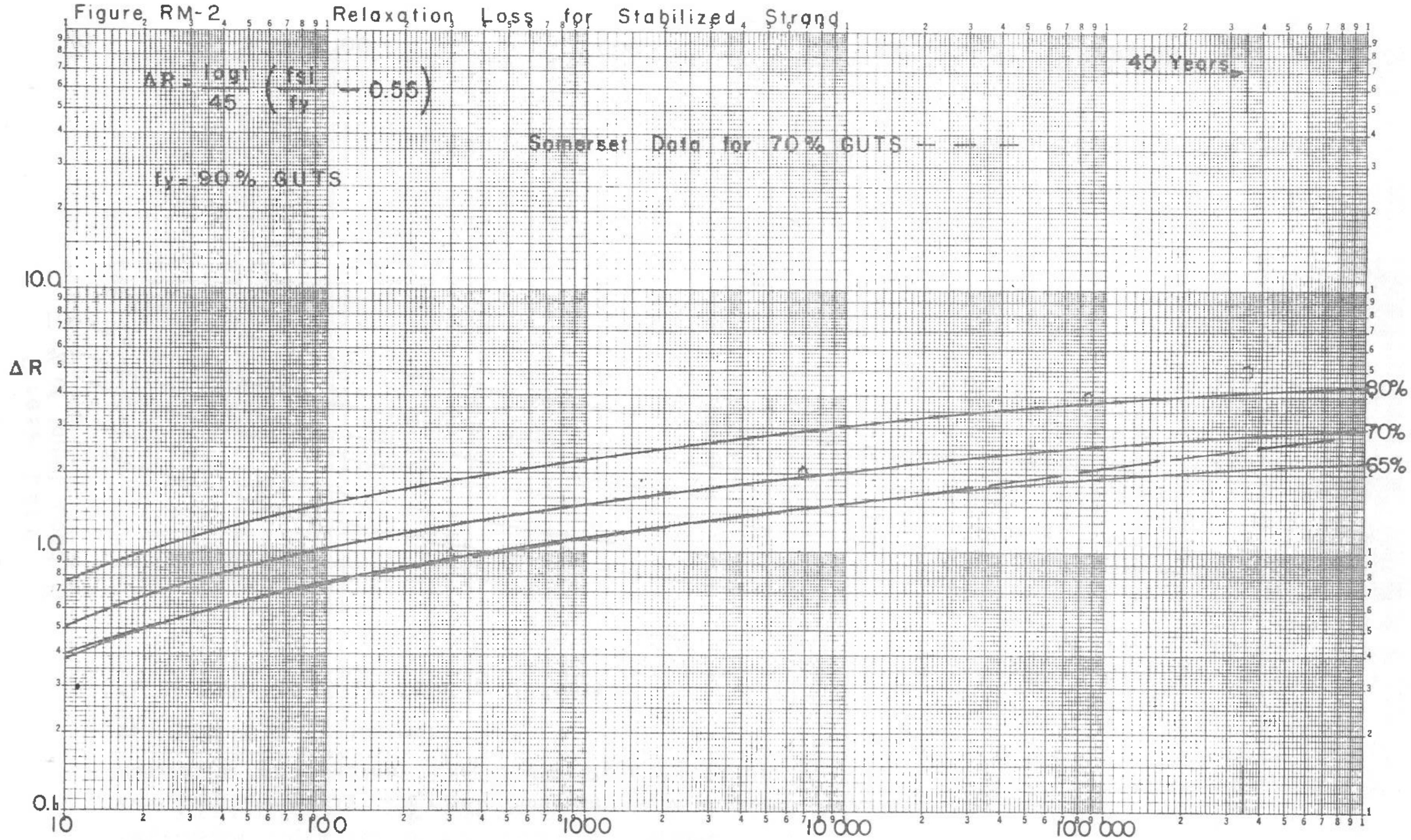


Figure RM-2
Relaxation Loss for Stabilized Strand
$\Delta R = \frac{\log t}{45}\left(\frac{f_{si}}{f_y} - 0.55\right)$
f_y = 90% GUTS
Somerset Data for 70% GUTS — — —
40 Years
80%
70%
65%
ΔR
10.0
1.0
0.1
10
100
1000
10 000
100 000
Time —— Hours

Figure C3

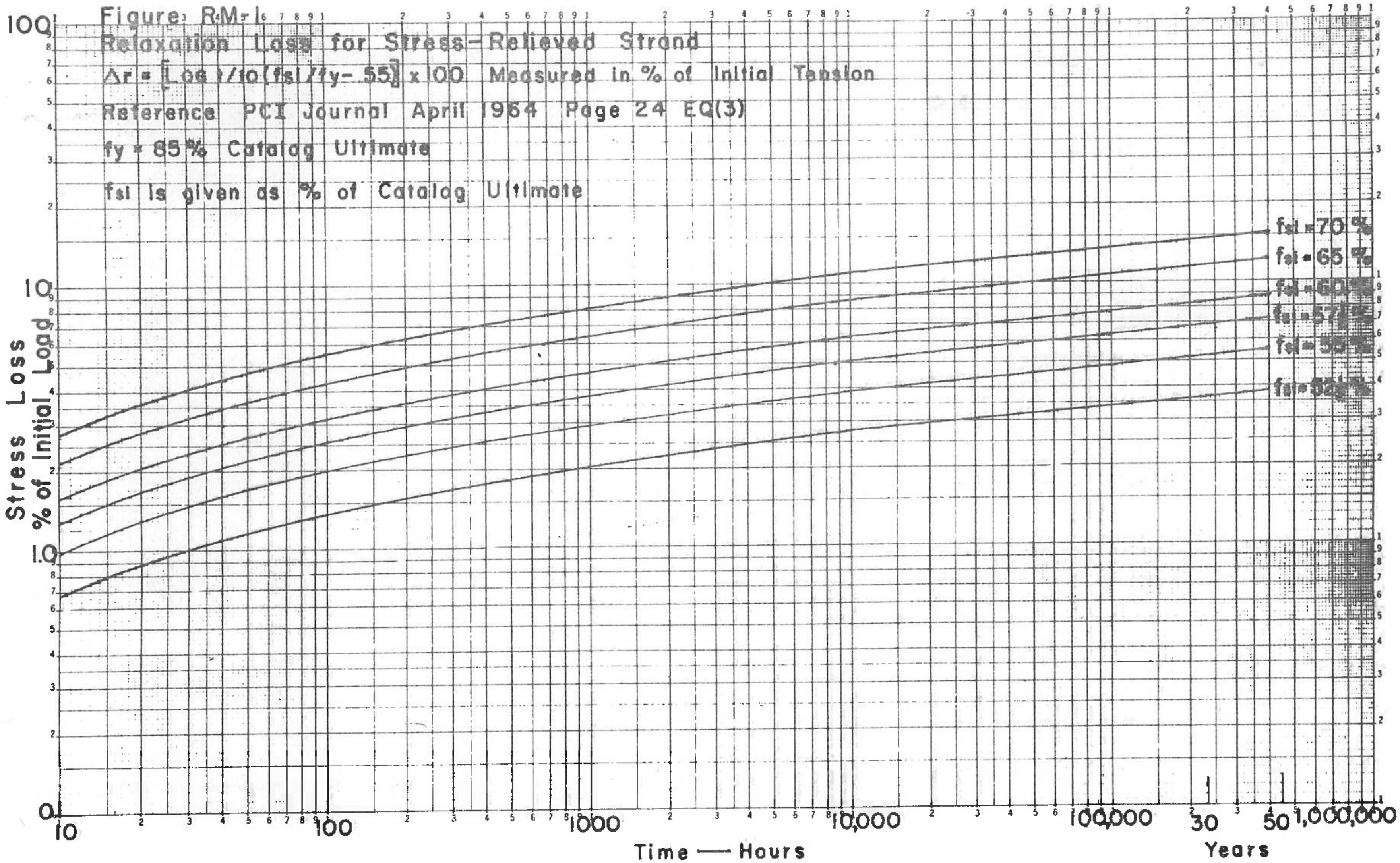


Figure RM-1
Relaxation Loss for Stress-Relieved Strand
Δr = [Log t/10 (fsi / fy - .55)] x 100 Measured in % of Initial Tension
Reference PCI Journal April 1964 Page 24 EQ(3)
fy = 85% Catalog Ultimate
fsi is given as % of Catalog Ultimate
fsi = 70 %
fsi = 65 %
Stress Loss
% of Initial Load
100
10
1.0
0.1
10
100
1000
10,000
100,000
1,000,000
30
50
Time — Hours
Years
S-LS-1

Figure C2

For post-tensioned members with only one tendon, there would be no elastic shortening loss because the member shortening takes place prior to tendon anchorage. However, most post-tensioned applications involve multiple tendons. When multiple tendons are used, the first tendon stressed and anchored sustains elastic shortening loss due to stressing of all subsequent tendons. The last tendon stressed sustains no elastic shortening loss. As a result of this, the formula for elastic shortening loss for post-tensioned structures includes a factor of 0.5 to approximate the total elastic shortening loss as the average loss in all tendons. In some applications, the total elastic shortening loss is eliminated by temporarily overstressing each tendon by the amount of elastic shortening anticipated. While the designer should be aware of this option for special structures, this procedure does complicate the stressing operation and it should only be used where the potential economies in tendon material justify the additional work in the field, and where the stressing crew is sufficiently trained that a different stress level can be specified in each tendon with reasonable assurance that the stressing will be properly carried out.

1.6.7(B)(1)(c) Creep of Concrete

The formula in the proposed revision for loss due to concrete creep is based on both field measurements and analytical studies conducted at the University of Illinois (1). This formula utilizes the same value of f_{cir} as defined and discussed above in the section on elastic shortening loss. This results in the same complications relative to calculating concrete creep loss as were discussed above with respect to elastic shortening.

The quantity f_{cds} related to the permanent loads applied after prestressing and represents the change of concrete fiber stress at c.g.s. caused by these loads. Usually, the application of load causes the concrete fiber stress at c.g.s. to decrease in magnitude, and thereby reduces the loss due to concrete creep; hence the negative sign in the formula for CR_C.

1.6.7(B)(1)(d) Relaxation of Prestressing Steel

New formulas are added in the section on steel relaxation relative to the various types of steel used for tendons of post-tensioned structures.

The steel relaxation loss formulas for post-tensioned structures utilizing strand and wire tendons include terms reflecting the influence of friction loss. It is intended that steel relaxation losses for post-tensioned structures be computed at the same points of elastic shortening and concrete creep loss. For post-tensioned structures utilizing bar tendons, a flat value of steel relaxation loss of 3000 psi (20.68 MPa) is specified. Representatives of companies supplying bar tendons have suggested that the steel relaxation loss for bars should also consider the friction loss reduction and other loss modification factors used for strand and wire tendons. While this was acknowledged as theoretically correct, it was considered that such a formula was an unnecessary refinement in view of the very nominal steel relaxation loss specified for bar tendons. This loss value for bar tendons is an arbitrary figure developed on consideration of constant strain steel relaxation loss curves for both smooth and deformed bars (3).

All formulas for steel relaxation loss consider, in an approximate way, the inter-relationship between steel relaxation and other losses. The higher the losses due to elastic shortening, concrete creep, and shrinkage, the lower will be the steel stress and the related steel relaxation loss. This inter-relationship can be evaluated accurately by use of time-related curves for losses due to steel relaxation, shrinkage, and concrete creep, and by use of a step-by-step integration process considering the inter-relationship of the losses. The formulas in the proposed revision were developed to provide approximations of loss values that would be obtained by this more precise integration process.

During final development of the revision, it was suggested that the base loss values of 20,000 psi (137.90 MPa) for strand and 18,000 psi (124.11 MPa) for wire should be modified to reflect the actual constant strain steel relaxation loss for wire and strand which is about 15 percent for tendons stressed to an initial stress of 0.70 f_s'. These values would be about 28,300 psi (195.12 MPa) for strand, and about 25,200 psi (173.75 MPa) for wire. It was felt that this would put the equations in a more direct relationship to research results from constant strain strand tests. While there was agreement that such a procedure would be preferable from a technical standpoint, it was noted that all the modification factors in the formulas would have to be increased so that the net losses would remain the same as those resulting from the proposed equations. Further, it was noted that studies would have to be made to develop appropriate values for the reduction coefficients in the formulas. In view of these considerations, it was decided to retain the equations as previously developed with base loss values of 20,000 psi (137.90 MPa) for strand and 18,000 psi (124.11 MPa) for wire.

In application of the friction loss reduction in the equations for steel relaxation loss for post-tensioned wire and strand tendons, note that the value of "FR" to be used is limited to the reduction in stress level below stress level of 0.70 f_s'. Post-tensioning tendons are commonly stressed in excess of 0.70 f_s' (usually to about 0.74 f_s') to allow for anchor seating loss. While this process often results in some small portion of the tendon length retaining a stress slightly in excess of 0.70 f_s' immediately after anchoring, the equations for steel relaxation in wire and strand tendons are considered applicable. However, due to this over stressing, it is necessary to limit the value of "FR" in the equation to the reduction below a stress of 0.70 f_s' (rather than the total friction loss) in order to retain the basic presumption of the equations that the initial stress in 0.70 f_s'.

Steel relaxation increases rapidly with increases in initial stress above 0.70 f_s', and it decreased rapidly as stresses go below 0.70 f_s'. The decreases in stress are considered in the equations. It is emphasized, however, that the equations are not accurate where initial stessses at points of maximum moment exceed 0.70 f_s'. Where such over stressing is specified, an adjustment (increase) should be made in the steel relaxation loss values.

1.6.7(B)(2) Lump Sum Values For Prestress Loss

Nearly all prestressed concrete bridges now in service were designed using the lump sum loss values of 35,000 psi (241.32 MPa) for pretensioned applications and 25,000 psi (172.37 MPa) plus friction for post-tensioned applications. In general the performance and serviceability characteristics of these bridges have been excellent. The revision of prestress losses in the 1971 AASHTO Interim Specifications was motivated by an awareness that research had shown some of the initial assumptions made in developing the initial lump sum loss values might be unconservative. This was particularly true with respect to steel relaxation loss, but applied to some extent to shrinkage and concrete creep losses. However, the basic intent of the provisions of the 1971 AASHTO Interim Specifications on prestress losses was to permit designers to develop lump sum losses that considered the prestress level and exposure conditions of the structure as well as the results of research then available. Some designers inferred an accuracy, or a requirement for precision, in the 1971 Interim Specifications that was not intended. As has been noted above, precise calculation of prestress losses is quite involved and required detailed information on materials, exposure, and construction procedures not generally available to designers. Further, such precise calculations are usually not generally available to designers. Further, such precise calculations are usually not warranted from the standpoints of structural economy, serviceability, or strength.

There are numerous factors in the design of prestressed concrete bridges, as with bridges of other materials, that are included in the specifications for the convenience of the designer with the understanding that these factors are only approximations of actual behavior. Some of the provisions of Section 3–Distribution of Loads in the 1973 AASHTO Specifications exemplify such approximations. There are other similar approximations in the specifications and in regard to prestressed concrete bridge design, a significant conservative approximation exists in the general use of the gross concrete section to compute section properties. Use of transformed section properties for all loads applied after prestressing is perfectly valid technically, and can result in an increase in the bottom section modulus in the order of 10 to 15 percent.

In view of the generally satisfactory performance of bridges designed using lump sum loss values, the conservative approximations in other portions of the specifications, and the convenience to the designer, it was considered desirable that the revision should include lump sum loss values for "prestressed members or structures of usual design." The introductory paragraph to the section indicates that structures of usual design include those of normal weight concrete, normal prestress levels, and average exposure conditions. In addition, it is suggested that the more precise procedures of Section 1.6.7(B)(1) should be used for structures of exceptionally long span or unusual design. The table of lump sum values includes concrete strengths of 4000 and 5000 psi (27.58 and 34.47 MPa). However, the lump sum loss values may be used for bridges with concrete strengths 500 psi (3.45 MPa) above or below the values of 4000 psi and 5000 psi (27.58 and 34.47 MPa) listed in the table headings. Thus, the range of concrete strengths covered by the lump sum loss values may be considered to extend from 3500 psi to 5500 psi (24.13 to 37.92 MPa). These lump sum loss values were developed on the assumption that the full allowable compressive stress of concrete is required by stress conditions during the life of the member.

REFERENCES

1. "Proposed Amendment to the AASHO Standard Specifications for Highway Bridges, Article 1.6.7 (B)–Prestress Losses" Sumbitted by Department of Civil Engineering, University of Illinois, Urbana, in cooperation with Illinois Department of Transportation, Division of Highways under Project IHR-93, "Field Investigation of Prestressed Concrete Highway Bridges."*
2. "Proposed Amendment to the 1971 AASHO Specifications Section 1.6.7(B) (1)" dated March 20, 1973, submitted by Dr. Ti Huang based on research at Lehigh University for the Pennsylvania Department of Transportation.*
3. "Proposed Revision of AASHO Section 1.6.7 (B) (2)–Prestress Losses for Post-Tensioned Bridges" submitted by C.L. Freyermuth, Director, Post-Tensioning Division, Prestressed Concrete Institute, April 1972.*
4. Committee Closure to Discussion of "Tentative Recommendations for Prestressed Concrete" ACI Journal, Vol. 29, No. 7, January 1958.
5. "A Study of the Practical Significance of Variations in Prestress Losses" by C.L. Freyermuth, Director, Post-Tensioning Division, Prestressed Concrete Institute (Available on request to PCI).
6. "Criteria for Prestressed Concrete Bridges" U.S. Department of Commerce, Bureau of Public Roads, Washington D.C., 1954.
7. "Selected Climatic Maps of the United States" Superintendent of Documents, U.S. Government Printing Office, Washington D.C. 20402.
8. "Design and Control of Concrete Mixtures" Eleventh Edition, Chapter 12, Volume Changes of Concrete, Portland Cement Association, Old Orchard Road, Skokie, Illinois 60076.
9. "Some Aspects of Durability and Volume Change of Concrete for Prestressing" by Paul Klieger, Research Department Bulletin 118, Portland Cement Association, Old Orchard Road, Skokie, Illinois 60076.
10. "Prestress Loss as Affected by Type of Curing" by J.A. Hanson, Development Department Bulletin D-75, Portland Cement Association, Old Orchard Road, Skokie, Illinois 60076.
11. "Influence of Size and Shape of Member on Shrinkage and Creep of Concrete" by Torben C. Hansen and Alan H. Mattock, Development Department Bulletin D-103, Portland Cement Association, Old Orchard Road, Skokie, Illinois 60076.
12. "Tentative Recommendations for Estimating Prestress Losses," PCI Committee of Prestress Losses, Prestressed Concrete Institute, 20 North Wacker Drive, Chicago, Illinois 60606.
13. "International Recommendations for the Design and Construction of Concrete Structures" CEB-FIP Recommendations, Cement and Concrete Association, 52 Grosvenor Gardens, London SW1, England.

*References 1-3 were distributed at AASHTO regional Bridge Committee meetings in 1973 and 1972, but have not been published. Requests for copies of these references should be directed to the agencies concerned.

1.6.8 – FLEXURE

Prestressed concrete members may be assumed to act as uncracked members subjected to combined axial and bending stresses within specified service loads.

In calculations of section properties, the transformed area of bonded reinforcement may be included in pretensioned members and in post-tensioned members after grouting; prior to bonding of tendons, areas of the open ducts shall be deducted.

1.6.9 – ULTIMATE FLEXURAL STRENGTH

A. Rectangular Sections

For rectangular or flanged sections in which the neutral axis lies within the flange, the ultimate flexural strength shall be assumed as:

$$M_u = A_s{*} f^*_{su}\, d \left(1 - 0.6 \frac{p^* f^*_{su}}{f'_c}\right)$$

B. Flanged Sections

If the neutral axis falls outside the flange (usually if the flange thickness is less than $1.4dp^*f_{su}^*/f'_c$), the ultimate flexural strength shall be assumed as

$$M_u = A_{sr} f^*_{su}\, d\left(1 - 0.6 \frac{A_{sr} f^*_{su}}{b'd f'_c}\right) + 0.85 f'_c (b - b')t(d - 0.5t)$$

where

$A_{sr} = A^*_s - A_{sf}$ — The steel area required to develop the ultimate compressive strength of the web of a flanged section.

$A_{sf} = \dfrac{0.85 f'_c (b - b')t}{f^*_{su}}$ — Steel area required to develop the ultimate compressive strength of the overhanging portions of the flange.

C. Steel Stress

Unless the value of f^*_{su} can be more accurately known from detailed analysis, the following values may be used:

Bonded members $\quad f^*_{su} = f'_s \left(1 - 0.5 \frac{p^* f'_s}{f'_c}\right)$

Unbonded members $\quad f^*_{su} = f_{sc} + 15{,}000$

Provided that:

1. The stress-strain properties of the prestressing steel approximate those specified in ASTM A416.
2. The effective prestress after losses is not less than $0.5f'_s$.

1.6.10 – MAXIMUM AND MINIMUM STEEL PERCENTAGE

A. Maximum Steel

Prestressed concrete members shall be designed so that the steel is yielding as ultimate capacity is approached. In general the reinforcement index shall be such that

$$\rho^* = \frac{f^*_{su}}{f'_c} \quad \text{for rectangular sections}$$

and

$$A_{sr} = \frac{f^*_{su}}{b'df'_c} \quad \text{for flanged sections}$$

does not exceed 0.30. For steel with reinforcement indices greater than this, the ultimate flexural strength shall be assumed not greater than:

$$M_u = 0.25f'_c bd^2 \quad \text{for rectangular sections, or}$$

$$M_u = 0.25b'd^2 f'_c + 0.85f'_c\,(b - b')\,t(d - 0.5t) \quad \text{for flanged sections.}$$

B. Minimum Steel

The total amount of prestressed and non-prestressed reinforcement shall be adequate to develop an ultimate load in flexure at the critical section at least 1.2 time the cracking load calculated on the basis of the modulus of rupture (refer to Article 1.6.6(B)(3).

1.6.11 – NONPRESTRESSED REINFORCEMENT

Nonprestressed reinforcement may be considered as contributing to the tensile strength of the beam at ultimate strength in an amount equal to its area times its yield point, provided that

$$\frac{p f_{sy}}{f'_c} + \frac{p^* f^*_{su}}{f'_c} + \frac{p f'_y}{f'_c}$$ does not exceed 0.3 for rectangular sections, or

$$\frac{A_s f_{sy}}{b'df'_c} + \frac{A_{sr} f^*_{su}}{b'df'_c} + \frac{A'_s f'_y}{b'df'_c}$$ does not exceed 0.3 for flanged sections.

1.6.12 – CONTINUITY

A. General

Continuous beams and other statically indeterminate structures shall be designed for adequate strength and satisfactory behavior. Behavior shall be determined by elastic analysis, taking into account the reactions, moments, shear and axial forces produced by prestressing, the effects of temperature, creep, shrinkage, axial deformation, restraint of attached structural elements, and foundation settlement.

B. Cast-in-place Post-Tensioned Bridges

The effect of secondary moments due to prestressing shall be included in stress calculations at working load. In calculating ultimate strength moment and shear requirements the secondary moments or shears induced by prestressing (with a load factor of 1.0) shall be added algebraically to the moments and shears due to factored or ultimate dead and live loads.

