TO: 233. Walcolm D. Graham Facilities Design Subdivision Bldg. 5, Room 404 **MODIFIED BY EI 72-045 EFFECTIVE 4/18/1972** **SUPERSEDED BY EI 76-034 EFFECTIVE 5/4/1976**	**ENGINEERING INSTRUCTION** NEW YORK STATE DEPARTMENT OF TRANSPORTATION SUBJECT: Revisions to Standard Specifications for Highway Bridges Subject Code: 7.35.4
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APPROVED: R. M. Kamp Deputy Chief Engineer (Structures)	

The attached pages are revisions to Standard Specifications for Highway Bridges.

Page 43	-	No change
Page 44	-	Revision to Article 1.4.4(C)(1)(D). Redesignated "steel piles."
Page 51	-	Revision to Title of Article 1.4.5(I) Deleted last two lines under Article 1.4.5(J) since they are now on page 51-1 in Article 1.4.5(K).
Page 51-1	-	New Article 1.4.5(J) New Article 1.4.5(K). (Was 1.4.5(J).)
Page 51-2	-	Blank page due to revisions.
Page 52	-	Deleted top three lines since they are now on page 51-1 in Article 1.4.5(K).
Pages 82-1 to 82-29	-	New Articles 1.5.14 to 1.5.28 incl.
Page 113	-	Deleted Article 1.7.6 in its entirety.
Page 114	-	No change
Page 162	-	No change
Pages 162-1 to 162-24	-	New Articles 1.7.118 to 1.7.139 incl.
Page 165	-	Revised Articles 1.8.2 by changing 6 and 8-Bolt connections from 184 to 180 and 220 to 194 respectively.
Page 166	-	No change
Page 171	-	Deleted Articles 1.9.1 to 1.9.8 incl. New Articles 1.9.1 to 1.9.6 incl.
Page 172	-	Blank Page due to revisions
Page 173	-	Blank page due to revisions

Attachments

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 E PROBABLE DEPTH OF SCOUR OR FLOTATION OF MATERIAL AND THE CONDI- ..ON 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.

* NOTE: THE LIMITATION IN (C) AND (D) GOVERN EXCEPT WHERE THE POINT BEARING CAPACITY OF THE PILES IS DETERMINED BY LOADING TEST PILES