COMMENTARY

The current provisions of AASHTO 1.6.12(B) were first included in the 1970 interim specifications. The provisions were consistent with similar provisions then under consideration and later adopted for the ACI 318-71 Building Code. However, as pointed out in Professor Alan Mattock's discussion in the September 1970 ACI Journal, and in a January-February 1972 PCI Journal Article by Professor T.Y. Lin and Mr. Keith Thornton, it is necessary to have either full moment redistribution or a concordent tendon (which does not produce secondary moments) in order to correctly ignore secondary moments. In view of the fact that the amount of moment redistribution permitted by the ACI Code is generally quite small, and the fact that no moment redistribution at all is permitted by the AASHTO Specifications, these specifications, while normally quite conservative, are in error. Further, it has been demonstrated by tests that will be discussed below that the effects of secondary moments (and shears) are reflected in the moments in continuous post-tensioned beams throughout loading conditions up to failure.

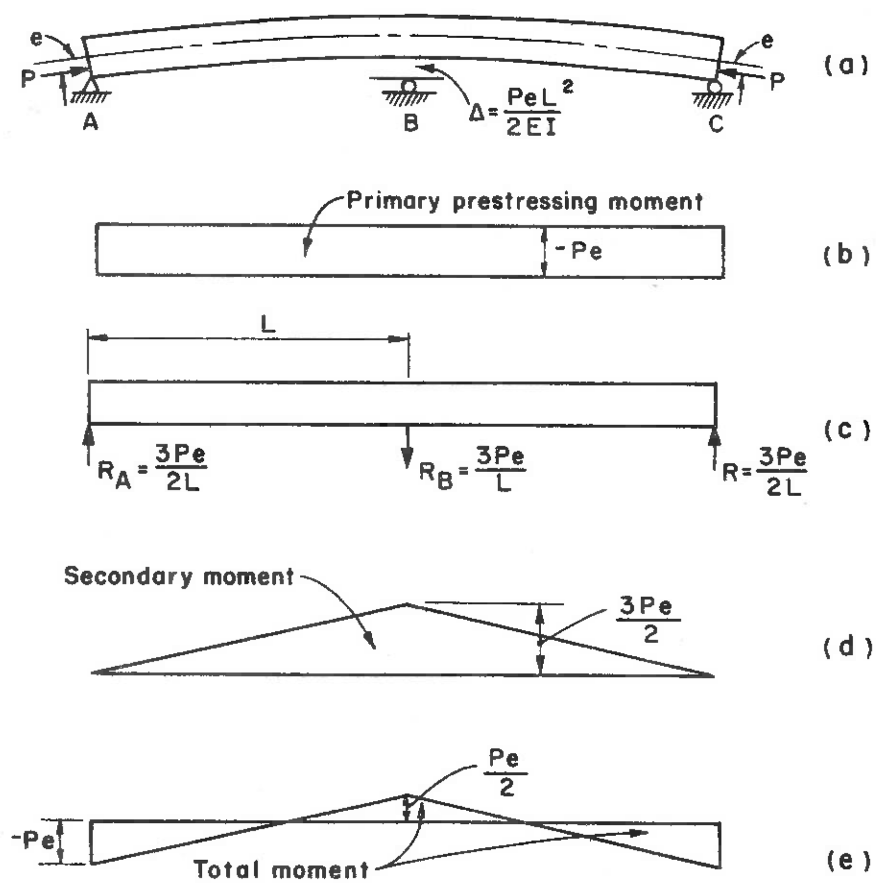

e
P
A
B
$\Delta = \frac{PeL^2}{2EI}$
C
e
P
(a)
Primary prestressing moment
-Pe
(b)
L
$R_A = \frac{3Pe}{2L}$
$R_B = \frac{3Pe}{L}$
$R = \frac{3Pe}{2L}$
(c)
Secondary moment
$\frac{3Pe}{2}$
(d)
$\frac{Pe}{2}$
-Pe
Total moment
(e)

Figure 1

Secondary Moments

To provide a common background for discussion, it may be well to define and illustrate the so-called secondary moment effect in continuous post-tensioned structures.

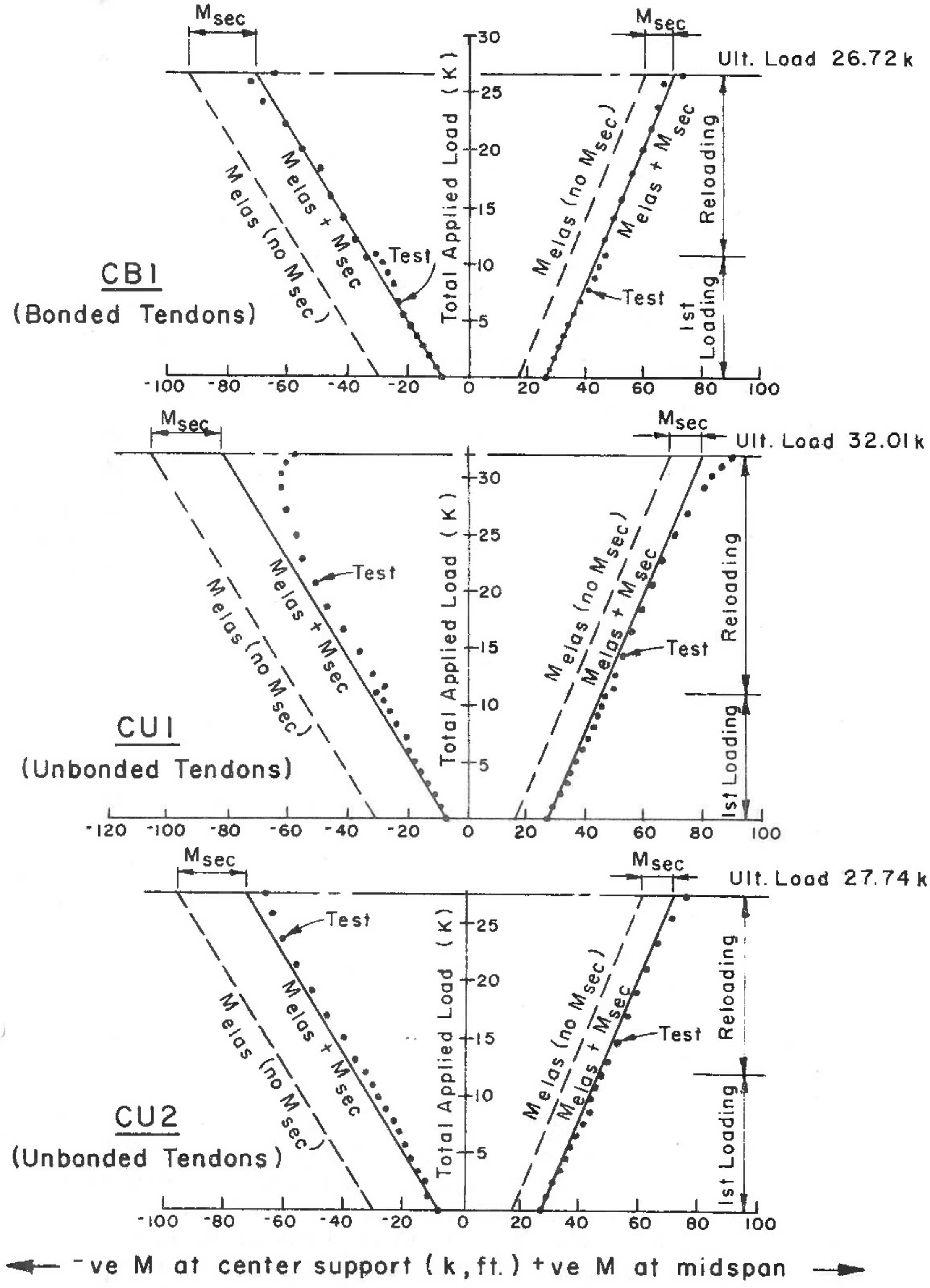
"Secondary moments" are developed due to restraints to the deformations resulting from the primary prestressing moments. This is illustrated in Fig. 1. Here a two-span continuous beam has a post-tensioning force, P, acting at a constant eccentricity, e. The moment created, Pe, is considered the "primary prestressing moment." If gravity loads are temporarily ignored, the moment due to post-tensioning will cause a theoretical upward deflection as shown. Should the beam be vertically restrained at the center support, the change in reactions resulting from the "primary prestressing moments" will be as shown in Fig. 1c. This change in reactions directly affects the beam shear diagram, and because the upward deflection at the interior support is prevented, the secondary moments of Fig. 1d are induced. For continuous beams the secondary moments may be expressed as functions of the reactions and they always vary linearly between supports. The moment at any section may then be expressed as the superposition of the primary and secondary moments, Fig. 1e. The term "secondary moment" is used because the moment is induced by the primary prestressing moment, and not because the secondary moment is negligible or necessarily smaller than the primary prestressing moment.

Test Results

A report on tests of three continuous post-tensioned beams was included in a research study published in 1969 by the University of Washington entitled "A Comparison of the Behavior of Post-Tensioned Prestressed Beams With and Without Bond" by Jan Yamasaki, Basil T. Katula, and Alan H. Mattock. A plot of the test results for these three beams is shown in Fig. 2. These test results illustrate that the moments in a continuous post-tensioned beam are correctly reflected by due consideration of secondary moments. When redistribution of moments takes place at extremely high loads, the redistribution starts from the lines including secondary moments. Thus, the true elastic moments in continuous post-tensioned structures must consider secondary moments, and these secondary moments influence the behavior of the beam clear to failure. Further, whatever moment redistribution might be considered, it should superimposed on moments which include the secondary moments resulting from post-tensioning.

Moment Redistribution

Current AASHTO Specifications for continuous conventionally reinforced and prestressed concrete highway bridges do not contain provisions for redistribution of elastically calculated design moments in determining the required flexural capacity at various locations in the structure. The load factor design provisions for steel structures do permit a reduction of 10 percent of elastic negative moments over supports of continuous beams of compact section, accompanied by appropriate increases in positive moments [AASHTO 1.7.124(A)(3)]. While consideration of moment redistribution provisions for continuous concrete structures is valid



(To convert Ft-k to N-m multiply by 1355.82)
(To convert k to N multiply by 4448.22)

Figure 2

technically, it was felt desirable to adopt a conservative approach on moment redistribution at the time the new reduced AASHTO load factors were adopted for reinforced and prestressed concrete bridges (1971 and 1972, respectively). After a suitable period of experience with the new load factor specifications, it may be desirable to give further consideration to use of moment redistribution provisions or limit design for both reinforced and prestressed continuous highway bridges. However, at this time, no modifications of current AASHTO Specifications are proposed with respect to moment redistribution in continuous prestressed concrete bridges.

It should be noted that the modification of design moments by secondary moments due to post-tensioning is independent of the concept of moment redistribution. Use of moment redistribution in addition to appropriate consideration of secondary moments due to post-tensioning in determining design moments, is suggested by the proposed revisions to ACI 318-7 Code printed in ACI Journal, Vol. 72, No. 1, Jan. 1975. Inspection of Fig. 2 will show both that consideration of secondary moments gives a more accurate representation of the actual moments in the structure at all loading stages and that moment redistribution (both negative and positive moments) occurs from the lines which result are presented in Fig. 2 were Tee beams. The limited amount of moment redistribution that occurred in specimens CB1 and CU2 reflected the fact that failrue occurred by compression failure of the concrete at the center support of the two span beams. Additional ductility and redistribution of moments could have been obtained by use of a nominal amount of compression reinforcement in the stem of the Tee beams in the negative moment area. A substantial amount of moment redistribution was observed in specimen CU1.

Practical Effects of the Proposed Revision

To provide an example of the effect of the proposed revision on design of a typical post-tensioned bridge, it will be applied to the two span structure illustrated in Fig. 3. Complete design calculations for this bridge are presented in "New Load Factors for Prestressed Bridges," Journal of the Prestressed Concrete Institute, May-June 1973, Vol. 18, No. 3.

Ultimate moment requirements due to dead load and live load at the center line of the bent and at 0.4 of the 162 ft. (14.380 m) span are as follows:

At 0.4 Span:

(1.3/.95 x 21.96 MNm)
1.3/.95 x 16,200 = + 22,200 ft-kips (30.10 MNm)
(1.3/.95 x 1.667 x 7.51 MNm)
1.3/.95 x 1.667 x 5,540 = + 12,260 ft. kips (17.11 MNm)
Load Factor Total M_u = + 34, 820 ft. kips (47.21 MNm)

At Bent:

(1.3/.95 x 36.88 MNm)
1.3/.95 x 27, 200 = – 37,200 ft. kips (–50.44 MNm)
1.3/.95 x 1.667 x 10.03 MNm)
1.3/.95 x 1.667 x 7,400 = –16,850 ft. kips (–22.84 MNm)
Load Factor Total M_u = – 54,050 ft. kips (–73.28 MNm)

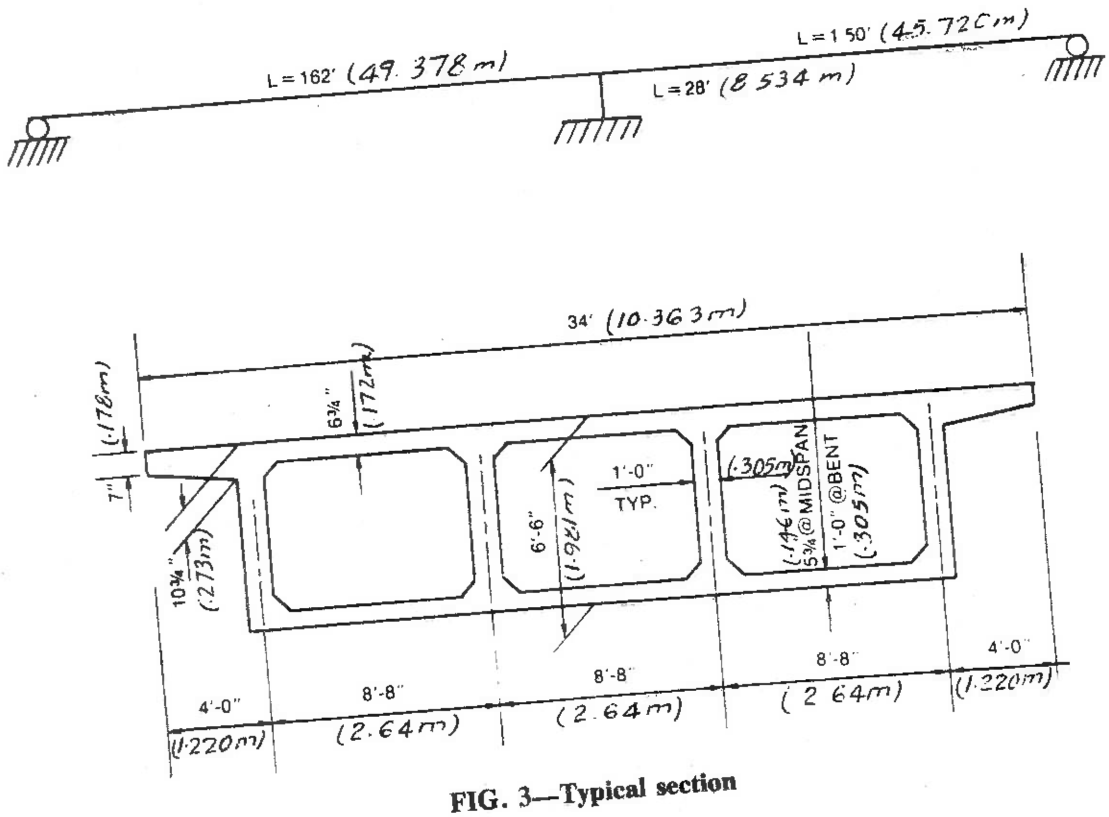


FIG. 3—Typical section

Secondary moments due to post-tensioning are superimposed on primary post-tensioning moments in Fig. 4. Note that the sign convention in Fig. 4 associates negative post-tensioning moments with compression in the top slab which is the opposite of the conventional assumption. At the bent, the secondary moment at service load after all losses is –1.16 x the initial jacking force in the tendon, and at the 0.4 span point the secondary moment is –0.46 x the initial jacking force in the tendon. Based on the required jacking force of 5920 kips (26.33 MN), the secondary moments then become (with consideration of the sign convention reversal of secondary moments):

At Bent:

1.16 x 5920 kips = + 6867 ft. kips
(.353 m x 26.33 MN) = (+9.30 MNm)

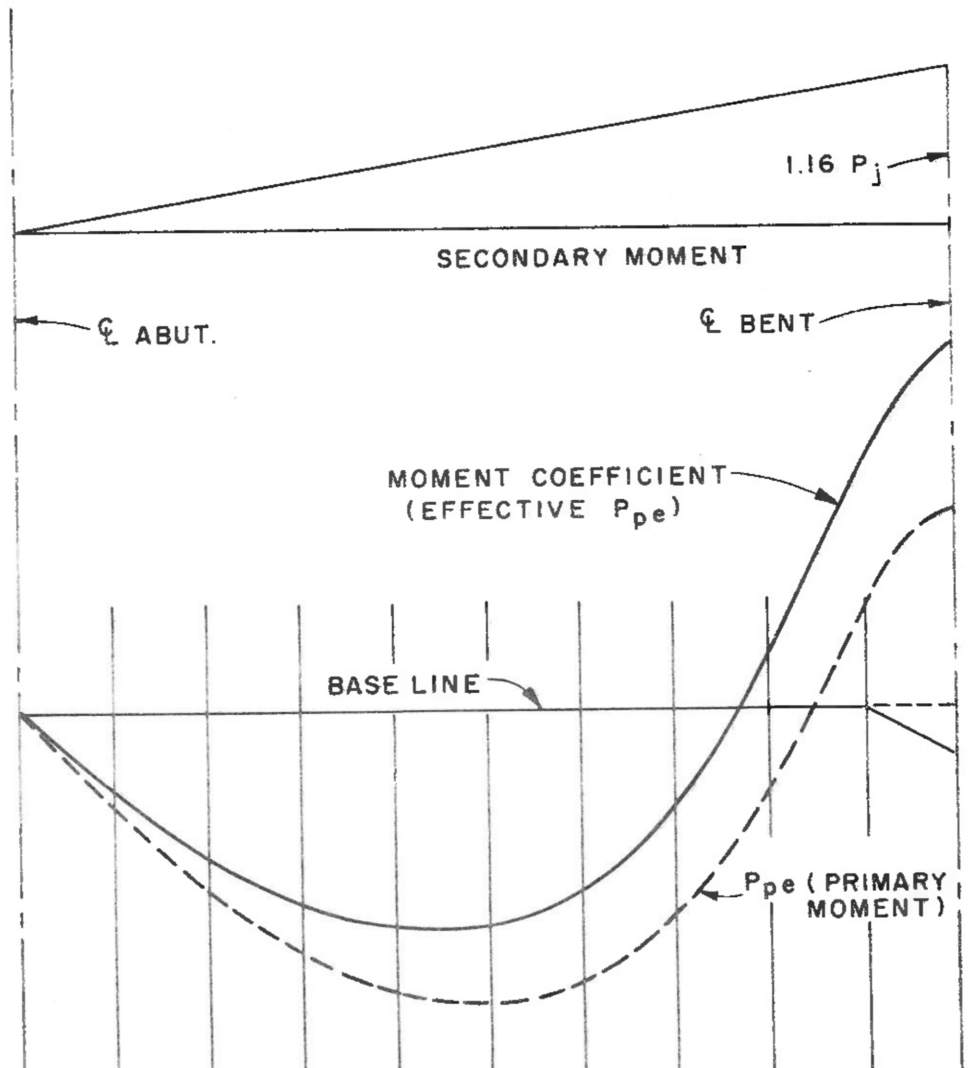
At 0.4 Span:

0.46 x 5920 kips - +2723 ft. kips
(.14 m x 26.33 MN) = (+3.69 MNm)

combining these secondary moments with the previously calculated ultimate results in the following ultimate moments:

	at 0.4 Span	at Bent
DL + LL	(+47.21 MNm) +34,820 ft. kips	(–73.28 MNm) –54,050 ft. kips
Secondary Moment	(+ 3.70 MNm) + 2,723 ft. kips	(+ 9.30 MNm) + 6,867 ft. kips
Modified Ultimate Moment	+37,543 ft. kips (+50.91 MNm)	–47,183 ft. kips (–63.98 MNm)

Thus, in this particular example, consideration of secondary moments in accordance with the revised specification increases the required ultimate moment capacity at midspan about 8 percent and reduces the required ultimate moment capacity at the bent by about 12.5 percent. The ultimate moment capacity provided by the post-tensioning tendon along in this example is 34,800 ft. kips (47.18 MNm) at the 0.4 span point, and 35,800 ft. kips (48.54 MNm) at the bent. Therefore, under the revised specification, additional conventional reinforcement would be required at both the bent and at midspan. Under current AASHTO Specifications, no conventional reinforcement would be required at midspan, and 34 Grade 60 No. 11 bars (35.81 mm) would be required at the bent. Under the revision, approximately 12 No. 8 bars (25.40 mm) would be required (for the entire structure cross section) for conventional positive moment reinforcement, and about structure cross section) for conventional positive moment reinforcement, and about 21 Grade 60 No. 11 bars (35.81 mm) would be required for



$P_j e$ =	0.70	1.27	1.68	1.93	2.03	1.88	1.39	0.52	-0.67	-1.72
M_s =	-0.12	-0.23	-0.35	-0.46	-0.58	-0.70	-0.81	-0.93	-1.04	-1.16
MOM.COEFF. =	0.58	1.04	1.33	1.47	1.45	1.18	0.57	-0.41	-1.71	-2.88

EFFECTIVE P_{pe} DIAGRAM

Figure 4

conventional negative moment reinforcement. Therefore, the revision reduces the required conventional reinforcement for negative moments in this example by about 38 percent. Post-tensioning tendon requirements are controlled by permissible stress considerations and are therefore not affected by the revision.

The revision also has an effect on the shear reinforcement requirements. In the case of the example presented in Fig. 3 and discussed above, the shear at the interior support is decreased by 6867 ft. kips/162 ft. (9.30 MNm/49.38 m), or 42.4 kips (.189 MN), and the shear at the abutment is increased by the same amount. Current AASHTO Specifications result in the following shears at Abutment 1

	(1.3/.90 x 2.35 MN)		
DL =	1.3/.90 x 529 kips	=	763 kips (3.394 MN)
	(1.3/.90 x 1.667 x .987 MN)		
LL =	1.3/.90 x 1.667 x 222 kips	=	531 kips (2.362 MN)
	TOTAL	=	1294 kips (5.756 MN)

at Bent

	(1.3/.90 x 3.847 MN)		
DL =	1.3/.90 x 865 kips	=	1250 kips (5.560 MN)
	(1.3/.90 x 1.667 x 1.112 MN)		
LL =	1.3/.90 x 1.667 x 250 kips	=	601 kips (8.233 MN);

Therefore the modification of design shears is an increase of about 3.3 percent at the abutment (42.4/1294) (.189 MN/5.756 MN), and a decrease of about 2.3 percent at the bent (42.4/1851) (.1886 MN/8.2332 MN). In general, consideration of secondary moments has much less impact on shear design than on moment requirements.

Summary

The revision to the AASHTO Specifications relative to consideration of secondary moments and shears due to post-tensioning in continuous bridges significantly improves the economy of this type of bridge by elimination of unnecessary conventional reinforcement. The revision also provides distribution of reinforcement which relates more closely to the actual moments and shears a structure might be required to carry under extreme overloads. The serviceability of continuous post-tensioned bridges is largely determined by the stresses at service loads, and this aspect of design remains unchanged under the revision. At some future time after experience has been obtained with structures designed under recently revised AASHTO Specifications, consideration should be given to the desirability of addition of moment redistribution or limit design provisions.

C. Bridges Composed of Simple-Span Precast Prestressed Girders Made Continuous

1. General

When structural continuity is assumed in calculating live loads plus impact and composite dead load moments, the effects of creep and shrinkage shall be considered in the design of bridges incorporating simple span precast, prestressed girders and deck slabs continuous over two or more spans.

2. Positive Moment Connection at Piers

Provision shall be made in the design for the positive moments that may develop in the negative moment region due to the combined effects of creep and shrinkage in the girders and deck slab, and due to the effects of live load plus impact in remote spans. Shrinkage and elastic shortening of the pier shall be considered when significant.

Non-prestressed positive moment connnection reinforcement at piers may be designed at a working stress of 0.6 times the yield strength but not to exceed 36 ksi.

3. Negative Moments

Negative moment reinforcement shall be proportioned by ultimate strength design with load factors in accordance with Article 1.6.5.

The effect of initial precompression due to prestress in the girders may be neglected in the negative moment calculation of ultimate strength if the maximum precompression stress is less than $0.4f'_c$ and the continuity reinforcement, "p" in the deck slab is less than 0.015; where $p = A_s/bd$.

The ultimate negative resisting moment shall be calculated using the compressive strength of the girder concrete regardless of the strength of the diaphragm concrete.

4. Compressive Stress in Girders at Piers at Service Loads

The compressive stress in ends of girders at piers resulting from addition of the effects of prestressing and negative live load bending shall not exceed $0.60f'_c$.

5. Shear

In continuous bridges of this type, shear reinforcement shall be designed according to Article 1.6.13

The horizontal shear connection between the cast-in-place slab and the precast girder shall be designed in accordance with Article 1.6.14.

1.6.13 – SHEAR (*)

Prestressed concrete members shall be reinforced for diagonal tension stresses. Shear reinforcement shall be placed perpendicular to the axis of the member.

The area of web reinforcement shall be

$$A_v = \frac{(V_u - V_c)\, s}{2 f_{sy}\, Jd}$$

but not less than

$$A_v = \frac{100\, b's}{f_{sy}}$$

where f_{sy} shall not exceed 60,000 psi

where $V_c = 0.06 f'_c bjd$ but not more than $180b'jd$.

Web reinforcement may consist of:

1. Stirrups perpendicular to the axis of the member
2. Welded wire fabric with wire located perpendicular to the axis of the member.

The spacing of web reinforcement shall not exceed three-fourths the depth of the member.

The critical sections for shear in simply supported beams will usually not be near the ends of the span where the shear is a maximum but at some point away from the ends in a region of high moment.

For the design of the web reinforcement in simply supported members carrying moving loads, it is recommended that shear be investigated only in the middle half of the span length. The web reinforcement required at the quarter points should be used throughout the outer quarters of the span.

For continuous bridges whose individual spans consist of precast prestressed girders, web reinforcement shall be designed for the full length of interior spans and for the interior 3/4 of the exterior span.

(*) The method for design of web reinforcement presented in ACI 318-71 is an acceptable alternate but web reinforcement shall not be less than $A_v = 100\, b'\, s/f_{sy}$.

1.6.14 – COMPOSITE STRUCTURES

A. General

Composite structures in which the deck is assumed to act integrally with the beam shall be interconnected in accordance with B, C and D of this Article to transfer shear along contact surfaces and to prevent separation of elements.

B. Shear Transfer

Full transfer of the ultimate horizontal shear forces may be assumed when contact surfaces are clean and intentionally roughened, minimum vertical ties are provided in accordance with D of this Article, all stirrups are fully anchored into all intersecting components, and the web members are designed to resist the entire vertical shear. Otherwise, ultimate horizontal shear stress shall be calculated and limited according to C and D of this Article.

C. Shear Capacity

In lieu of the requirements of B of this Article, ultimate horizontal shear stress may be computed by the formula $v = V_u Q / Ib$. To resist the computed shear stress, the following values of shear capacity shall be assumed at the contact surfaces:

When the minimum steel tie requirements of D of this Article are met 75psi

When the minimum steel tie requirements of D of this Article are met and the contact surfaces of the present elements are clean and artificially roughened 300psi

In addition to the above values, for each percent of stirrup or vertical tie reinforcement crossing the joint in excess of the minimum requirements of D of this Article . . . 150psi

D. Vertical Ties

All web reinforcement shall extend into cast-in-place decks. The minimum total area of vertical ties per linear foot of span shall be not less than the area of two No. 3 bars spaced at 12 in. Web reinforcement may be used to satisfy the vertical tie requirement. The spacing of vertical ties shall not be greater than four times the average thickness of the composite flange and in no case greater than 24 in.

E. Shrinkage Stresses

In structures with a cast-in-place slab on precast beams, the differential shrinkage tends to cause tensile stresses in the slab and in the bottom of the beams. Because the tensile shrinkage develops over an extended time period, the effect on the beams is reduced by creep. Differential shrinkage may influence the cracking load and the beam deflection profile. When these factors are particularly significant the effect of differential shrinkage should be added to the effect of loads.

1.6.15 – ANCHORAGE ZONES

For beams with post-tensioning tendons, end blocks shall be used to distribute the concentrated prestressing forces at the anchorage. Where all tendons are pretensioned wires or 7-wire strand, the use of end blocks will not be required.

End blocks shall have sufficient area to allow the spacing of the prestressing steel as specified in Article 1.6.16. Preferably, they shall be as wide as the narrower flange of the beam. They shall have a length at least equal to three-fourths of the depth of the beam and in any case 24 in. In post-tensioned members a closely spaced grid of both vertical and horizontal bars shall be placed near the face of the end block to resist bursting stresses. Amounts of steel in the end grid should follow recommendations of the supplier of the anchorage. Where such recommendations are not available the grid shall consist of at least No. 3 bars on 3-in. centers in each direction placed not more than 1½ in. from the inside face of the anchor bearing plate.

Closely spaced reinforcement shall be placed both vertically and horizontally throughout the length of the end block in accordance with accepted methods of end block stress analysis.

In pretensioned beams, vertical stirrups acting at a unit stress of 20,000 psi to resist at least 4 percent of the total prestressing force shall be placed within the distance of d/4 of the end of the beam, the end stirrups to be as close to the end of the beam as practicable. For at least the distance d from the end of the beam, nominal reinforcement shall be placed to enclose the prestressing steel in the bottom flange. For box girders provide transverse reinforcement anchored by extending the leg into the web of the girder.

1.6.16 – COVER AND SPACING OF STEEL

A. Minimum Cover

The following minimum concrete cover shall be provided for prestressing and conventional steel:

1. Prestressing steel and main reinforcement . . . . 1½ in.
2. Slab Reinforcement
    a. Top of slab . . . . 1½ in.
        (1) When de-icers are used . . . . 2 in.
    b. Bottom of slab . . . . 1 in.
3. Stirrups and ties . . . . 1 in.

When de-icer chemicals are used, drainage details shall dispose of de-icer solutions without constant contact with the prestressed girders. Where such contact cannot be avoided, or in locations where members are exposed to salt water, salt spray or chemical vapor, additional cover should be provided.

B. Minimum Spacing

The minimum clear spacing of prestressing steel at the ends of beams shall be as follows:

Pretensioning steel: three times the diameter of the steel or 1-1/3 the maximum size of the concrete aggregate, whichever is greater.

Post-tensioning ducts: 1½ in. or 1½ times the maximum size of the concrete aggregate, whichever is greater.

C. Bundling

When post-tensioning steel is draped or deflected, post-tensioning ducts may be bundled in groups of three maximum provided that the spacing specified in (B) is maintained in the end three feet of the member. Where pretensioning steel is bundled, the deflection points shall be investigated for secondary stresses and all bundling done in the middle third of the beam length.

D. Size of Ducts

For tendons made up of a plurality of wires, bars or strands, duct area shall be at least twice the net area of the prestressing steel.

For tendons made up of a single wire, bar or strand, the duct diameter shall be at least 1/4 in. larger than the nominal diameter of the wire, bar or strand.

1.6.17—POST-TENSIONING ANCHORAGES AND COUPLERS

Anchorages, couplers and splices for bonded post-tensioned reinforcement shall develop at least 95 percent of the minimum specified ultimate strength of the prestressing steel, tested in an unbonded state without exceeding anticipated set. Bond transfer lengths between anchorages and the zone where full prestressing force is required under service and ultimate loads shall normally be sufficient to develop the minimum specified ultimate strength of the prestressing steel. Couplers and splices shall be placed in areas approved by the engineer and enclosed in a housing long enough to permit the necessary movements. When anchorages or couplers are located at critical sections under ultimate load, the ultimate strength required of the bonded tendons shall not exceed the ultimate capacity of the tendon assembly, including the anchorage or coupler, tested in an unbonded state.

The anchorages of unbonded tendons shall develop at least 95 percent of the minimum specified ultimate strength of the prestressing steel without exceeding anticipated set. The total elongation under ultimate load of the tendon shall not be less than 2 percent measured in a minimum gauge length of 10 ft. (3.048 m).

For unbonded tendons, a dynamic test shall be performed on a representative specimen and the tendon shall withstand, without failure, 500,000 cycles from 60 percent to 66 percent of its minimum specified ultimate strength, and also 50 cycles from 40 percent to 80 percent of its minimum specified ultimate strength. The period of each cycle involves the change from the lower stress level to the upper stress level and back to the lower. The specimen used for the second dynamic test need not be the same used for the first dynamic test. Systems utilizing multiple strands, wires, or bars may be tested utilizing a test tendon of smaller capacity than the full size tendon. The test tendon shall duplicate the behavior of the full size tendon and generally shall not have less than 10 percent of the capacity of the full size tendon. Dynamic tests are not required on bonded tendons, unless the anchorage is located or used in such manner that repeated load applications can be expected on the anchorage.

Couplings of unbonded tendons shall be used only at locations specifically indicated and/or approved by the Engineer. Couplings shall not be used at points of sharp tendon curvature. All couplings shall develop at least 95 percent of the minimum specified ultimate strength of the prestressing steel without exceeding anticipated set. The coupling of tendons shall not reduce the elongation at rupture below the requirements of the tendon itself. Couplings and/or coupling components shall be enclosed in housings long enough to permit the necessary movements. All the coupling components shall be completely protected with a coating material prior to final encasement in concrete.

Anchorage, end fittings, couplers, and exposed tendons shall be permanently protected against corrosion.

COMMENTARY ART-1.6.17

The proposed revision of Section 1.6.17 is intended to eliminate ambiguity concerning the meaning of the phrase ". . . the required ultimate capacity of the tendons." Also, more specific and more restrictive requirements are proposed for unbonded tendons. The proposed revision basically follows the requirements of the "PCI Guide Specifications for Post-Tensioning Materials" which are presented in the Appendix. The PCI Guide Specifications contain "Notes to Specifiers" which are essentially a commentary, and which supplement the commentary presented here.

Some engineers interpret the words "the required ultimate capacity of the tendons" to mean that the anchorages have to be capable of developing 100 percent of the "minimum specified ultimate strength of the prestressing steel." Another possible interpretation is that the anchorage or coupler must be capable of developing the value of f_{su}^* specified in Section 1.6.9.(C).

From an engineering point of view, the second interpretation seems the more logical one, since f_{su}^* represents the highest possible tensile stress that the prestressing steel can develop compatible with the assumed condition of failure of the structural member. In the case of bonded tendons, the value of f_{su}^* is normally considerably less than 95% of the minimum specified ultimate strength of the prestressing steel. In the case of unbonded tendons, the maximum possible value of f_{su}^* is much lower still. It follows, therefore, that the practical and more realistic approach is to relate the requirements of anchorages and couplers to the value of f_{su}^* and not to the minimum specified ultimate strength of the prestressing steel. However, this interpretation does not provide a common basis for evaluating post-tensioning tendon anchorage assemblies because the value of f_{su}^* varies widely with structural design parameters. For this reason, the basic tendon strength requirement in the proposed revision has been set at 95 percent of the minimum specified ultimate strength of the prestressing steel. The value of 95 percent was selected for the reasons presented below.

In testing any large post-tensioning any large post-tensioning tendon such as those normally used in bridge construction today, the following factors must be taken into account:

1) Every anchorage system creates a stress riser in the tendon material, thus reducing slightly (usually 2 to 3 percent) the actual ultimate strength of the prestressing steel.
2) In the relatively short length of a test tendon [usually not more than 10 feet(3.048 m)] it is extremely difficult to distribute the test force uniformally among all the prestressing steel elements.
3) Due to the multiplicity of elements, the test result will reflect, in general, the strength of the weakest elements in the group and not even the average. In summary, a full size tendon test is not always a true representation of the performance of the tendon in an actual structure and, even though most post-tensioning systems do meet the criteria of developing the minimum specified ultimate strength of the prestressing steel, actual tests may, in some cases, fall short of this value.

The PCI Guide Specifications for Post-Tensioning Materials [See Appendix, Section 3.1.5(1)] requires that anchorages for bonded tendons tested in an unbonded state develop only 90 percent of the minimum specified ultimate tensile strength of the prestressing steel. However, all post-tensioning materials fabricators in the U.S. have indicated that their anchorages are capable of meeting the 95 percent requirement. The State of California has been using this requirement for many years.

In general, the specification requirements are more restrictive for unbonded tendons due to the increased importance of the anchorage and coupler capacity.

Specific dynamic test requirements have been added for unbonded tendon assemblies. It should be noted that the 6 percent stress range specified represents a very severe test for an unbonded tendon which could not be duplicated in an actual structure. This is so because stresses developed in unbonded tendons due to load are distributed over the full tendon length rather than concentrated at the point of maximum moment. This is reflected in the formula for ultimate steel stress for unbonded tendons in AASHTO 1.6.9(C) which specifies an increase in tendon stress above service load stresses of only 15,000 psi (103.42 MPa). The 6 percent stress range for 270,000 psi (1862 MPa) seven wire strand is over 16,000 psi (110.32 MPa), or more than the specifications permit for the total increase in stress between service load and ultimate strength.

For further information relative to Article 1.6.17, See:

PCI Appendix, 3-1 "Guide Specification for Post-Tensioning Materials."

1.6.18 – EMBEDMENT OF PRESTRESSED STRAND

Three or seven-wire pretensioning strand shall be bonded beyond the critical section for a development length (in inches) not less than

$$\left(f^*_{su} - \frac{2}{3} f_{se}\right) D$$

where

D is the nominal diameter in inches, f^*_{su} and f_{se} are kips per sq. in. and the parenthetical expression is considered to be without units.

Investigation may be limited to those cross-sections nearest each end of the member which are required to develop their full ultimate capacity.

Where strand is debonded at the end of a member, the development length required above shall be doubled.

1.6.19 – CONCRETE STRENGTH AT STRESS TRANSFER

Unless otherwise specified, stress shall not be transferred to concrete until the compressive strength of the concrete as indicated by test cylinders cured by methods identical with the curing of the members is at least 4000 psi for pretensioned members and 3500 psi for post-tensioned members.

1.6.20 – BEARINGS

Bearing devices for prestressed concrete structures shall be designed in accordance with Articles 1.7.50 through 1.7.57 and Section 12.

1.6.21 – SPAN LENGTHS

The effective span lengths of slabs shall be as specified in Article 1.3.2.

The effective span lengths of simply supported beams shall not exceed the clear span plus the depth of the beam.

The span length of continuous or restrained floor slabs and beams shall be the clear distance between faces of support. Where fillets making an angle of 45 degrees or more with the axis of a continuous or restrained slab are built monolithic with the slab and support, the span shall be measured from the section where the combined depth of the slab and the fillet is at least one and one-half times the thickness of the slab. Maximum negative moments are to be considered as existing at the ends of the span, as above defined. No portion of the fillet shall be considered as adding to the effective depth.

1.6.22 – EXPANSION AND CONTRACTION

In all bridges, provisions shall be made in the design to resist thermal stresses induced or means shall be provided for movement caused by temperature changes. Movements not otherwise provided for including shortening during stressing shall be provided for by means of hinged columns, rockers, sliding plates, elastomeric pads or other devices.

1.6.23 — T-BEAMS

In beam and slab composite construction, the junction between flange slab and web beam shall meet the requirements of Article 1.6.14, if the slab is to be considered an integral part with the beam. In cast-in-place or precast T-beams, equally effective shear resistance shall be provided at the junction of slab and beam.

A. Effective Flange Width

In beam and slab construction, using precast, prestressed beams and composite cast-in-place slabs, effective and adequate bond and shear resistance shall be provided at the junction of the beam and slab. The slab may then be considered as an integral part of the beam, but its assumed effective width as a T-beam flange shall not exceed the following:

1. One-fourth of the span length of the beam
2. The distance center-to-center of the beams
3. Twelve times the least thickness of the slab plus the width of the girder web.

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

For monolithic prestressed construction, with normal slab span and girder spacing, the effective flange width is the distance center-to-center of beams, for very short spans, or where girder spacing is excessive, analytical investigations shall be made to determine the anticipated width of flange acting with the beam.

B. Construction Joints

When a construction joint is required between slab and beam of a cast-in-place T-beam the joint shall meet the requirements of Article 1.6.14.

C. Diaphragms

Diaphragms of precast or cast-in-place construction using prestressed or non-prestressed reinforcement are recommended at span ends. Intermediate diaphragms are not required in spans up to 40 ft.; are recommended at mid-span for spans from 40 ft. to 80 ft.; and are recommended at span third points for spans in excess of 80 ft.

D. Isolated Beams

Isolated beams in which the T-form is used only for the purpose of providing additional compression area, shall have a flange thickness not less than one-half the web width. For composite construction flange width shall not exceed 4 times the web width. For monolithic prestressed construction, flange width shall not exceed 15 times the web width and shall be adequate for all design loads.

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 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 ($4 \leqslant N_B \leqslant 10$)
S = beam spacing, in feet ($6.75 \leqslant S \leqslant 11.00$)
L = span length, in feet
k = $0.07W - N_L\,(0.10N_L - 0.26) - -.20N_B - 0.12$
w = roadway width between curbs, in feet (defined as W_c in Art. 1.2.6)($32 \leqslant W \leqslant 66$)

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 integral part of the girder and the entire width shall be assumed to be effective in compression.

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 intersections 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 0.15f'c.

Changes in girder stem thickness shall be tapered for a minimum distance of 12 times the difference in web thickness.

F. Diaphragms*****

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

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

Section 7 — STRUCTURAL STEEL DESIGN

1.7.1 — ALLOWABLE STRESSES

TABLE 1.7.1

Allowable unit stresses shown in pounds per square inch.

The modulus of elasticity of all grades of steel shall be assumed to be 29,000,000 psi and the coefficient of linear expansion 0.0000065 per degree Fahrenheit.

		Structural Carbon Steel	High Strength Low Alloy Structural Steel			
ASTM Designation		A36	A588	A441		
Thickness of Plates		(5) Up to 8" Incl.	(5) Up to 4" Incl.	Over 1½" To 4" Incl.	¾" and Under	Over ¾" To 1½" Incl.
				Shapes (6)		
				Group 4,5	Group 3	Group 1,2
Minimum tensile strength	Fu	58,000	70,000	63,000	67,000	70,000
Minimum yield point	Fy	36,000	50,000	42,000	46,000	50,000
Axial Tension, net section Tension in extreme fiber of rolled shapes, girders and built-up sections subject to bending, net section Axial compression, gross section: stiffeners of plate girders. Compression in splice material, gross section.	Fb = .55Fy	20,000	27,000	23,000	25,000	27,000

		Structural Carbon Steel	High Strength Low Alloy Structural Steel			
ASTM Designation		A36	A588	A441		
Thickness of Plates		(5) Up to 8" Incl.	Up to 4" Incl.	Over 1½" To 4" Incl.	Over ¾" To 1½" Incl.	¾" and Under
				Shapes (6)		
				Group 4, 5	Group 3	Group 1, 2
Compression in extreme fibers of plates, rolled shapes, girders and built-up sections, subject to bending, gross section, when compression flange is: (A) Supported laterally its full length by embedment in concrete.	$F_b = .55F_y$	20,000	27,000	23,000	25,000	27,000
(B)(1) Partially supported or is unsupported.	(2) $F_b = .55F_y \left[1 - \frac{(\frac{\ell}{r'})^2 F_y}{4\pi^2 E}\right]$ with $r'^2 = \frac{b^2}{12}$ $F_b = .55F_y \left[1 - \frac{3(\frac{\ell}{b})^2 F_y}{\pi^2 E}\right]$	$= 20{,}000 - 7.5\left(\frac{\ell}{b}\right)^2$	$27{,}000 - 14.4\left(\frac{\ell}{b}\right)^2$	$23{,}000 - 10.2\left(\frac{\ell}{b}\right)^2$	$25{,}000 - 12.2\left(\frac{\ell}{b}\right)^2$	$27{,}000 - 14.4\left(\frac{\ell}{b}\right)^2$
with $\frac{\ell}{b}$ not greater than:		36	30	34	32	30
Compression in concentrically loaded columns, (3) with $C_c = (2\pi^2 E/F_y)^{1/2}$ = ...		126.1	107.0	116.7	111.6	107.0
when $KL/r \leqslant C_c$	$F_a = \frac{F_y}{F.S.}\left[1 - \frac{(KL/r)^2 F_y}{4\pi^2 E}\right] =$	$16{,}980 - .53(KL/r)^2$	$23{,}580 - 1.03(KL/r)^2$	$19{,}810 - .73(KL/r)^2$	$21{,}700 - .87(KL/r)^2$	$23{,}580 - 1.03(KL/r)^2$
when $KL/r \geqslant C_c$	$F_a = \frac{\pi^2 E}{F.S.(KL/r)^2} = \frac{135{,}000{,}740}{(KL/r)^2}$ with F.S. = 2.12					
Shear in beam and girder webs gross section	$F_v = 0.33\ F_y$	12,000	17,000	14,000	15,000	17,000

		Structural Carbon Steel	High Strength Low Alloy Structural Steel			
ASTM Designation		A36	A588	A441		
Thickness of Plates		(5) Up to 8″ Incl.	(5) Up to 4″ Incl.	Over 1½″ To 4″ Incl.	¾″ and Under	Over ¾″ To 1½″ Incl
				Shapes (6)		
				Group 4,5	Group 3	Group 1,2
Bearing on milled stiffeners and other steel parts in contact . . . . (rivets and bolts excluded)	0.08 Fy	29,000	40,000	34,000	37,000	40,000
Stress in extreme fiber of pins. (4)	0.80 Fy	29,000	40,000	34,000	37,000	40,000
Shear in pins	Fv = 0.40 Fy	14,000	20,000	17,000	18,000	20,000
Bearing on pins not subject to rotation . . . . (7)	0.80 Fy	29,000	40,000	34,000	37,000	40,000
Bearing on pins subject to rotation (such as used in rockers and hinges)	0.40 Fy	14,000	20,000	17,000	18,000	20,000
Bearing on power-driven rivets and high strength bolts	1.22 Fy	44,000	60,000	50,000	56,000	60,000

FOOTNOTES:

Other types of steel may be used only with written approval of the Deputy Chief Engineer, and with submission of complete material and design specifications.

(1) Continuous or cantilever beams or girders may be proportioned for negative moment at interior supports for an allowable unit stress 20 percent higher than permitted by this formula but in no case exceeding allowable unit stress for compression flange supported its full length. If cover plates are used, the allowable static stress at the point of theoretical cutoff shall be as determined by the formula.

(2) l = length, in inches, of unsupported flange between lateral connections, knee braces or other points of support. For continuous beams and girders, l may be taken as the distance from interior support to point of dead load contraflexure if this distance is less than designated above.

For cantilever beam and girder, l may be taken as twice the distance from the support to the end of the cantilever if this distance is less than designated above.

r = radius of gyration, in inches, of the compression flange about the axis in the plane of the web.
b = flange width, in inches.

(3) E = modulus of elasticity of steel
r = governing radius of gyration
L = actual unbraced length
K = effective length factor. See Appendix C
F.S. = factor of safety = 2.12

For members with eccentric loading see Article 1.7.17.
For Graphic representation of these formulas see Appendix C.
The formulas do not apply to members with variable moment of inertia. Procedures for designing members with variable moments of inertia can be found in the following references: "Engineering Journal", American Institute of Steel Construction, January, 1969, Volume 6, No. 1, and October, 1972, Volume 9, No. 4, and "Steel Structures", by William McGuire, 1968, Prentice-Hall, Inc., Englewood Cliffs, N.J.

(4) See also Article 1.7.4.

(5) Limited to 4 in. thickness for structural members other than bearing assembly components.

(6) Groups 1 and 2 include all shapes except those in Groups 3, 4 and 5.
Group 3 includes angles zees and tees over 3/4 inch in thickness, the following wf shapes, and steel bearing piles over 102 pounds/ft.

Nominal Depth, In. X Nominal Width, In.	Weight Per Ft., Lb.
36 X 16 1/2	All Weights
33 X 15 3/4	All Weights
14 X 16	142 to 211 incl.
12 X 12	120 to 190 incl.

Group 4 includes wide flange shapes having nominal depth of 14 inches, nominal width of 16 inches, and weight per foot of 219 to 550 pounds, inclusive.
Group 5 includes wide flange shapes having nominal depth of 14 inches, nominal width of 16 inches, and weight per foot of 605 to 730 pounds, inclusive.
For breakdown of Groups 1 and 2 see ASTM A6.

(7) "Except for the mandatory notch toughness requirements, the AASHTO designations are similar to the ASTM designations."

1.7.2 – ALLOWABLE STRESSES FOR WELD METAL

Unless otherwise specified, the yield point and ultimate strength of weld metal shall be equal to or greater than minimum specified value of the base metal. Allowable stresses on the effective areas of weld metal shall be as follows:

Butt Welds–

The same as the base metal joined, except in the case of joining metals of different yields when the lower yield material shall govern.

Fillet Welds–

F_V = 12,400 psi on base metal with minimum specified yield point of 36,000 psi or 1100 lbs. per inch per 1/8 in. of leg.

F_V = 14,700 psi on base metal with minimum specified yield point of between 40,000 psi and 50,000 psi inclusive or 1300 lbs. per inch per 1/8 in. of leg.

Plug Welds–

F_V = 12,400 psi for resistance to shear stress only.

Where F_V = Allowable basic shear stress.

1.7.3 – FATIGUE STRESSES

The number of cycles of maximum stress to be considered in the design shall be selected from Table 1.7.3A unless traffic and loadometer surveys or other considerations indicate otherwise.

TABLE 1.7.3A – STRESS CYCLE

Main (Longitudinal) Load Carrying Members

Case	ADTT*	Truck Loading or Truck Trailer	Lane Loading**
I	2500 or more	Over 2,000,000	500,000
II	Less than 2500	500,000	100,000

Transverse Members and Details Subjected to Wheel Loads

Case	ADTT**	Truck Loading
I	2500 or more	Over 2,000,000
II	Less than 2500	2,000,000

*Average Daily Truck Traffic

**Longitudinal members should also be checked for truck loading.

Allowable fatigue stresses shall apply to those Group Loadings that include live load or wind load.

The number of cycles of stress to be considered for wind loads in combination with dead loads, except for structures where other considerations indicate a substantially different number of cycles, shall be 100,000 cycles.

Members and fasteners subject to repeat variations or reversals of stress shall be designed so that the maximum stress does not exceed the basic allowable stresses given in Articles 1.7.1 and 1.7.2, and that the actual range of stress does not exceed the allowable fatigue stress range given in Table 1.7.3B for the appropriate type and location of material shown in Table 1.7.3C and illustrated in Figure 1.7.3.

The range of stress is defined as the algebric difference between the maximum stress and the minimum stress. Tension stress is considered to have the opposite algebric sign from compression stress.

In Table 1.7.3C, "T" signifies range in tensile stress only; "Rev." signifies a range of stress involving both tension and compression during a stress cycle.

TABLE 1.7.3B

Category (See Table 1.7.3C)		Allowable Range of Stress, F_{sr} (ksi)			
		For 100,000 Cycles	For 500,000 Cycles	For 2,000,000 Cycles	For over 2,000,000 Cycles
Base Metal	A	60	36	24	24
Base Metal Adj. to Con. Fillet or Groove Welds	B	45	27.5	18	16
Base Metal in Girder Webs adj. to stiffeners	C	32	19	13	10
Base Metal adj. to groove welded attachments with special transitions or stud shear conn.	D	27	16	10	7
Base Metal adj. to fillet welds	E	21	12.5	8	5
Weld Metal	F	15	12	9	8

TABLE 1.7.3C

General Condition	Situation	Kind of Stress	Stress Category	Example of (See Fig. 1.7.3)
Plain Material	Base metal with rolled or cleaned surfaces. Flame cut edges with ANSI smoothness of 1000 or less	T or Rev.	A	1, 2
Built-up	Base metal and weld metal in members without attachments, built-up of plates or shapes connected by continuous full or partial penetration groove welds or by continuous fillet welds parallel to the direction of applied stress	T or Rev.	B	3
	Calculated flexural stress at toe of transverse stiffener welds on girder webs on girder flanges	T or Rev.	C E	4
	Base metal at end of partial length welded over plates having square or tapered ends with or without welds across the ends	T or Rev.	E	5
Fillet Welds	Shear stress on throat of fillet welds	Shear	F	7
Groove Welds	Base metal and weld metal at full penetration groove welded splices of rolled and welded section having similar profiles when welds are ground flush and weld soundness established by non-destructive inspection	T or Rev.	B	6, 8

TABLE 1.7.3C (continued)

General Condition	Situation	Kind of Stress	Stress Category	Example of (See Fig. 1.7.3)
Grove Welds (Con't.)	Base metal and weld metal in or adjacent to full penetration groove welded splices at transitions in width or thickness, with welds ground to provide slopes no steeper than 1 to 2½ with grinding in the direction of applied stress, and weld soundness established by non-destructive inspection	T or Rev.	B	9, 10
	Base metal at details attached by groove welds subject to transverse and/or longitudinal loading	T or Rev.	E	11
	Above With 1' - 0" Rad. Transition	T or Rev.	C	12
Fillet Welded Connections	Base metal adjacent to Stud Shear Connectors	T or Rev.	D	13
	Base metal at details attached by fillet welds	T or Rev.	E	14, 15
Mechanically Fastened Connections	Base metal at gross section of high-strength bolted slip resistant connections except axially loaded joints which induce out-of-place bending in connection material	T or Rev.	B	16
	Base metal at net section of high-strength bolted bearing-type connections and other mechanically fastened joints	T	B	16
Fillet Welds	Shear stress on throat of fillet welds	Shear	F	7

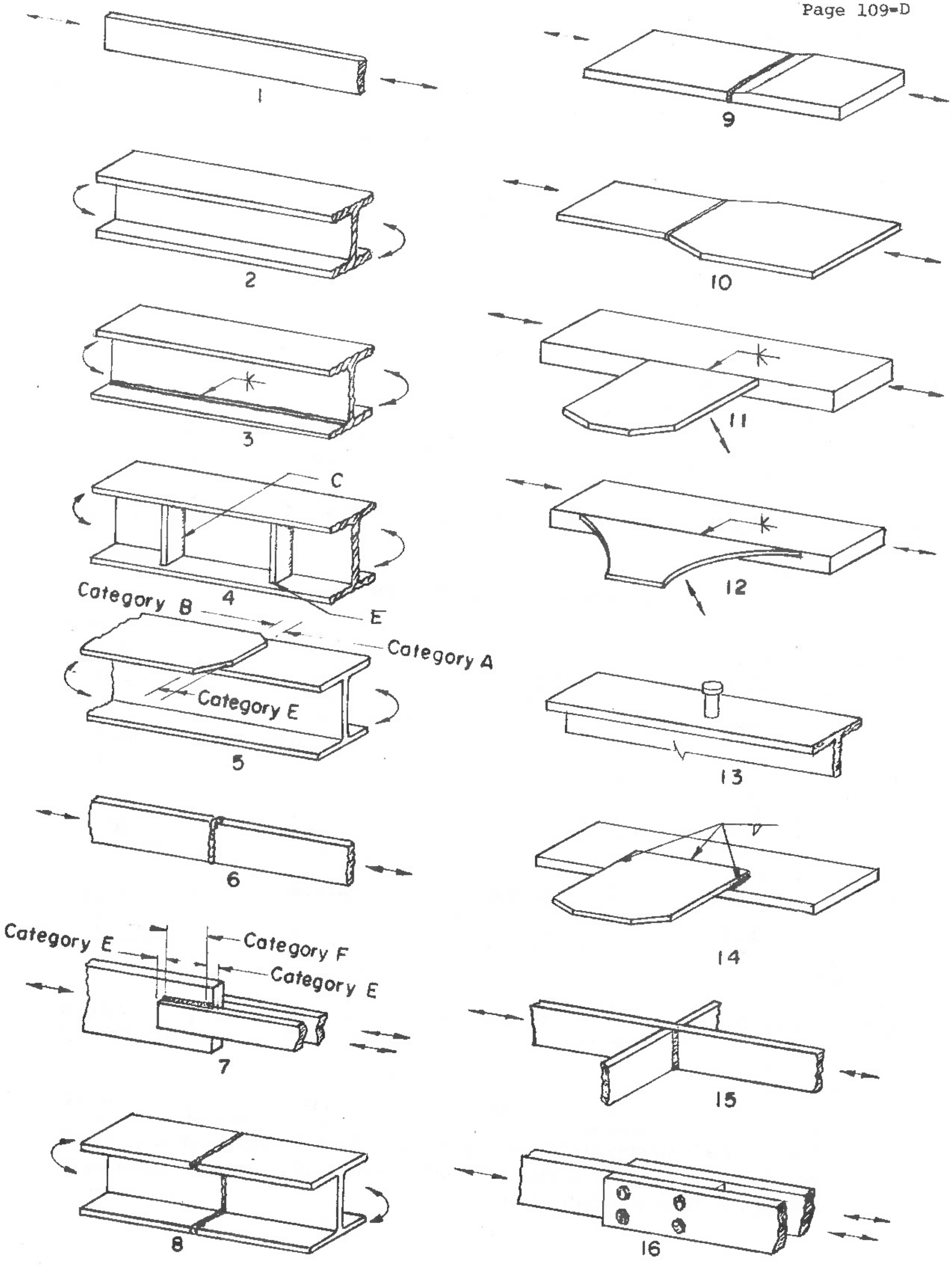

1
2
3
4
C
E
Category B
Category A
Category E
5
6
Category E
Category F
Category E
7
8
9
10
11
12
13
14
15
16

Fig. 1. 7. 3

1.7.4 – PINS, ROLLERS AND EXPANSION ROCKERS

The effective bearing area of a pin shall be its diameter multiplied by the length in bearing. When parts in contact have different yield point, F_y shall be the smaller value.

Bearing per linear inch on expansion rockers and rollers shall not exceed the values obtained by the following formulas:

Diameters up to 25 inches $p = \frac{F_y - 13{,}000}{20{,}000} 600d$

Diameters from 25 to 125 inches $p = \frac{F_y - 13{,}000}{20{,}000} 3{,}000\sqrt{d}$

where

p = allowable bearing in pounds per linear inch

d = diameter of rocker or roller in inches

F_y = yield point in tension of steel in the roller or bearing plate, whichever is the smaller.

Steel may conform to one of the following designations in addition to the designations listed in Article 1.7.1.

Steel forgings carbon and alloy for general industrial use, (AASHTO M102-ASTM A668)

ALLOWABLE STRESSES IN PINS

ASTM Designation with Grade or Class	A668 Class D	A668 Class F	A668 Class G
Size Limitations		10-in dia. or less	Over 10-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

(*) This shall apply to pins used primarily in axially loaded members such as truss members and cable adjusting links. It shall not apply to pins used in members having rotation caused by expansion or deflection.

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	Bearing Type Connection
(A) Low carron 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***

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:

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

Where s = the computed unit stress in shear

T = the computed unit stress in tension

S = the allowable unit stress in shear

*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.

Where shown in the design drawings and at other locations approved by the designer, oversize, short-slotted, and long-slotted holes may be used with high strength bolts 5/8-in. (15.9 mm) and larger in diameter proportioned to meet the allowable unit stresses, except as hereinafter restricted:

1. Oversize or slotted holes shall be used on secondary members only.
2. Oversize holes are 3/16-in. (4.8 mm) larger than bolts 7/8-in. (22.2 mm) and less in diameter, 1/4-in. (6.4 mm) larger than bolts 1-in. in diameter (25.4 mm), and 5/16-in. (7.9 mm) larger than bolts 1.-1/8-in. (28.6 mm) and greater in diameter. They may be used in any or all plies of friction-type connections. Hardened washers shall be installed over exposed oversize holes.
3. Short-slotted holes are 1/16-in. (1.6 mm) wider than the bolt diameter and have a length which does not exceed the oversize diameter provisions of subparagraph 1 by more than 1/16-in. (1.6 mm). They may be used in any or all plies of friction-type connections. Hardened washers shall be installed over exposed short slotted holes.
4. Long-slotted holes are 1/16-in. (1.6 mm) wider than the bolt diameter and have a length more than allowed in subparagraph 2, but not more than 2-1/2 times the bolt diameter.

In friction-type connections, oversize or slotted holes may be used without regard to direction of loading if one-third more bolts are provided than needed to satisfy the allowable unit stresses, except as herein restricted.

Oversize or slotted holes must be shown on the design drawings.

Long-slotted holes may be used in only one of the connected parts of friction-type connection at an individual faying surface.

Structural plate washers, or a continuous bar not less than 5/16-in. (7.9 mm) in thickness, are required to cover long slots that are in the outer plies of joints. These washers or bars shall have a size sufficient to completely cover the slot after installation.

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	A27 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:

Refer to Articles 1.5.26(A)(3) and 1.5.36.

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 slenderness ratio, "KL/r", 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 force, or to brace or reduce the unbraced length of other members, main or secondary.

In determining the radius of gyration, r, for the purpose of applying the limitations of the "KL/r" ratio, 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 "KL/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.

Actual unbraced length L 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 unbraced 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, webs of plate girders (see Article 1.7.71), closed ribs in orthotropic decks, fillers and in railings, shall be not less than 5/16 inch in thickness. The web thickness of rolled beams, channels, or structural tees shall not be less than 0.23 inch. The thickness of closed ribs in orthotropic decks shall not be less than 3/16 inch.

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 stiffeners and outstanding legs of angles, etc., refer to Article 1.7.15.

For stiffeners 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 T 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 T 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

In all bridges, provisions shall be made in the design to resist thermal stresses induced, or means shall be provided for movement caused by temperature changes. Provisions shall be made for changes in length of span resulting from live load stresses. In spans more than 300 feet (91,440 m) long, allowance shall be made for expansion and contraction in the floor. The expansion end shall be secured against lateral movement.

COMMENTARY ART. 1.7.16

The revision will give the designer permission to determine the temperature range for steel design as is permitted for reinforced and prestressed concrete designs.

1.7.17 – COMBINED STRESSES

All members subjected to both axial compression and bending stresses shall be proportioned to satisfy the following requirements:

$$\frac{f_a}{F_a} + \frac{C_{mx}\, f_{bx}}{\left(1 - \frac{f_a}{F_{ex}'}\right) F_{bx}} + \frac{C_{my}\, f_{by}}{\left(1 - \frac{f_a}{F_{ey}'}\right) F_{by}} \leq 1.0$$

and:

$$\frac{f_a}{0.472\, F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \qquad \text{(At points of support)}$$

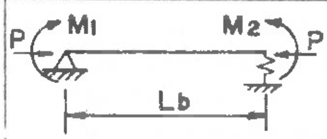
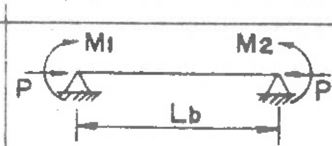
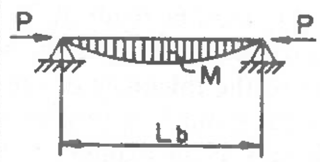
where:

$$F_e' = \frac{\pi^2 E}{F.S. \left(\frac{K_b L_b}{r_b}\right)^2}$$

For

f_a	=	Computed axial stress.
f_{bx} or f_{by}	=	Computed compressive bending stress aboiut the x axis and y axis, respectively.
F_a	=	Axial stress that would be permitted if axial force alone existed, regardless of the plane of bending.
F_{bx} or F_{by}	=	Compressive bending stress that would be permitted if bending moment alone existed about the X axis and Y axis respectively, as evaluated according to Table 1.7.1.
E	=	Modulus of elasticity of steel.
K_b	=	Effective length factor in the plane of bending. See Appendix C.
L_b	=	Actual unbraced length in the plane of bending.
r_b	=	Radius of gyration in the plane of bending.
C_{mx} or C_{my}	=	A coefficient about the X axis and Y axis respectively, whose value is taken from Table 1.7.17A.
F.S.	=	Factor of Safety = 2.12.

TABLE 1.7.17A

Loading conditions	Remarks	C_m
Computed moments maximum at end; joint translation not prevented		0.85
Computed moments maximum at end; no transverse loading, joint translation prevented		$\left[(.4)\frac{M1}{M2}+.6\right] \geq 0.4$
Transverse loading; joint translation prevented		0.85
Transverse loading; joint translation prevented.		1.0

M_1 = Smaller end moment.

$M1/M2$ is positive when member is bent in single curvature.

$M1/M2$ is negative when member is bent in reverse curvature.

In all cases C_m may be conservatively taken equal to 1.0.

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 the 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.

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:

$$\frac{s^2}{4g}$$

where

s = pitch of any two successive holes in the chain.

g = gage of the same holes.

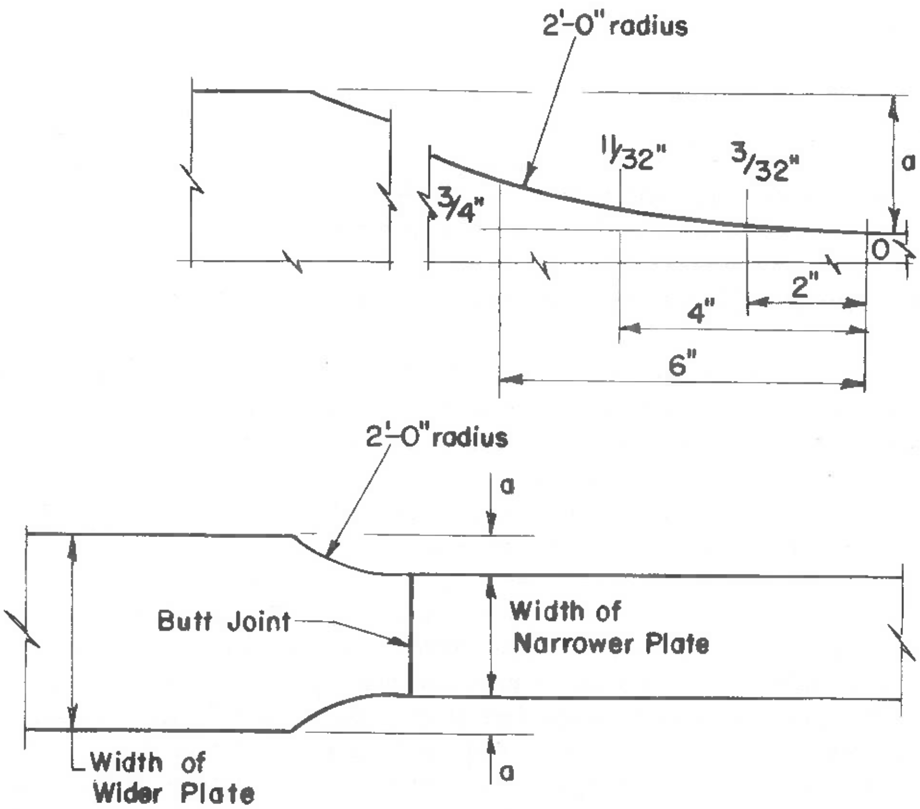
The net section of the part is obtained from the chain which gives the least net width.

At a splice the total stress in the member being spliced is transferred by fasteners to the splice material.

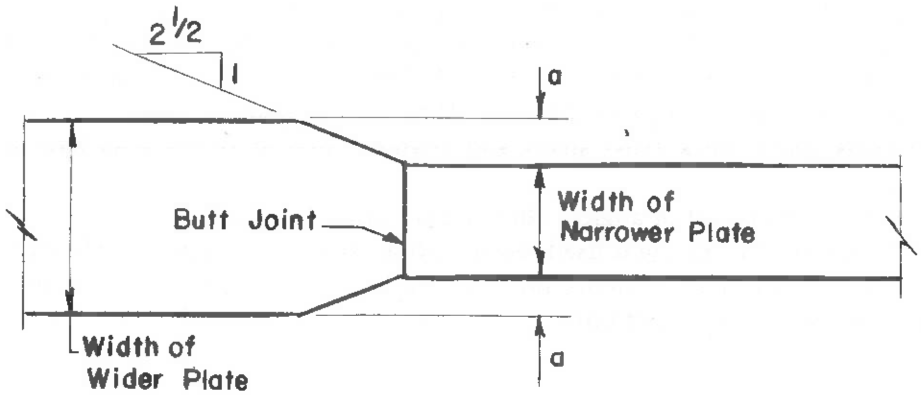
When determining the unit stress on any least net width of either splice material or member being spliced, the amount of the stress previously transferred by fasteners adjacent to the section being investigated shall be considered in determining the unit stress on the net section.

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

FIGURE 1.7.19



(a) 2'–0" Radius Transition



(b) Straight Tapered Transition

SPLICE DETAILS

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 simple 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 nonrigid flooring shall have a bottom lateral systems 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 in outside bays if straight and in alternate bays if curved.

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 2½ 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 36,000 psi Y.P. min.
24 for steel with 42,000 psi Y.P. min.
23 for steel with 46,000 psi Y.P. min.
22 for steel with 50,000 psi 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 Filler 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 1½ 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 thickness 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 T JOINTS

When joining material in skewed T 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 plus 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 stress, the distance between the center of the nearest fastener and that end of 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 proportionately less where the stress per fastener is less than that of the maximum permitted, 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:

$$\frac{AC}{T} \quad \text{for single shear}$$

or:

$$\frac{2AC}{T} \quad \text{for double shear}$$

Where A is the 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 connected part. This end distance may be proportionately less where the shear stress per fastener is less than that permitted in Article 1.7.5, but not less than 1½ times the fastener diameter. It need not exceed 1½ 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:

$$\frac{3}{4}+\frac{1}{4}\,\frac{\text{yield point of steel}}{100{,}000}$$

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 bearings shall be designed to prevent horizontal movement. 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 ¾ 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 two bolts 1-inch in diameter, set 6 inches into the masonry; where flooding may occur, anchor bearings against lateral movement.

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 as specified in Article 1.2.16.

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. 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 percent 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 load 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 two 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 Article 1.7.19. 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. Cover plates wider than the flange to which they are attached must be provided with transverse end welds. The end weld shall be returned around the beam flange.

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 ratio of compression flange plate width to thickness shall not exceed the value determined by the formula:

$$b/t = \frac{3250}{\sqrt{f_b}} \text{ but in no case b/t exceed 24}$$

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 Article 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 ½ in.

The unsupported compression flanges for straight plate girders shall be checked for lateral buckling during erection.

The equation to check buckling stress is:

$$*f_{cr} = \frac{E}{L}\sqrt{\left(\frac{.65 A_f}{D}\right)^2 + \left(\frac{5 W_t^{\,2}}{\left(\frac{A_w}{A_f} + 6\right) L}\right)^2}$$

where

f_{cr}	=	Critical Buckling Stress (kips/sq. in.)
E	=	Modulus of Elasticity of Steel (kips/sq. in.)
L	=	Unsupported Length of Compression Flange (inches)
W_t	=	Width of Compression Flange (inches)
D	=	Total Depth of Girder (inches
A_w	=	Area of Web (sq. in.)
A_f	=	Area of Compression Flange (sq. in.)

A factor of safety shall be provided against buckling of 1.1.

$$F.S. = 1.1 = \frac{f_{cr}}{F_b}$$

Where Fb = Compression stress due to bending (kips/sq. in.)

When it is necessary to increase the width and/or thickness of the compression flange because of potential buckling, it may also be desirable to decrease the web depth to obtain the most economical stringer section.

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:

$$t = \frac{D\sqrt{f_b}}{23{,}000} \qquad \text{(See Figure 1.7.70)}$$

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:

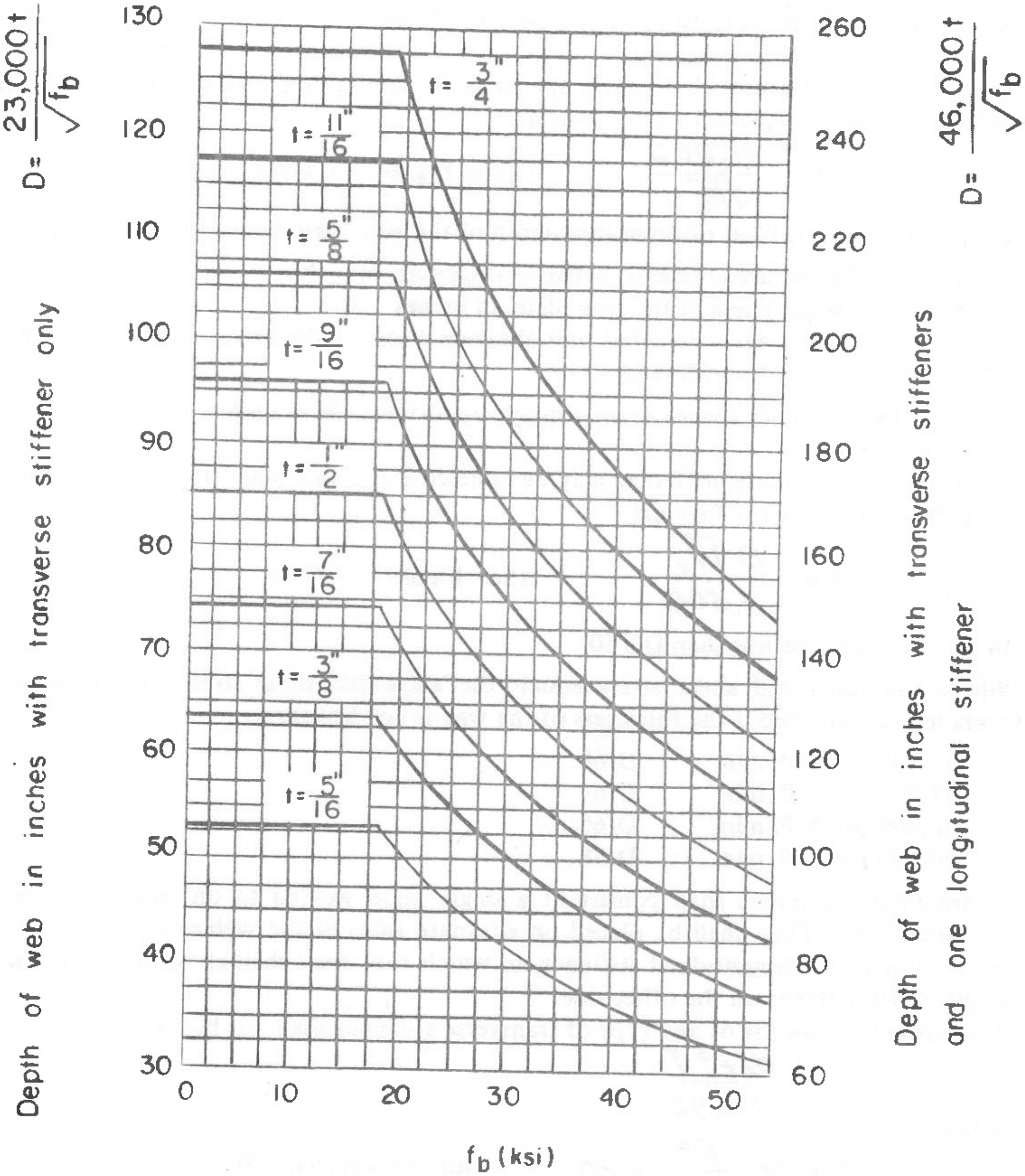
$$t = \frac{D\sqrt{f_b}}{46{,}000} \qquad \text{(See Figure 1.7.70)}$$

but in no case shall the thickness be less than D/340 or less than 1/2 inch.

Where the calculated bending stress in the flange 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 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
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.



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

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:

$$d = \frac{11{,}000}{\sqrt{f_v}} \qquad \text{(See Figure 1.7.71A)}$$

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:

$$t = \frac{D\sqrt{f_v}}{7{,}500} \qquad \text{(See Figure 1.7.71B)}$$

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 be less than:

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

where

$$J = 25\frac{D^2}{d^2} - 20 \qquad \text{but not less than 5.0.}$$

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 flange 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.

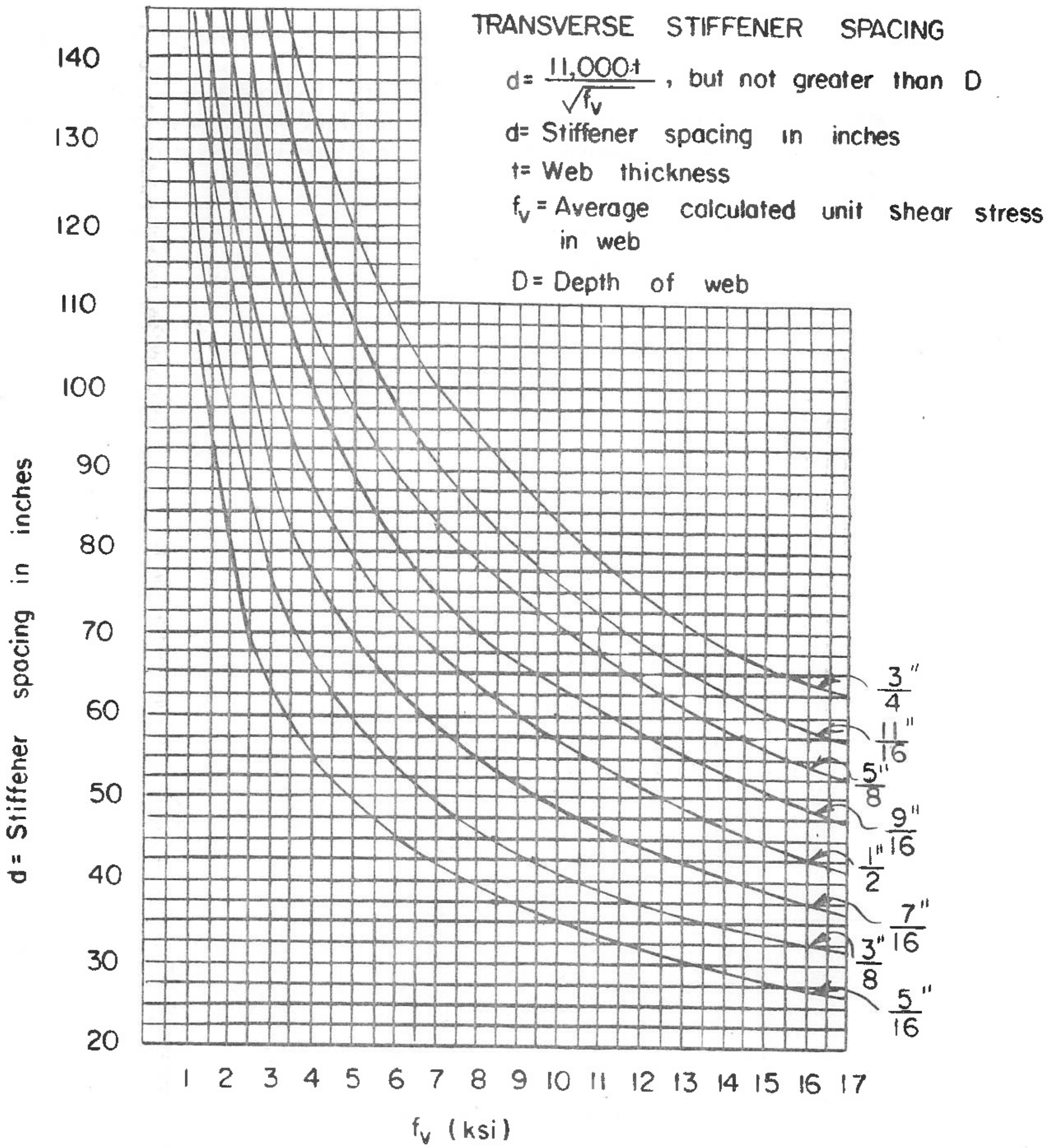


FIGURE 1.7.71A

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

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

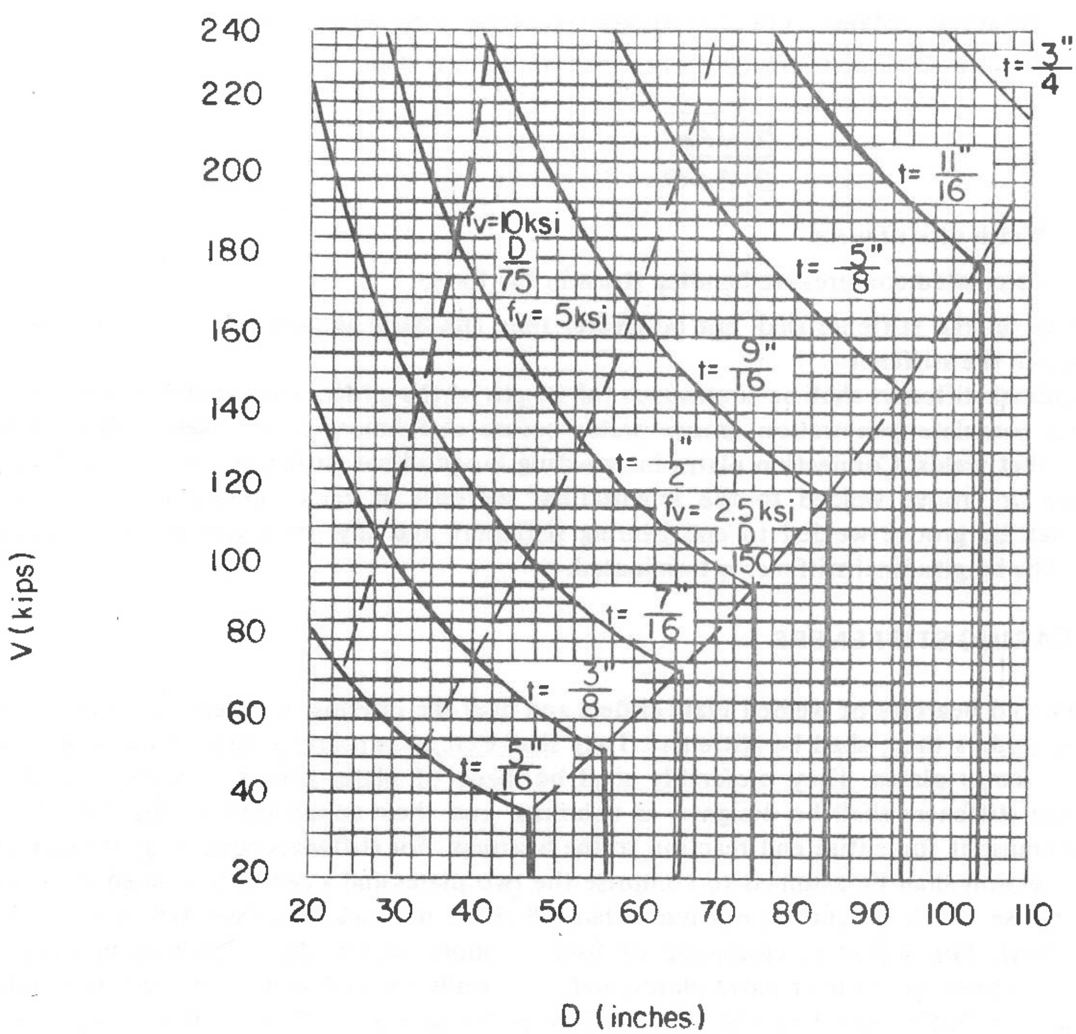


FIGURE 1.7.71B

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:

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

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 distance transverse stiffeners, in inches.

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

$$\frac{b'\sqrt{f_b}}{2,250}$$

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 shall be continuous full length of the girder. They shall be assembled full length using complete penetration groove welds before attachment to the web with full-length continuous fillet welds. Connection plates intersecting longitudinal stiffeners shall be notched and fillet welded or groove welded to the longitudinal stiffener at each intersection. Longitudinal stiffeners shall be groove welded to end bearing stiffeners and any other stiffener or connection plate where the longitudinal stiffener is terminated.

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 Article 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 Article 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. Generally, 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 paltes 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 diphragm. 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 1½ 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

$$\frac{11{,}000}{\sqrt{F_y}} \text{ times its thickness,}$$

the edge shall be stiffened. Below are the values of the expression:

$$\frac{11{,}000}{\sqrt{F_y}}$$

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 other 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, outstanding legs of angles 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 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 the 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 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:

$$b/t = \frac{6000}{\sqrt{f_a}} \quad \text{but in no case shall "b/t" be greater than 55.}$$

(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 of 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 requirments 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:

$D/t = \dfrac{7200}{\sqrt{f_a}}$ 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 falsework 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 Department of Transportation Standard Specifications. 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. The reinforcement steel embedded in the concrete will not be used in computing section properties for negative moments and shear connectors will not be provided in these portions of the span but additional anchorage connectors shall be placed in the region of the point of dead load contraflexure in accordance with Article 1.7.100.

In the negative moment regions of continuous spans, the minimum longitudinal reinforcement including the longitudinal distribution reinforcement must equal or exceed 1 percent of the cross-section area of the concrete slab. Two-thirds of this required reinforcement is to be placed in the top layer of slab within the effective width. Placement of distribution steel as specified in Article 1.3.2E is waived within the effective width.

The longitudinal reinforcement shall be extended into the positive moment region beyond the anchorage connectors at least 40 times the reinforcement diameter.

1.7.100 – SHEAR

A. Horizontal Shear

The maximum pitch of shear connectors 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 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 neutral 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:

$$Z_r = \alpha\, d^2 \qquad \text{in which}$$

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 cross section "Z" by the horizontal range of shear "S_r" 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 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 equal or exceed the number given by the formula:

$$N_1 = \frac{P}{\phi S_u}$$

where N_1 = the number of connectors between points of maximum positive moment and adjacent end supports.

S_u = the ultimate strength of the shear connector as given below.

ϕ = a reduction factor = 0.85.

P = force in the slab as defined hereafter P_1 or P_2.

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 the steel being used.

f'_c = compressive strength of concrete at age of 28 days.

b = effective flange width given in Article 1.7.99.

c thickness of the concrete slab.

The number of connectors N_2 required between the points of maximum positive moment and points of adjacent maximum negative moment shall equal or exceed the number given by the formula:

$$N_2 = \frac{P + P_3}{\phi S_u}$$

At points of maximum negative moment the force in the slab is taken as:

$$P_3 = A_s^r F_y^r$$

where 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:

Channels:

$$S_u = 550\left(h + \frac{t}{2}\right)W\sqrt{f'_c}$$

Welded Studs $(H/d \geq 4)$

$$S_u = 0.4d^2\sqrt{f'_c E_c}$$

where E_c = Modulus of Elasticity of the concrete, psi

S_u = ultimate strength of individual shear connector, in pounds.

h = the average flange thickness of the channel flange, in inches.

t = the thickness of the web of a channel, in inches.

W = length of a channel shear connector, in inches.

f'_c = compressive strength of the concrete at 28 days, psi.

d = diameter of stud, in inches.

w = unit weight of concrete, in lbs. per cu. ft.

3. Additional connectors to develop slab stress.

The number of additional connectors required at points of contraflexure shall be computed by the formula:

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

where N_c = number of additional connectors for each beam at point of contraflexure.

A_rs = total area of longitudinal slab reinforcement steel for each beam over interior support.

f_r = range of stress due to live load plus 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 connector, "N_c" shall be placed adjacent to the point of dead load contraflexure within a distance equal to 1/3 the effective slab width, i.e., placed either side of this point or centered about it. It is preferable to locate field splices so that they clear the connectors.

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 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 parapets) beyond the exterior web, 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.

The provisions of Division I, DESIGN, shall govern where applicable, 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.

$$W_L = 0.1 + \frac{1.7NL}{NB} + \frac{0.85}{NL}$$

where:

NL = Number of Lanes (See Article 1.2.6)
NB = Number of Box Girders

$\frac{NL}{NB}$ 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 equation:

$$V_w = \frac{V_v}{\cos \phi}$$

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 ot 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 the span length. If the flange plate width exceeds one-fifth of the span, an amount equal to one-fifth 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 $0.55F_y$ shall have a width to thickness ratio equal to or less than the value obtained by the use of the formula:

$$\frac{b}{t} = \frac{6140}{\sqrt{f_y}}$$

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:

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

For values of "b/t" exceeding $13{,}000/\sqrt{F_y}$, 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 spacing 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

ϕ = $0.07k^3 n^4$ for value of n greater than 1.
ϕ = $0.125k^3$ for the 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 $0.55F_y$, the ratio "w/t" shall not exceed the value given by the formula:

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

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:

$$f_b = 0.55\,F_y - 0.244\,F_y \left[1 - \sin\left(2.92 - \frac{w\sqrt{F_y}}{2280t\sqrt{k}}\right)\right]$$

For values of "w/t" exceeding $6650\sqrt{k_1} / \sqrt{F_y}$ but not exceeding 60, the stress in the flange, including stiffeners, shall not exceed the value given by the formula:

$$f_b = 14{,}400{,}000k(t/w)^2$$

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.

If the longitudinal stiffeners are placed at their maximum "w/t" ratio in order for the flange to be designed for the basic allowable design stresses of "$0.55F_y$" and the number of longitudinal stiffeners exceeds 2, then transverse stiffeners should be considered.

D. Compression Flanges Stiffened Longitudinally and Transversely

The 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:

$$I_s = 8\,t^3\,w$$

The transverse stiffeners shall be proportioned so that the moment of inertia of each stiffener about an axis through the centroid of the section and parallel to its bottom edge is at least equal to:

$$I_t = 0.10\,(n+1)^3\,w^3\,\frac{f_s A_f}{Ea}$$

where

A_f	=	area of bottom flange including longitudinal stiffeners
a	=	spacing of transverse stiffeners
f_s	=	maximum longitudinal bending stress in the flange of the panels on either side of the transverse stiffener
E	=	modulus of elasticity of steel

For the flange, including stiffeners, to be designed for the basic allowable stress of "$0.55F_y$", the ratio "w/t" for the longitudinal stiffeners shall not exceed the value given by the formula:

$$\frac{w}{t} = \frac{3070\sqrt{k_1}}{\sqrt{F_y}}$$

where:

$$k_1 = \frac{\left[1 + \left(\frac{a}{b}\right)^2\right]^2 + 87.3}{(n+1)^2\left(\frac{a}{b}\right)^2\left[1 + 0.1\,(n+1)\right]}$$

For greater values of w/t, but not exceeding $6650 \sqrt{k_1} / \sqrt{F_y}$ or 60, whichever is less, the stress in the flange, including stiffeners shall not exceed the value given by the formula:

$$f_b = 0.55\, F_y - 0.224\, F_y \left[1 - \sin\left(2.92 - \frac{w\sqrt{F_y}}{2280\, t \sqrt{k_1}}\right)\right]$$

For values of "w/t" exceeding $6650\sqrt{k_1}/\sqrt{F_y}$ but not exceeding 60, the stress in the flange including stiffeners, shall not exceed the value given by the formula:

$$f_b = 14{,}400{,}000 k_1 (t/w)^2$$

The maximum value of the buckling coefficient "k_1" shall be 4. When "k_1" has its maximum value, the transverse stiffeners shall have a spacing, "a", equal to or less than "4w". If the ratio "a/b" exceeds 3, transverse stiffeners are not necessary.

The transverse stiffeners need not be connected to the flange plate but shall be connected to the webs of box and to each longitudinal stiffener. The connection to the web shall be designed to resist the vertical force determined by the formula:

$$R_w = \frac{F_y\, S_s}{2b}$$

where S_s = the section modulus of the transverse stiffener

The connection to each longitudinal stiffener shall be designed to resist the vertical force determined by the formula:

$$R_s = \frac{F_y\, S_s}{nb}$$

E. Compression Flange Stiffeners, General

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

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

where

b' = width of the outstanding stiffener element
t' = thickness of any outstanding stiffener element

Longitudinal stiffeners shall be extended to locations where the maximum stress in the flange does not exceed that allowed for base metal adjacent to or connected by fillet welds.

COMMENTARY

The formulas of Article 1.7.105(C) are expressed graphically in Figure 1.7.105A. This figure relates the required longitudinal stiffener strength to the b/t ratio for various allowable stress levels and number of stiffeners.

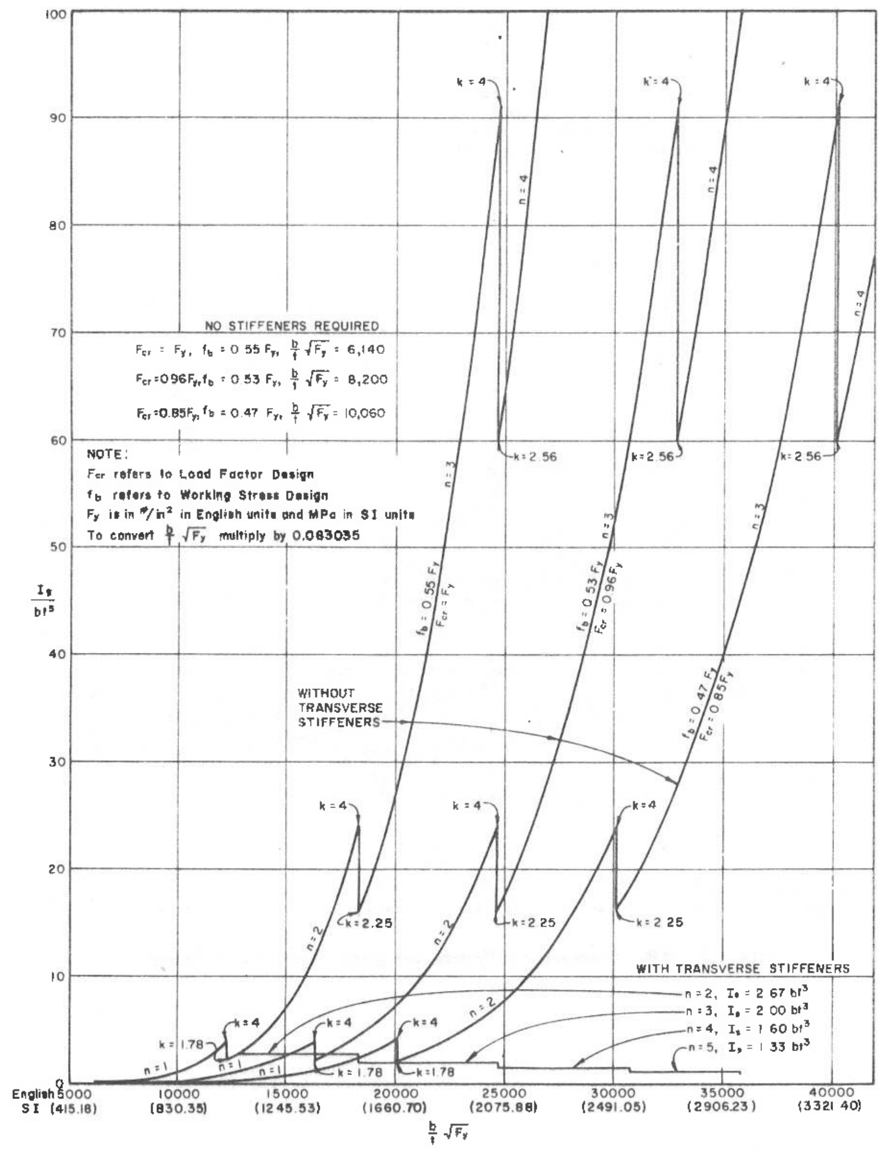


FIG. 1.7.105A—Longitudinal stiffeners box girder compression flange

92

The formulas of Article 1.7.105(D) are expressed graphically in Figure 1.7.105B. This figure relates the required transverse stiffener strength and the a/b ratio to the b/t ratio for various allowable stress levels and number of stiffeners.

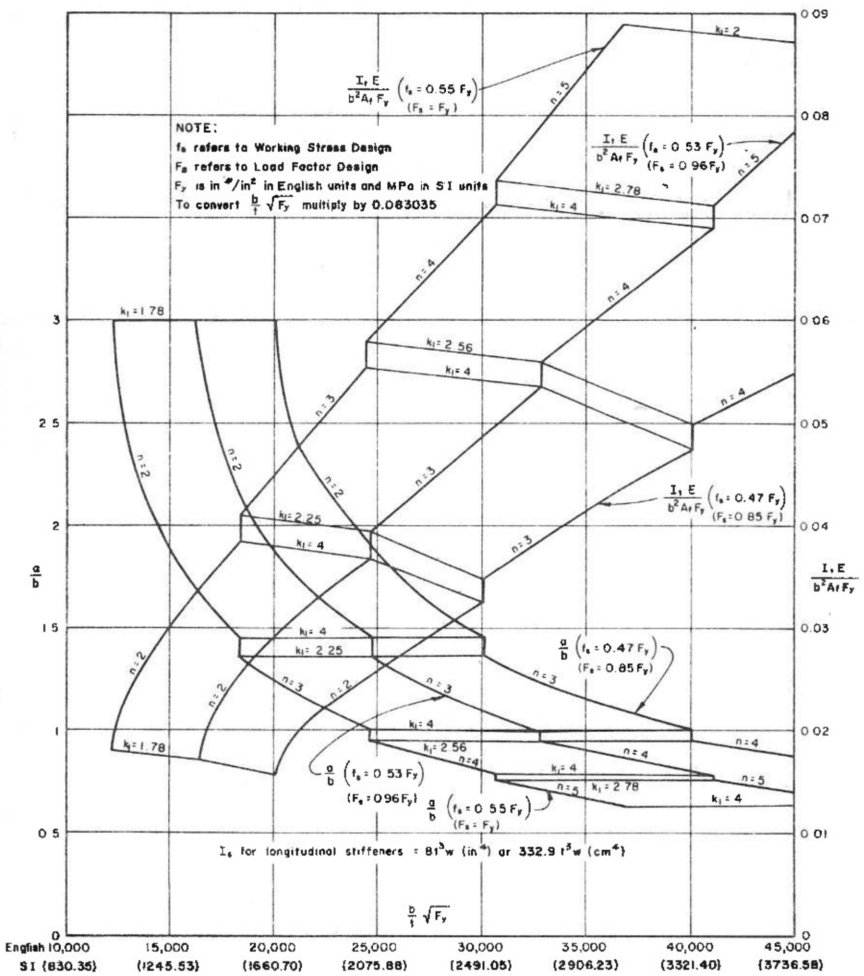


FIG. 1.7.105B—Transverse stiffeners box girder compression flange

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 welds 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. This 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.

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 than the 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 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 Article 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)

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.

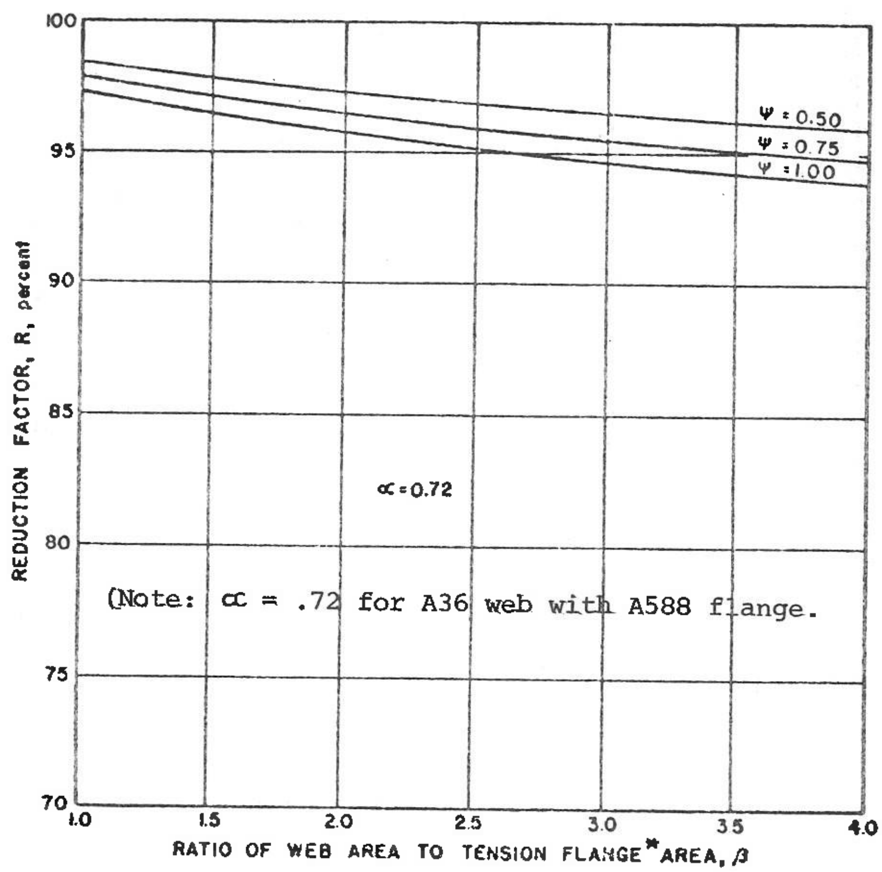
The bending stress in the concrete slab in composite girders shall not exceed the allowable stress for the concrete multiplied by "R."

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 weld connections shall be based on the flange steel.



* Bottom flange of orthotropic deck bridges.

FIGURE 1.7.III A

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 continuoud 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:

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

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

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 L 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 Δ_{DL} is the camber in inches at any point along the length "L" calculated by usual procedures to compensate for deflection due to dead loads or any other specified loads. "Δ_m" is the maximum value of "Δ_{DL}" in inches within the length "L", "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 fro non-symmetrical sections), and "L" is the span length for simple spans or the distance between a simple end support and the dead load contraflexure point; or the distance between points of dead load contraflexure for continuous spans. "L" is measured 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 transverse 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)
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 Article 1.2 (except Article 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 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 member shall be computed from the formulas shown in Sec. 1.2.22. Each part of the structure shall be proportioned for the group loads that are applicable and the maximum design required by the group loading combinations shall be used.

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

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

where

b′ is the width of the projecting flange element,
t is the flange thickness.

*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.

b. Web thickness

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

where

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

c. Lateral bracing

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

or

$$\frac{l_b}{r_y} \leq \frac{12{,}000}{F_y} \quad \text{when } M_2 \leq 0.7M_1$$

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).

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

$$P \leqslant 0.15 F_y A$$

where

A is the area of the cross section.

e. Maximum shear force

$$V \leqslant 0.55 F_y d t_w$$

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 1.7.24(A) are given in Table 1.

TABLE 1

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

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 reductions 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:

$$M_u = F_y S$$

where S is the section modulus.

1. The above equation is applicable to beams meeting the following requirements:
   a. Projecting flange element

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

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

   b. Web thickness

$$D/t_w \leqslant 150$$

where D is the clear unsupported distance between flange components.

   c. Spacing of lateral bracing for compression flange

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

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

$$V \leq \frac{3.5\, E t_w^3}{D}$$

but not more than $0.58 F_y D t_w$

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 $F_y S$.

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 $0.15 F_y A$ 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:

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

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 requirements 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) (41) shall be computed as:

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

where:

$$V_p = 0.58\, F_y\, D t_w$$

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

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:

$$\frac{M}{M_u} = 1.375 - 0.625 \left(\frac{V}{V_u}\right)$$

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:

$$d_o = 14{,}500 \sqrt{\frac{D t_w^{\,3}}{V}}$$

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

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

where b′ is the projecting width of the stiffener.

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

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

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 mid-plane of the web shall be not less than:

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

where:

$$J = 2.5\left(\frac{D}{d_o}\right)^2 - 2 \quad \text{but not less than } 0.5.$$

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 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 stiffeners shall meet the requirement:

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

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 (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:

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

c. The radius of gyration of the stiffener is not less than:

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

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:

$$S_t = \frac{\left(\frac{D}{d_o} \right) S_t}{3}$$

where "D" is the total panel depth (clear distance between flange components) and "S", 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 (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).

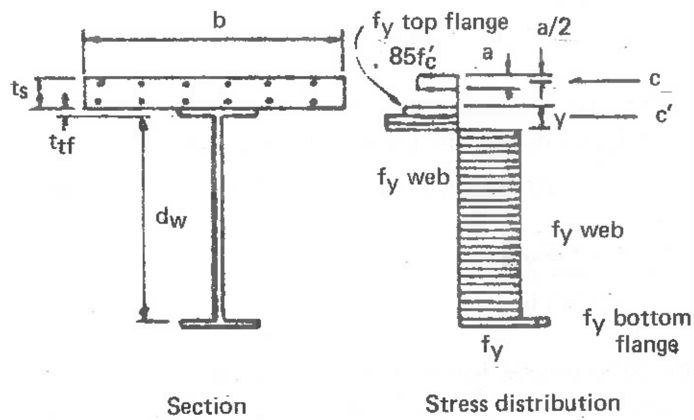


Fig. 1.7.127

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

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. $C = (A\ F_y)_{bf} + (A\ F_y)_{tf} + (A\ F_y)_w$

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. $C = \Sigma Q_u$

where

ΣQ_u is the 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.

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

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

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

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

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

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 along 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:

$$V_w = \frac{V}{\cos \theta}$$

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

$$c = \frac{13{,}000 - \frac{b}{t}\sqrt{F_y}}{7160}$$

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

ϕ = $0.07k^3 n^4$ when n equals 2, 3, 4 or 5.
ϕ = $0.125k^3$ 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:

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

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

$$\rho = \frac{F_{yw}}{F_{yf}}$$

$$\beta = \frac{A_w}{A_f}$$

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:

$$\frac{P}{0.85 A_s F_{cr}} + \frac{MC}{M_u\left(1 - \frac{P}{A_s F_e}\right)} \leq 1.0$$

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

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)

$$F_e = \frac{\pi^2 E}{\left(\frac{KL_c}{r}\right)^2} = \text{the Euler buckling stress in the plane of bending,}$$

C is the equivalent moment factor, as defined below.
$M_p = F_y Z$ 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.

*B.G. Johnston, "Guide to Design Criteria for Metal Compression Members," John Wiley and Sons, Inc., New York, 1966.

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 hv 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.

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.

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[6]	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 7/8 in. The design value listed is for bolts up to 7/8 in. diameter. For diameters greater than 7/8 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.

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 plate intersects the threads.

High-strength bolts preferably shall be used for fasteners subject to tension or combined shear and tension.

For combined tension and shear in bearing type connections, bolts and rivets shall be proportioned so that the shear stress does not exceed:

$$F_{ve} \leq \sqrt{F^2 - (0.6f_t)^2}$$

where

F_y = 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, compresssion, 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 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_yS. For such beams designed for Group IA loading, the moment caused by D+2.2(L+I) shall not exceed 0.8 F_yS. 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 griction-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

$$f_v = 21{,}000\left(1 - \frac{f_t}{0.53\,F_u}\right)$$

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).

3. 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.*

*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.

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 Articles 1.7.1 and 1.7.3.

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 neet 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
- Fy = 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.

Section 8 — CORRUGATED METAL AND STRUCTURAL PLATE PIPES AND PIPE-ARCHES

1.8.1 — GENERAL

The materials for the structure shall conform to the specifications set forth below, and the construction and installation shall conform to Section 23, Division II. The minimum gage or thickness shall be as determined by design in accordance with Article 1.8.2 except that such thickness shall be increased in accordance with Article 1.8.4 to provide for corrosion or abrasion unless there is evidence that corrosion or abrasion is not likely to occur.

Corrugated metal pipe composed of a smooth liner and corrugated shell attached integrally at seams spaced not more than 30 inches apart may be designed in accordance with Article 1.8.1 on the same basis as a standard corrugated metal pipe having the same corrugations as the shell and a weight per foot equal to the sum of the weights per foot of liner and corrugated shell.

This shall be limited to corrugations of 2-2/3" x 1/2" and 3" x 1". The thickness of the corrugated shell shall be at least 60 percent of the total thickness of shell and liner, and the specified backfill compaction shall be a minimum of 85 percent of standard density.

Where corrosion or abrasion are anticipated, thickness of shell and liner shall be increased in accordance with Article 1.8.4 or suitable coatings shall be specified.

Corrugated metal pipe and pipe-arches may be of riveted, welded, or helical fabrication. The specifications are:

	Aluminum	Steel
Riveted	AASHO M 196	AASHO M 36*
Continuous Welded	AASHO M 197	AASHO M 36
Spot Welded	Specification Pending	AASHO M 36
Helical Underdrain Culvert	AASHO M 197 or AASHO M 211	AASHO M36

Structural plate pipe and pipe-arches shall be bolted. The specifications are:

	Aluminum	Steel
Bolted	AASHO M 219	AASHO M 167 (6 x 2 corrugations)

Nothing included in this section shall be interpreted as prohibiting the use of new developments where usefulness can be substantiated.

*For 3 x 1 corrugations an equal number of 1/2 in. round ASTM A 325 bolts may be substituted for rivets.

1.8.2 – DESIGN

Four criteria must be considered in the structural design of a flexible buried conduit. Each considers the mutual function of the metal ring and the soil envelope surrounding it; interaction of these two materials produces a composite structure.

The criteria are:

A. Seam Strength
B. Handling and Installation Strength
C. Failure of the Conduit Wall
D. Deflection or Flattening

A. Seam Strength

Seam strength must be sufficient to withstand the thrust developing from the total load supported by the conduit.

This thrust, in lbs. per lineal ft. of structure is:

$$T = (LL + DL) \times \frac{\text{Span}}{2}$$

where

LL = design live load, psf. See Article 1.3.3
DL = dead load, psf. See Article 1.2.2(A) and 1.8.8
Span (or diameter), in ft.

Thrust, T, multiplied by safety factor, (see Article 1.8.8) should not exceed the seam strength. The strengths shown in Table 1.8.1 are recommended in the determination of fill heights. Longitudinal seams for corrugated metal pipe and pipe-arch shall develop the minimums shown in Table 1.8.1.

Table 1.8.1

Minimum longitudinal seam strengths (ultimate strength in kips per foot)

2 x 1/2 and 2-2/3 x 1/2 Corrugated Steel Pipe

Thickness	Single Rivets	Double Rivets
0.064	16.7	21.6
0.079	18.2	29.8
0.109	23.4	46.8
0.138	24.5	49.0
0.168	25.6	51.3

3 x 1 Corrugated Steel Pipe

Thickness	Double Rivets
0.064	28.7
0.079	35.7
0.109	53.0
0.138	63.7
0.168	70.7

6 x 2 Structural Plate Steel Pipe

Thickness	4 Bolts/Ft.	6 Bolts/Ft.	8 Bolts/Ft.
0.109	42.0		
0.138	62.0		
0.168	81.0		
0.188	93.0		
0.218	112.0		
0.249	132.0		
0.280	144.0	180.0	194.0

2 x 1/2 and 2-2/3 x 1/2 Corrugated Aluminum Pipe

Thickness	Single Rivets	Double Rivets*
0.060	9.0	14.0
0.075	9.0	18.0
0.105	15.6	31.5
0.135	16.2	33.0
0.164	16.8	34.0

9 x 2-1/2 Structural Plate Aluminum Pipe

Thickness	Aluminum Bolts 5 1/3 per ft.	Steel Bolts 5 1/3 per foot
0.09	22.2	
0.10	26.4	
0.125	34.8	
0.15	44.4	
0.175	52.8	
0.20		60.0
0.225		66.0
0.250		72.0

*Pipes with 42" or larger diameters require double rivets.

B. Handling and Installation Strength

Handling and installation strength must be sufficient to withstand impact forces associated with shipping and placing of pipe. Both shop and field assembled pipe must have strength adequate to withstand compaction of the backfill without interior bracing to maintain pipe shape.

Handling rigidity is measured by a flexibility factor determined by the formula:

$$FF = \frac{D^2}{(EI)}$$

where

- D = pipe diameter or maximum span, inches.
- E = modulus of elasticity of the pipe material, psi.
- I = moment of inertia per unit length of cross section of the pipe wall, inches to the 4th power per inch.

For steel conduits, FF should generally not exceed the following values:

2 in. x 1/2 in. and
2-2/3 in. x 1/2 in. corrugation FF = 4.3×10^{-2}
3 in. x 1 in. corrugation FF = 3.3×10^{-2}
6 in. x 2 in. corrugation FF = 2.0×10^{-2}

For aluminum conduits, FF should generally not exceed the following values:

2 in. x 1/2 in. and
2-2/3 in. x 1/2 in. corrugation FF = 9.5×10^{-2}
9 in. x 2-1/2 in. corrugation FF = 2.5×10^{-2}

C. Failure of the Conduit Wall

Failure of the wall by wall crushing may occur if the wall flexibility is low (regardless of the quality of backfill). Failure of the wall by elastic buckling may occur if the wall flexibility is high and the backfill is compressible (poorly consolidated). Interaction of these two failure conditions, crushing and buckling, may develop in a zone between high and low wall flexibility.

It is assumed that a flexible conduit in a "soil-structure interactions system" does not fail at a specific stress defined by bending, since the conduit in yielding may transfer more of its load to the surrounding soil.

The ring compression, at which buckling becomes critical, in the interaction zone for diameters less than

$$\frac{r}{k} \qquad D = \frac{r}{k}\sqrt{\frac{24E}{f_u}}$$

$$f_c = f_u - \frac{f_u^2}{48E}\left(\frac{kD}{r}\right)^2, \text{ PSI}$$

The ring compression stress, at which buckling becomes critical, in the elastic buckling zone for diameters greater than

$$D = \frac{r}{k}\sqrt{\frac{24E}{f_u}}$$

$$f_c = \frac{12E}{\left(\frac{kD}{r}\right)^2}, \ PSI$$

where

- f_u = minimum tensile strength, psi.
- f_c = critical stress, psi, not to exceed the yield strength
- k = soil stiffness factor
- D = pipe diameter or span, in.
- r = radius of gyration (corrugation)
- E = modulus of elasticity, psi.

Design for buckling is accomplished by limiting the ring compression thrust, T, to the buckling stress multiplied by the conduit wall area per linear foot of structure divided by the safety factor.

D. Deflection or Flattening

Deflection or flattening. The Iowa Deflection Formula provides one approach to prediction of ring deflection. It relates ring deflection to the passive side pressure resisting horizontal movement of the pipe wall and to the inherent strength of the pipe. Pipe arches need not be checked for deflection.

The Iowa Deflection Formula is:

$$X = D_l \frac{K W_c R^3}{EI + 0.061 E' R^3}$$

where

- X = horizontal deflection of the pipe, in.
- D_l = deflection lag factor
- K = a bedding constant (depends on bedding angle).
- W_c = vertical load per unit length of pipe, lb/lin. in.
- E = modulus of elasticity of pipe, psi (see Article 1.8.3)
- R = mean radius of pipe, in.
- I = moment of inertia per unit length of cross section of pipe wall, inches to the fourth power per in.
- E' = horizontal soil modulus, psi/in.

Other methods are available for predicting ring deflection.

1.8.3 – CHEMICAL AND MECHANICAL REQUIREMENTS

A. Aluminum – Corrugated Metal Pipe and Pipe-Arch

Chemical – AASHO M 196 (ASTM C 478) and M 197
Mechanical

Thickness, In.	Minimum Tensile Strength PSI	Minimum Yield Strength PSI	Minimum Elongation 2 inches	Modulus of Elasticity PSI
0.051 to 0.113	31,000	24,000	4%	10×10^6
0.114 to 0.249	31,000	24,000	5%	10×10^6

B. Aluminum – Structural Plate Pipe and Pipe-Arch

Chemical – AASHO M 219 ALLOY 5052
Mechanical

Thickness Inches	Minimum Tensile Strength psi	Minimum Yield Strength psi	Minimum Elon-gation 2 inches	Modulus of Elasticity psi
0.090 to 0.175	35,500	28,000	6%	10×10^6
0.175 to 0.250	34,000	26,000	6%	10×10^6

C. Steel – Corrugated Metal Pipe and Pipe-Arch

Chemical – AASHO M 36
Mechanical

Minimum Tensile Strength psi	Minimum Yield Strength psi	Minimum Elon-gation 2 inches	Modulus of Elasticity psi
45,000	33,000	20%	29×10^6

D. Steel – Structural Plate Pipe and Pipe-Arch

Chemical – AASHO M 167
Mechanical

Minimum Tensile Strength psi	Minimum Yield Strength psi	Minimum Elon-gation 2 inches	Modulus of Elasticity psi
42,000	28,000	30%	29×10^6

The mechanical properties shown above are for flat material prior to corrugating. A certificate of compliance shall be required from the manufacturer.

1.8.4 – ABRASIVE OR CORROSIVE CONDITIONS

For corrugated metal and structural plate pipes and pipe-arches having a thickness less than 0.25 inches, the entire conduit, or bottom plate only in the case of structural plate pipe, shall be of greater thickness, or protected by some other means, when required for resistance to abrasion or corrosion.

1.8.5 – RIVETS AND BOLTS

Rivets for corrugated sections and bolts for structural plate sections shall conform with the following:

Aluminum Corrugated Section:
Rivets-Aluminum, ASTM B 316, Alloy 6053-T4

Aluminum Structural Plates:
Bolts-Aluminum, ASTM B 211, Alloy 6061-T6
Bolts-Steel, AASHO M 164 (ASTM A 325)

Steel Corrugated Section:
Rivets-Steel, AASHO M 36

Steel Structural Plates:
Bolts-Steel, AASHO M 164 (ASTM A 325)

Where end treatment requires a rigid headwall, the plates or pipe shall be anchored to the headwall with not less than 3/4 inch anchor bolts at not more than 19 inch centers. Steel bolts for structural plate sections shall be torqued during installation to a minimum of 100 ft-lbs. and a maximum of 300 ft-lbs. Aluminum bolts for structural plate sections shall be torqued during installation to a minimum of 100 ft-lbs., and a maximum of 150 ft-lbs. For power-driven tools, the hold-on period may vary from 2 to 5 seconds.

Bolts shall be of sufficient length to provide for a full nut.

1.8.6 – MULTIPLE STRUCTURES

Where multiple lines of pipes or pipe-arches greater than 48 inches in diameter or span are used, they shall be spaced so that adjacent sides of the pipe shall be at least one-half diameter or 3 feet apart, whichever is less, to permit adequate compaction of backfill material. For diameters up to 48 inches, the minimum spacing shall be not less than 24 inches.

1.8.7 – SLOPED ENDS – SKEWED

When the skew angle exceed 20 degrees and the structure has the ends cut to fit the slope, the ends shall be reinforced.

1.8.8 – MAXIMUM DEPTHS OF COVER

The maximum depths of cover may be determined by use of the Iowa Deflection Formula and the following basic data. (Nothing included herein shall prohibit the use of other appropriate basic values.)

Weight of Embankment - 100 lbs/cu. ft.

K = 0.44, soil stiffness coefficient for good side fill material compacted to 85 percent of standard density based on AASHO Specification T 99 (ASTM D 698).

E^1 = Modulus of passive soil (side fill) resistance: 700 psi.

Elongation:
5 percent of nominal diameter

Maximum deflection:
5 percent of nominal diameter below circular shape

Safety factors used:
Longitudinal test seam strength = 4.0
Pipe wall buckling = 2.0

For pipe-arch structures placed on a stable foundation, the confining backfill must be capable of supporting a corner pressure of 2 tons per square foot. Marginally stable or compressible foundations require special investigation. Fill heights exceeding 100 feet shall be used only after a thorough investigation of the foundation material.

Section 9 – STRUCTURAL PLATE ARCHES

1.9.1 – GENERAL

Structural Plate Arches shall conform to Section 8, Division I, and to the specifications set forth below.

1.9.2 – RATIO, RISE TO SPAN

The design of single radius structural plate arches should be based on ratios of rise to span varying from 0.3 to 0.5.

1.9.3 – MINIMUM HEIGHT OF COVER

The minimum cover for design loads shall be $\frac{\text{Span}}{6}$ but not less than 12″.

1.9.4 – SCOUR CONDITIONS

Invert slabs shall be provided when scour is anticipated.

1.9.5 – MULTIPLE ARCHES

Where multiple arch spans are used, the distance between plates shall not be less than 1/10 of the longer adjoining span.

1.9.6 – SUBSTRUCTURE DESIGN

The substructure shall be designed according to specifications herein for substructures of bridges.

Section 10 – TIMBER STRUCTURES

1.10.1 – ALLOWABLE STRESSES

A. Allowable Unit Stresses for Stress-Grade Lumber*

The allowable unit stresses given in Table 1.10.1 are for normal duration of loading for stress grades of sawn lumber used under continuously dry conditions as in most covered structures. For other service conditions, the following modification shall apply. For lumber used under conditions in which the moisture content of the wood is at or above the fiber saturation point, as when continuously submerged, the allowable unit stresses in Table 1.10.1 in compression parallel to the grain shall be reduced 10 percent, in compression perpendicular to the grain shall be reduced one-third, and the values for the modulus of elasticity shall be reduced one-eleventh.

1. Use of Stress Grades in Flexure:

Allowable unit stresses in flexure for Joist and Plank grades apply to material with the load applied to either the narrow or wide face.

Allowable unit stresses in flexure for Beam and Stringer grades apply only to material with the load applied to the narrow face.

Beam grades ordinarily are graded for use on simple spans. When used as a continuous beam the grading provisions customarily applied to the middle third of the length of simple spans shall be applied to the middle two-thirds of the length of pieces to be used over double spans and to the entire length of pieces to be used over three or more spans.

2. Modification for Condition of Use for Bearing Perpendicular to Grain:

The allowable unit stresses for compression perpendicular to the grain assume the material will be surface seasoned when installed. When used under continuously wet conditions, the tabulated values should be reduced one-third.

B. Allowable Unit Stresses for Glued Laminated Timber

1. The allowable unit stresses for softwood species shall be as recommended in American Institute of Timber Construction 203-70 "Standard Specifications for Structural Glued Laminated Timber of Douglas Fir, Western Larch, Southern Pine and California Redwood," and as given in Tables 1.10.1(A) and 1.10.1(B) herein. For hardwood species, the allowable unit stresses shall be as given in Table 2.10 of the "Timber Construction Manual," by the American Institute of Timber Construction, published by John Wiley & Sons, New York City, New York.

2. The stress tables given in AITC 203-70 are divided into sections for dry-use or wet-use conditions. Allowable unit stresses for dry-use conditions are applicable when the moisture content in service is less than 16% as in most covered structures. Allowable unit stresses for wet-use conditions are applicable when the moisture content in service is 16% or more, as may occur in exterior or submerged construction, and in some structures housing wet processes or otherwise having constant high relative humidities.

*Tables 1.10.1, 1.10.1A, 1.10.1B, together with their footnotes, have been deleted from this manual. If it is necessary to refer to these Tables, they can be found in the Standard Specifications for Highway Bridges, adopted by the American Association of State Highway Officials – Eleventh Edition – 1973.

3. The stress tables in AITC 203-70 give stresses for members stressed primarily in bending (load applied perpendicular to the wide face of the lamination) and for members stressed primarily in axial tension, axial compression or loaded in bending parallel or perpendicular to the wide face of lamination. Edge joints shall be glued only in members loaded normal to the lamination edges or in members where torsion is a significant design consideration.

4. Slope of grain, type and location of end joints, and other requirements, together with certain manufacturing requirements, must be met for these allowable unit stresses to apply. The requirements for slope of grain for softwoods are given in AITC 203-70, whereas these requirements for hardwoods are incorporated in Table 2.10 of the AITC "Timber Construction Manual." Other requirements are given in U.S. Commercial Standard 253-63.

5. Species other than those specifically included herein may be used provided allowable unit stresses are established for them in accordance with U.S. Commercial Standard 253-63.

C. Allowable Unit Stresses for Normal Loading Conditions.

The tabulated allowable unit stresses are for normal load duration which contemplates fully stressing a member to the allowable unit stress by the application of the full design load for a duration of approximately ten years (either continuously or cumulatively). For other loading conditions, adjustments should be made as given in the following sections.

D. Allowable Unit Stresses for Permanent Loading

When a member is fully stressed to the maximum allowable stress for long term loading conditions (greater than ten years either continuously or cumulatively), use 90 percent of the tabulated allowable unit stresses. The provisions of this paragraph do not apply to modulus of elasticity.

E. Allowable Unit Stresses for Wind, Earthquake or Short Time Loading

When the duration of full maximum load resulting from short time loading, other than live load, does not exceed the period indicated, increase the tabulated unit stresses as follows:

15 percent for static load for 2 months' duration, such as snow or ice
25 percent for static load for 7 days' duration, such as snow or ice
33 1/3 percent for wind or earthquake.

The above increases are not cumulative. The resulting structural members shall be smaller than required for a longer duration of loading. The provisions of this paragraph do not apply to modulus of elasticity. The increases apply to mechanical fastenings except as otherwise noted.

F. Combined Stresses

These specifications do not cover the application of loadings which produce combined axial and bending stresses, nor the effective reductions in the tabulated stresses as a result of these loadings.

For this condition, attention is directed to Section 4, page 11, of the "Timber Construction Manual," 1966, by the American Institute of Timber Construction, published by John Wiley & Sons, New York, N.Y.

1.10.2 – FORMULAS FOR THE COMPUTATION OF STRESSES IN TIMBER

In calculating live load stresses in timber, impact shall be neglected. See Article 1.2.12(B).

A. Horizontal Shear in Beams

Horizontal shear in beams shall be computed from the maximum shear occurring at a distance from the support equal to three times the depth of the beam, or at the quarter point, whichever is the lesser distance from the support. The live load used in computing horizontal shear shall be placed so as to produce maximum external shear at this distance from the support. This external live load shear shall be one-half the sum of 60 per cent of the shear from the undistributed wheel loads and of the shear from the wheel loads distributed laterally as specified for moment in Article 1.3.1. For undistributed wheel loads, one line of wheels is assumed to be carried by one beam.

The shear shall be calculated according to the following formula:

$$f_v = \frac{3V}{2bd}$$

where

f_v = horizontal shear stress in pounds per square inch
b = width of beam in inches
d = depth of beam in inches
V = vertical shear in pounds

B. Secondary Stresses in Curved Glued Laminated Members

1. Curvature Factor

For the curved portion of members, the allowable stress in bending shall be modified by multiplication by the following curvature factor:

$$1\text{-}2000\,(t/R)^2$$

in which

t = thickness of lamination in inches
R = radius of curvature of a lamination in inches

and t/R should not exceed 1/125 for Douglas fir, larch and California redwood and 1/1000 for Southern pine.

No curvature factor shall be applied to stress in the straight portion of an assembly regardless of curvature elsewhere.

2. Radial Tension or Compression

The radial stress, f_v induced by a bending moment in a curved member, shall be defined by the following equation:

$$f_v = \frac{3M}{2Rbd}$$

where

M = bending moment in inch pounds
R = radius of curvature at centerline of members in inches
b = width of cross section in inches
d = depth of cross section in inches

When M is in the direction tending to decrease curvature (increase the radius), the stress is tension across the grain. For this condition, the tension stress across the grain shall be limited to 1/3 the allowable unit stress in horizontal shear for Southern pine and California redwood for all load conditions and for Douglas fir and larch for wind or earthquake loadings. The limit shall be 15 psi for Douglas fir and larch for other types of load. These values are subject to modifications for duration of load. If these values are exceeded, mechanical reinforcing shall be used and shall be sufficient to resist all radial tension stresses.

When M is in the direction tending to increase curvature (decrease the radius), the stress is compression across the grain and shall be limited to the allowable unit stress in compression perpendicular to the grain for all species included herein.

C. Compression or Bearing Perpendicular to Grain

The allowable unit stresses for compression perpendicular to the grain apply to bearings of any length at the ends of the beam, and to all bearings 6 inches or more in length at any other location. When calculating the bearing area at the ends of beams, no allowance shall be made for the fact that, as the beam bends, the pressure upon the inner edge of the bearing is greater than at the end of the beam.

For bearings of less than 6 inches in length and not nearer than 3 inches to the end of a member, the maximum allowable load per square inch is obtained by multiplying the allowable unit stresses in compression perpendicular to grain by the following factor:

$$\frac{L + \frac{3}{8}}{L}$$

in which L is the length of bearing in inches measured along the grain of the wood.

The multiplying factors for indicated lengths of bearing on such small areas as plates and washers become:

In using the preceding formula and table for round washers or bearing areas, use a length equal to the diameter.

D. Simple Solid Column Design

These formulas for simple solid columns are based on pin-end conditions but shall be applied also to square-end conditions.

Allowable unit stresses in pounds per square inch of cross-sectional area of simple solid columns shall be determined by the following formula, but such unit stresses shall not exceed the tabular values for compression parallel to grain, F_e as provided in Article 1.10.1 and adjusted in accordance with the applicable provisions of Article 1.10.1:

$$F'_c = \frac{\pi^2 E}{2.727\left(\frac{\ell}{r}\right)^2} = \frac{3.619\,E}{\left(\frac{\ell}{r}\right)^2}$$

where

F'_c = allowable unit stress in compression parallel to grain, in psi, adjusted for l/d ratio
E = modulus of elasticity, psi
ℓ = unsupported overall length, in inches, between points of lateral support of simple columns
r = least radius of gyration of section

For columns of square or rectangular cross-section, this formula becomes:

$$F'_c = \frac{0.30E}{\left(\frac{\ell}{d}\right)^2}$$

where d = dimensions of least side of simple solid column in inches

For simple solid columns, the ℓ/d ratio may not exceed 50.
The values of F'_c as determined from the formulae listed are subject to adjustment for duration of load as given in Articles 1.10.1(D) and 1.10.1(E).

E. Spaced Column Design

Spaced columns are formed of two or more individual members with their longitudinal axes parallel, separated at the ends and middle points of their length by blocking and joined at the ends by timber connectors capable of developing the required shear resistance. To obtain spaced column action, end blocks with connectors and spacer blocks are required when the individual members of a spaced column assembly have an l/d ratio greater than

$$\sqrt{\frac{0.30E}{F_c}}$$

For an assembly of members having a lesser l/d ratio, the individual members are designed as simple solid columns. Spaced columns are classified as to fixity, i.e., condition "a" or condition "b," which introduces a multiplying factor applicable in the design of its individual members.

For individual members of a spaced column, ℓ/d shall not exceed 80, nor shall ℓ/d exceed 40.

The individual members in a spaced column are considered to act together to carry the total column load. Each member is designed separately on the basis of its l/d ratio.

A greater l/d ratio than allowed for simple solid columns is permitted because of the end fixity developed by the connectors and end blocks. This fixity is effective only in the thickness direction. The l/d ratio in the direction of width is subject to the provisions for simple solid columns.

When a single spacer block is located within the middle tenth of the column length (l), connectors are not required for this block. If there are two or more spacer blocks, connectors are required and the distance between two adjacent blocks shall not exceed one-half the distance between centers of connectors in the end blocks.

For spaced columns used as compression members of a truss, a panel point which is stayed laterally shall be considered as the end of the spaced column, and the portion of the web members, between the individual pieces making up a spaced column, may be considered as the end blocks.

If there are two or more connectors in a contact face, the center of gravity of the group shall be used in measuring the distance from connectors in the end block to the end of the column for determining fixity condition "a" or "b."

Thickness of spacer and end blocks shall not be less than that of individual members of the spaced column, nor shall thickness, width, and length of spacer and end blocks be less than required for connectors of a size and number capable of carrying the load computed in accordance with Article 1.10.2(I). Blocks thicker than a side member do not appreciably increase load capacity.

The total allowable load for a spaced column is the sum of the allowable loads for each of its individual members. Allowable unit stresses shall be determined as follows, but the maximum unit stress shall not exceed the values for compression parallel to grain "F_c" in Table 1.10.1, or as tabulated in the reference listed in Article 1.10.1(B)(1), and as adjusted in accordance with provisions of Article 1.10.1, nor shall the load exceed that permitted by the following provisions.

The net section shall be determined by subtracting, from the full cross-sectional area of the timber, the projected area of that portion of the connector groove within the members and that portion of the bolt hole not within the connector groove located at the critical plane. (See Table 2.20.1 for typical dimensions for Timber Connectors). Where connectors are staggered, adjacent connectors, with parallel-to-grain spacing equal to or less than one connector diameter, shall be considered as occurring at the same critical section.

In tension and compression members the required net area, in square inches, shall be determined by dividing the total load transferred through the critical section by the allowable tension stress for tension members, or by the allowable compression parallel to grain stress for compression members, for the species and grade of lumber used.

For condition "a", the allowable unit stress for individual members of a spaced column, in which the connectors in end blocks are placed at a distance not exceeding 1/20 from the ends, shall be determined by the formula:

$$F_c = \frac{0.75E}{\left(\frac{l}{d}\right)^2}$$

For condition "b", the allowable load for the individual members of a spaced column in which the connectors in end blocks are placed a distance of 1/20 to 1/10 from the ends and the blocks extend to the ends of the column shall be determined by the formula:

$$F_c = \frac{0.90E}{\left(\frac{l}{d}\right)^2}$$

The total load capacity determined by the foregoing procedure should be checked against the sum of the load capacities of the individual members taken as simple solid columns without regard to fixity, using their greater d and the l between the lateral supports which provide restraint in a direction parallel to the greater d.

The values for F_c, as above determined, are subject to the duration of load adjustments as provided in Article 1.10.1

F. Safe Load on Round Columns

The safe load on a round column shall not exceed that permitted for a square column of the same cross-sectional area, or as determined by the formula:

$$F'_c = \frac{3.619E}{\left(\frac{l}{r}\right)^2}$$

The values for F'_c determined by the formula may not exceed the values for compression parallel to the grain, F'_c adjusted for service conditions and duration of load in accordance with Article 1.10.1. The allowable unit stress values as determined from the formula are subject to the duration of loading adjustments as given in Articles 1.10.1(D) and 1.10.1(E).

In determining the least dimension, d, for tapered columns, the diameter of a round column or the least dimension of a rectangular column, tapered at one or both ends, is taken as the sum of the minimum diameter or least dimensions and one-third the difference between the minimum and maximum diameters or lesser and greater dimensions.

G. Notched Beams

Beams notched upward in the bearing face on supports shall be limited to maximum end load R as determined by the formula:

$$R = \frac{2bd^2 F_v}{3h}$$

R = maximum end load

F_v = allowable unit horizontal shear stress

b = breadth of beam

d = depth of beam above the notch

h = total depth of beam

H. Bearing on Inclined Surfaces

$$N = \frac{PQ}{P \sin^2 \Theta + Q \cos^2 \Theta}$$

N = unit bearing on an inclined surface
P = unit stress in compression parallel to the grain
Q = unit stress in compression perpendicular to the grain
Θ = angle in degrees between the direction of load and the direction of grain

I. Timber Connectors

Timber connectors shall consist of devices to be used at surfaces of contact in bolted timber joints, to increase the strength or shear resistance of wood-to-wood or wood-to-steel connections.

Allowable loads, spacing of connectors, edge and end distance, bolt and washer sizes and other details of design shall be those recommended or approved by the "Design Manual for Timber Connector Construction," 1970, Timber Engineering Company; or the allowable loads may be determined by actual tests of full size joints for each condition of connector used in accordance with standard procedure.

J. Size Factor

When the depth of a rectangular beam exceeds 12 inches, the tabulated unit stress in bending F_b, shall be reduced by multiplying the tabulated stress by the size factor, C_f, as determined from the following relationship

$$C_f = (12/d)^{1/9}$$

where C_f = size factor
d = depth of member in inches

The size factor relationship as given above is applicable to a bending member satisfying the following basic assumptions: (a) simply supported beam, (b) uniformly distributed load, and (c) span to depth ratio (ℓ/d) of 21. This factor can thus be applied with reasonable accuracy to most commonly encountered design situations. Where greater accuracy is desired for other sizes and conditions of loading, the percentage changes given in the following table may be applied directly to the size factor calculated for the basic conditions as previously stated. Straight line interpolation may be used for other ℓ/d ratios.

Span to Depth Ratio ℓ/d	% Change	Loading Condition for Simply Supported Beams	% Change
14	+2.3	Center Point	+7.8
24	−1.6	Third Point	−3.2

For more detailed analysis of the size factor and its application to the design of bending members, the designer is referred to the AITC "Timber Construction Manual." The reduction in bending stresses for deep members based on the size effect factor is only applicable to glued laminated members.

K. Lateral Stability

1. The tabulated allowable unit bending stresses given under Article 1.10.1 are applicable to members which are adequately braced. When deep, slender members not adequately braced are used, a reduction to the allowable unit bending stresses must be applied based on a computation of the slenderness factor of the member. In the check of lateral stability, the slenderness factor is computed by the relationship:

$$C_s = \sqrt{\frac{\ell_e d}{b^2}}$$

where C_s = slenderness factor
ℓe = effective length of beam, in. (see table below)
d = depth of beam, in.
b = breadth of beam, in.

EFFECTIVE LENGTH OF GLUED LAMINATED BEAMS

Type of Beam Span and Nature of Load	Value of Effective Length, ℓe[1]
Single span beam, load concentrated at center	1.61 ℓ
Single span beam, uniformly distributed load	1.92 ℓ
Single span beam, equal end moments	1.84 ℓ
Cantilever beam, load concentrated at unsupported end	1.69 ℓ
Cantilever beam, uniformly distributed load	1.06 ℓ
Single span or cantilever beam, any load (conservative value)	1.92 ℓ

[1]Where = unsupported length.

2. Beams with Various Lateral Support Conditions

a. Without lateral support. When the depth of a beam does not exceed its breadth, no lateral support is required and the allowable unit stress is determined by applying the appropriate provisions of Articles 1.10.1 and 1.10.2.

b. With lateral support. If lateral movement of the compression flange is prevented by a continuous support, there is no danger of lateral buckling, and the allowable stresses require no reduction based on a slenderness ratio concept. Also, there is no need to limit the depth-breadth ratio to 5 or 6.

c. When the depth of a beam exceeds the breadth, bracing must be so arranged as to prevent rotation of the beam at those points in a plane perpendicular to its longitudinal axis. The allowable stresses are calculated by the formulae contained in the following paragraphs for short, intermediate and long beams.

3. The allowable unit stresses are determined from the following equations:

a. Short beams. When the slenderness factor, C_s, does not exceed 10, the tabular allowable unit stress in bending, F_b, adjusted in accordance with the applicable provisions of Articles 1.10.1(D), 1.10.1(E), 1.10.2(B) and 1.10.2(J) is used for design.

b. Intermediate beams. When the slenderness factor, C_s, is greater than 10 but does not exceed C_k, a unit stress in bending based on slenderness considerations, F_b', is calculated by the formula:

$$F_b' = F_b \left[1 - \frac{1}{3}\left(\frac{C_s}{C_k}\right)^4\right]$$

where F_b = tabular allowable unit stress in bending, psi

$$C_k = \sqrt{\frac{3E}{5F_b}}$$

E = modulus of elasticity, psi

c. Long Beams. When the slenderness factor, C_s, is greater than C_k, but less than 50, the unit stress in bending is calculated by the formula:

$$F_b' = \frac{0.40E}{(C_s)^2}$$

Note: in no case shall C_s be greater than 50.

For both intermediate and long beams, the allowable unit stress for design based on slenderness considerations is obtained by adjusting F_b' in accordance with the applicable provisions of Articles 1.10.1(D), 1.10.1(E) and 1.10.2(B).

Regardless of the slenderness classification into which a beam may be categorized, in no case shall the allowable unit stress in bending used for design exceed the value as obtained by adjusting the tabular allowable unit stress based on the applicable provisions of Articles 1.10.1(D), 1.10.1(E), 1.10.2(B) and 1.10.2(J).

1.10.3 – GENERAL

All wood used in timber structures shall be preservatively treated as provided in Division II, Section 21, unless otherwise specified.

1.10.4 – BOLTS

Bolts shall be spaced center to center not closer than 4 times the bolt diameter. The distance from the center of a bolt to the end of any timber shall be not less than 7 times the bolt diameter if loaded in tension parallel to grain, nor 4 times the bolt diameter if loaded in compression parallel to the grain or in tension or compression perpendicular to the grain. For parallel to grain loading in tension or compression, the distance from any edge of the timber to the center of the nearest bolt shall be at least 1 1/2 times the bolt diameter, except that for /d ratios more than 6, use one-half the distance between rows of bolts. For perpendicular to grain loading, the edge distance toward which the load is acting shall be at least 4 times the bolt diameter and the edge distance on the opposite edge shall be at least 1 1/2 bolt diameters.

1.10.5 – WASHERS

A washer shall be used under all bolt heads and nuts which would otherwise come in contact with wood. Either cast or plate washers may be used and they shall be designed to prevent excessive crushing of the wood when the bolts are tightened. For bolts or rods in tension, washers shall be sufficient size to develop the tension stress in the bolt or rod without exceeding the allowable unit stress in compression perpendicular to grain for the species and grade of lumber used.

A standard circular washer shall be used under the heads of all lag screws.

1.10.6 – HARDWARE FOR SEACOAST STRUCTURES

The hardware for structures on the seacoast shall be galvanized or cadmium plated.

1.10.7 – COLUMNS AND POSTS

No column shall have an unsupported length greater than 50 times its least dimension.

The strength of built-up columns composed of two or more sticks bolted together, either with or without packing blocks, shall be considered as equal to the combined strength of the single sticks each considered as an independent column.

The strength of connector-joined spaced columns shall be determined as provided in Article 1.10.2(I).

1.10.8 – PILE AND FRAMED BENTS

A. Pile Bents

Pile bents generally shall not exceed 40 feet in height. Pile bents over 10 feet high shall be sway-braced transversely with diagonal braces on each side of the bent, and shall be adequately braced longitudinally. In general, pile bents shall contain not less than four piles each and the outside piles, preferably, shall be battered. The piles shall be designed for safe bearing and for column action.

B. Framed Bents

Framed bents may be supported on piles, concrete pedestals or mud sills. All bents shall be sway-braced transversely and adequate provision shall be made for longitudinal bracing. In general, framed bents shall contain not less than four posts each and the outside posts of the bent shall be battered. The posts shall be designed as columns.

C. Sills and Mud Sills

When possible, sills shall be located clear of all earth so that there may be a free circulation of air around them. Sills shall be fastened to mud sills or piles with drift bolts of not less than 3/4-inch diameter and extending into the mud sills or piles at least 6 inches. Sills shall be fastened to pedestals with dowels of not less than 3/4-inch diameter, set in the pedestals and extending into the sills at least 6 inches.

Posts shall be fastened to sills by dowels of not less than 3/4-inch diameter, extending at least 6 inches into the posts and sills, or by drift bolts of not less than 3/4-inch diameter driven diagonally through the base of the posts and extending at least 9 inches into the sill. Posts shall be fastened to pedestals with dowels of not less than 3/4-inch diameter and extending into the posts at least 6 inches.

D. Caps

Timber caps shall be not less in size than 10 by 10 inches. They shall be fastened with drift bolts of not less than 3/4-inch diameter, extending at least 9 inches into the piles or posts.

E. Bracing

Single-story bracing shall not exceed 20 feet in height. The minimum size of transverse sway braces shall be 3 by 8 inches. All bracing shall be bolted through the piles, posts or caps at the ends; at intermediate intersections, it may be bolted or spiked. In all cases, spikes shall be provided in addition to bolts. The bolts used shall be not less than 5/8-inch diameter.

F. Pile Bent Abutments

Pile bent abutments shall be adequately braced or anchored to resist earth pressure. Bulkhead plank shall be not less than 3 inches thick. It shall be fastened to the piles with spikes, the length of which shall be at least 3 inches greater than the thickness of the plank.

1.10.9 – TRUSSES

A. Joints and Splices

Joints shall be detailed to shed water to the maximum degree practicable. Joints and splices shall be designed to develop the computed stresses in the members connected and, preferably, to develop the full strength of the members. Posts or struts bearing against the sides of timber members, preferably, shall be provided with metal end bearings. Joints involving end bearing on inclined surfaces shall be avoided, preferance being given to square-cut ends of timbers bearing against blocks.

Bearing surfaces of castings connecting timber members shall be milled to provide smooth, even surfaces permitting accurate fitting and complete contact of the wood and metal bearing surfaces. Rolled plates, bars and shapes used in chord splice plates, or other parts bearing upon wood surfaces shall be true and even. The wood surfaces taking bearing upon metal parts shall be not less than 5/8 inch in width. Bolts engaging castings and structural parts shall hold them rigidly in position so that bending on the parts in contact will be reduced to a minimum. The joint details at truss panel points shall provide definite lines of action and shall be simple and as susceptible as possible of definite strength analysis. When inclined bolts are used to connect end posts or web members with chord members, they shall be placed approximately at an angle of not more than 60 degrees with the latter and when used in conjunction with joint castings, the holes in one of the connected members shall be bored 1/8 inch larger than the nominal diameter of the bolts. No daps in chords for butt blocks shall be less than 3/4 inch deep.

Splices for tension members shall be designed to reduce to a minimum the effects of cross shrinkage of the timber. Neither steel splice plates of the batten type nor shear pin splices shall be used when the timbers to be spliced are more than 8 inches thick, since the shrinkage will permit the joint to become loose. Shear pin joints shall be used only with fully seasoned timber.

B. Floor Beams

Floor beams shall be sized at bearing points. In floor beams composed of two or more timbers, the timbers preferably shall be separated by at least 2 inches for air circulation. Floor beams shall be connected to the main truss members by means of rods or structural shapes.

C. Hangers

Hangers generally shall be rods having upset ends with a suitable designed washer or bearing plate at each end. Upset ends shall conform to the requirements specified for Structural Steel Design, Division I.

D. Eyebars and Counters

The requirements specified for Structural Steel Design, Division I, for counters, eyebars and eyebar packing shall apply to such members when used in timber trusses.

E. Bracing

Timber trusses shall be provided with a rigid system of laterals in the plane of the loaded chord. When the details will premit, this lateral bracing shall be securely fastened to all longitudinal stringers. Lateral bracing, preferably rigid, in the plane of the unloaded chord, and rigid portal and sway-bracing shall be provided in all trusses having sufficient headroom. Outrigger brackets connected to extensions of the floor beams shall be used for bracing through-trusses having headroom insufficient for a top lateral system.

F. Camber

Camber, in addition to that required to provide for dead load and shrinkage, shall be provided in timber trusses in sufficient amount to give the structure a good appearance.

1.10.10 – FLOORS AND RAILINGS

A. Stringers

Stringers shall be of sufficient length to take bearing over the full width of caps or floor beams, except outside stringers which may have butt joints. Preferably, they shall be of two panel lengths placed with staggered joints. The lapped ends of untreated stringers shall be separated at least 1/2 inch for air circulation. Stringers shall be secured to caps or floor beams.

B. Bridging

Stringers shall be braced by cross bridging in each panel. The bridging shall be not less in size than 2 by 4 inches.

C. Nailing Strips

When timber floors are supported by steel joists, the joists shall be provided with nailing strips which shall be bolted either to the top flanges or the webs.

When nailing strips are bolted to the flanges, they shall be used on all joists. They shall be not less than 4 inches deep and shall be wider than the supporting flange. They shall be secured with 5/8-inch bolts through the flanges, spaced not more than 4 feet apart and not more than 18 inches from the ends of the strips.

Nailing strips bolted to the webs shall be not less than 4 inches thick and shall be fastened with bolts spaced not farther apart than 5 feet. They shall be held clear of the flanges by blocks between the web and strip, and bolted through the web with 5/8-inch bolts spaced not more than 4 feet apart and not more than 18 inches from the ends of the strips.

D. Flooring

Roadway floor plank shall have a nominal thickness of not less than 3 inches. Sidewalk floor plank shall have a nominal thickness of not less than 2 inches.

The minimum size of material used for laminated or strip floors shall be 2 by 4 inches.

E. Retaining Pieces

Retaining pieces, where required, shall be not less than 6 inches in width. In general, they shall be secured in place by 5/8-inch bolts at 3-foot intervals and spiked at 1-foot intervals.

F. Wheel Guards

Wheel guards having a cross section of not less than 4 by 6 inches shall be provided on each side of the roadway. The guard timbers shall be in lengths of not less than 12 feet. They shall be secured with 5/8-inch bolts at the ends and at intermediate points not more than 4 feet apart.

In strip floors or cambered floors, not provided with retaining pieces, the wheel guards shall be placed directly on the flooring with scupper holes at suitable intervals. In other floors the wheel guards shall be supported by scupper blocks not less than 4 inches thick and 1 foot long, held in place by spikes and a bolt through the wheel guard and flooring, and spaced not more than 4 feet center to center.

G. Drainage

Adequate provision shall be made for the proper drainage of timber floors.

H. Railings

Wood railings shall consist of not less than 2 horizontal lines of rails. Rails, rail posts and fastenings shall be designed for the loads specified in Article 1.1.9.—Railings.

Preferably, rails shall be surfaced 4 sides (S4S) and painted.

1.10.11 – FIRE STOPS

To check the spread of the fire lengthwise of the structure, timber floors or trestles of any considerable length, preferably shall be provided with fire stops.

In timber floors these fire stops should be provided at intervals of not over 75 feet. They may consist of diaphragms of wood or fire-resistant material at least as thick as the flooring, located over caps or floorbeam and completely filling the opening between the joists.

In timber trestle bridges, in addition to the fire stops in the floor, fire curtains may consist of plank or asbestos-covered metal spiked to the bents. They should extend downward from the bottom of the joists at least 5 feet and horizontally at least to the ends of the caps. A fire stop between the joists should be located over each curtain.

Section 11 — LOAD CAPACITY RATING OF EXISTING BRIDGES

For requirements relative to load capacity rating of existing bridges see *Manual for Maintenance Inspection of Bridges 1974.*

Section 12 – ELASTOMERIC BEARINGS

1.12.1 – GENERAL*****

Elastomeric bearings shall be subject to the requirements of this section and to the sections applicable to the particular types of construction with which they are used.

The elastomers to be used shall conform to requirements given in the material specifications of the New York State Department of Transportation.

1.12.2 – DESIGN*****

Bearings may be plain (consisting of elastomer only) or laminated (consisting of layers of elastomer restrained at their interfaces by bonded laminates). Elastomer compounds of nominal 70 durometer hardness may be used in plain bearings only, which will generally be restricted by the requirements of this specification to conditions where little movement is anticipated.

The following terms will be used:

- L = Length of a rectangular bearing parallel to the direction of translation.
- W = Width of a rectangular bearing perpendicular to the direction of translation.
- D = Diameter of a circular bearing.
- B = Average thickness of a plain bearing or of any individual layer of elastomer in a laminated bearing (including the top and bottom layer.)
- T = Total effective elastomer thickness (summation of B.)
- S = Shape factor (the area of the loaded face divided by the side area free to bulge.)

$$S = \frac{LW}{2t\,(L+W)} \quad \text{for rectangular bearings}$$

$$S = \frac{R}{2t} \quad \text{for circular bearings}$$

Generally, the maximum value of S shall be 6.

The size of the elastomeric pad shall be such that both surfaces are in complete contact with the bearing areas.

The compressive strain of a plain bearing or of any individual layer of a laminated bearing is a function of the average unit compressive stress, the hardness of the elastomer and the shape factor. The compressive deflection of each laye; is the product of the strain and the thickness of the layer. The total deflection of the bearing is the sum of the layer deflections. The shear strain of a bearing is a function of the temperature, hardness of the elastomer and average shearing unit stress. The shear deflection of a bearing is the product of the shear strain and the total effective elastomer and average shearing unit stress. The shear deflection of a bearing is the product of the shear strain and the total effective elastomer thickness. These relationships may be taken from existing test reports but for large bearings or groups of standard designs they shall preferably be verified by tests of the particular designs involved.

Bearings shall have built in taper when non-parallel load surfaces would otherwise produce a compressive deflection of .06 T under dead load. Such taper shall be limited to 5/8 inches per foot.

To insure stability the following limits shall be observed:

Plain Bearings	– Minimum L	= 5T
	– Minimum W	= 5T
	– Minimum R	= 3T
Laminated Bearings	– Minimum L	= 3T
	– Minimum W	= 2T
	– Minimum R	= 2T

The total movement caused by release of camber due to dead load and a temperature rise of 80 F shall not exceed .25 T.

The average unit pressure on elastomeric bearings shall not exceed 800 psi under a combination of dead load plus live load, not including impact. The average unit pressure due to dead load only shall not exceed 500 psi. When dead load plus live load uplift reduce the average pressure to less than 200 psi the bearing shall be secured against horizontal crawling preferably by positive attachment to the top surface or to the top and bottom surfaces. When secured to the top and bottom surfaces the bearing may be subject to momentary light tension.

The initial compressive deflection in a plain bearing or in any layer of a laminated bearing, under dead load plus live load, not including impact shall not exceed .07 T. The deflection can be determined from a plot showing the relationship shape factor, load and durometer hardness of the elastomer under consideration. These curves are generally available from manufacturers for their product.

Section 14 — TFE BEARING SURFACE

1.14.1 — GENERAL

Proprietary makes of TFE fixed and expansion bearings may be used if in the opinion of the Engineer and substantiated either by tests or experience that they meet design requirements.

Bearings having sliding surfaces of polytetrafluoroethylene (TFE) shall be subject to the requirements of this Section and to Sections applicable to the particular types of construction with which they are used.

The TFE material consisting of filled or unfilled sheet, fabric containing TFE fibers, interlocked bronze and filled TFE structures and TFE-perforated metal composite as shown on the plans together with adhesive materials, stainless steel mating surface and manufacturing processes shall conform to the requirements given in Section 27 of the Specification.

1.14.2 — DESIGN

TFE sliding surfaces are designed to translate or rotate by sliding of a self-lubricating polytetrafluoroethylene surface across a smooth hard mating surface, preferably of stainless steel or other equally corrosion resistant material.

Expansion bearings have sliding surfaces of TFE shall not be used without provision for rotation which shall be not less than .015 radians, to prevent excessive local stresses on the TFE sliding surface. Rotation shall be considered the sum of live load rotation, changes in camber during construction and misalignment of the bearing seats due to construction tolerances. The design shall include compensating provision for grade. Provision for rotation may be accomplished with a hinge, radiused sliding surfaces, elastomeric pads, preformed fabric pads or other means acceptable to the Engineer. The design, materials and construction of the means of equalizing the stress or provision for rotation in the bearing shall be in accordance with the applicable section of the Specification.

TFE sliding surfaces shall have the following minimum and maximum thickness:

Unfilled or filled TFE	1/32″ minimum	3/32″ maximum
Fabric containing TFE fibers	1/80″ minimum	1/8″ maximum
Interlocked bronze and filled TFE structures	1/32″ minimum	1/8″ maximum
TFE-Perforated metal composite	1/16″ minimum	1/8″ maximum

The TFE sliding surface must be either bonded under factory controlled conditions or mechanically connected to a rigid back-up material capable of resisting any bending stresses to which the sliding surfaces may be subjected. Alternatively, TFE material of twice the thickness specified above may be recessed for half its thickness in the back-up material and shall not be less than 1/8″ thick. If the other side of the back-up material is to be bonded to an elastomeric pad, the back-up material must have sufficient tensile strength to restrain the elastomeric pad. The elastomeric pad must be sufficiently hard to allow sliding of the contact surfaces, preferably at least 70 durometer hardness.

The mating surface to the TFE should be an accurate flat, cylindrical or spherical surface as required by the design and shall have minimum Brinnel hardness of 125 and a surface finish of less than 20 micro inches root mean square (rms). The mating surface shall completely cover the TFE surface in all operating positions of the bearing. Wherever possible, the mating surface shall be oriented so that sliding movements will cause dirt and dust accumulation to fall from the mating surface.

The minimum coefficient of friction used for design shall be as specified by the bearing manufacturer or as follows:

	Bearing Pressure 500psi	2000psi	3500psi
Unfilled TFE, Fabric containing TFE fibers TFE-Perforated Metal Composite	.08	.06	.04
Filled TFE	.12	.10	.08
Interlocked Bronze and Filled TFE Structures	.10	.07	.05

The average bearing pressure on the TFE sliding surface due to all loads shall not exceed:

Unfilled and Filled TFE	2,000 psi
Fabric containing TFE fibers	3,500 psi
Interlocked Bronze and Filled TFE Structures	6,000 psi
TFE-Perforated Metal Composite	5,000 psi

The edge load pressure due to all loads and rotation shall not exceed:

Unfilled and Filled TFE	5,000 psi
Fabric containing TFE fibers	10,000 psi
Interlocked Bronze and Filled TFE Structures	10,000 psi
TFE-Perforated Metal Composite	5,000 psi

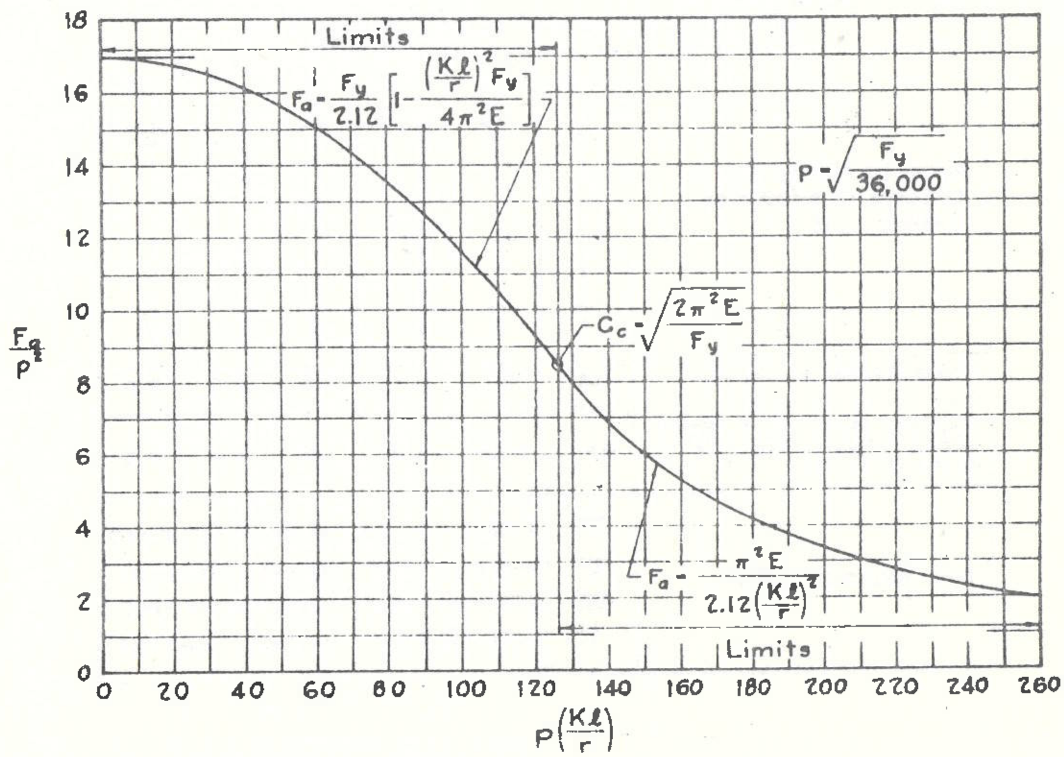
Holes or slots shall not be used in the sliding surfaces.

Welding to steel plate which has a bonded TFE surface may be permitted providing welding procedures are established which restrict the maximum temperature reached by the bond area to less than 300 F (150 C) as determined by temperature indicating wax pencils or other suitable means.

Means shall be provided in the design to positively locate all elements of the bearing. Where a thin non-corrosive smooth facing material is used as a mating sliding surface, it shall be structurally bonded by an approved adhesive system and may also be mechanically fastened by means of either screws or rivets to the back-up material, or if the materials permit, seal-welded around the entire perimeter of the facing material.

APPENDIX C

APPENDIX C
FORMULA FOR COMPRESSION IN CONCENTRICALLY LOADED COLUMNS
(See Table 1.7.1 for Specific Values)

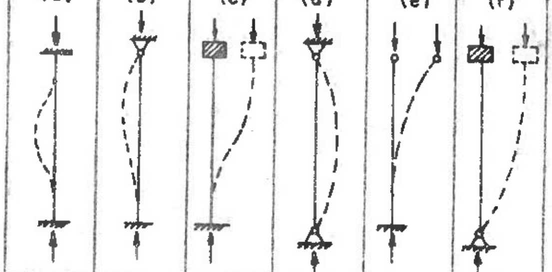


Appendix C (1)

EFFECTIVE LENGTH FACTOR, "K"

The Effective Length of a column, "KL" has been used in the equations for allowable compression stress in the column. "K" is the ratio of the effective length of an idealized pin-end column to the actual length of a column with various other end conditions. "KL" represents the length between inflection points of a buckled column. Restraint against rotation and translation of column ends influences the position of the inflection points in a column. Theoretical values of "K" for some idealized column end conditions are shown in Table C-1. Since column end conditions seldom comply fully with idealized restraint against rotation and translation, the recommended values suggested by the Column Research Council are higher than the idealized values.

TABLE C-1

EFFECTIVE LENGTH FACTORS, K						
	(a)	(b)	(c)	(d)	(e)	(f)
BUCKLED SHAPE OF COLUMN IS SHOWN BY DASHED LINE						
THEORETICAL K VALUE	0.5	0.7	1.0	1.0	2.0	2.0
DESIGN VALUE OF K WHEN IDEAL CONDITIONS ARE APPROXIMATED	0.65	0.80	1.2	1.0	2.1	2.0
END CONDITION CODE		ROTATION FIXED	TRANSLATION FIXED			
		ROTATION FREE	TRANSLATION FIXED			
		ROTATION FIXED	TRANSLATION FREE			
		ROTATION FREE	TRANSLATION FREE			

Columns in continuous frames unbraced by adequate attachment to shear walls, diagonal bracing, or adjacent structures depend upon the bending stiffness of the rigidly connected beams for lateral stability. The effective length factor, "K" is dependent upon the amount of bending stiffness supplied by the beams at the column ends. If the amount of stiffness supplied by the beams is small, then the value of "K" could exceed 2.0.

If it is assumed that elastic action occurs and that all columns buckle simultaneously in a frame, then it can be rationally shown that

$$\frac{G_a G_b \left(\frac{\pi}{K}\right)^2 - 36}{\sigma\,(G_a + G_b)} = \frac{\left(\frac{\pi}{K}\right)}{\tan\left(\frac{\pi}{K}\right)}$$

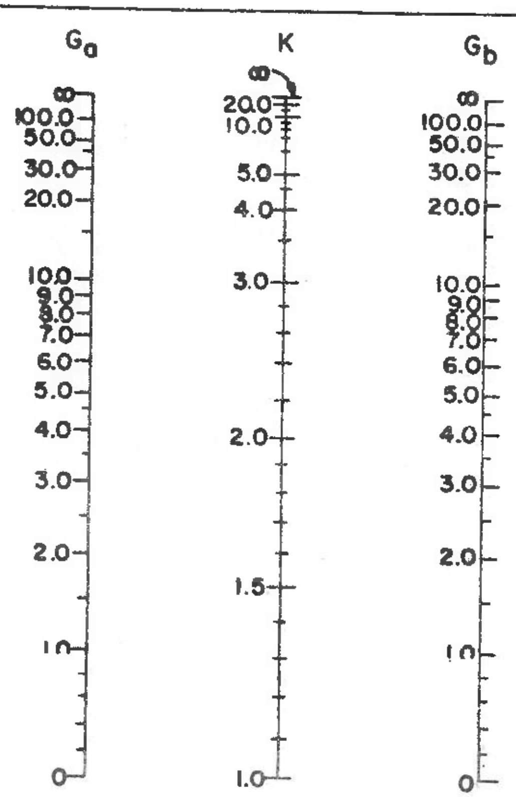
Where: subscripts a and b refer to the two ends of the column.

$$G = \frac{\Sigma\left(\frac{I_c}{L_c}\right)}{\Sigma\left(\frac{I_g}{L_g}\right)}$$

	=	Summation of all members rigidly connected to an end of the column in the plane of bending.
I_c	=	Moment of inertia of column.
L_c	=	Unbraced length of column.
I_g	=	Moment of inertia of beam or other restraining member.
L_g	=	Unsupported length of beam or other restraining member.
K	=	Effective length factor.

Table C-2 is a graphical representation of the relationship of C, g_a and G_b, and can be used to obtain the value of K easily. In frames which have

TABLE C-2



SIDESWAY PERMITTED

FOR COLUMN ENDS SUPPORTED BY BUT NOT RIGIDLY CONNECTED TO A FOOTING OR FOUNDATION, G IS THEORETICALLY EQUAL TO INFINITY, BUT UNLESS ACTUALLY DESIGNED AS A TRUE FRICTIONLESS PIN, MAY BE TAKEN EQUAL TO 10 FOR PRACTICAL DESIGN. IF THE COLUMN END IS RIGIDLY ATTACHED TO A PROPERLY DESIGNED FOOTING, G MAY BE TAKEN EQUAL TO 1.0. SMALLER VALVES MAY BE TAKEN IF JUSTIFIED BY ANALYSIS.

Appendix C(4)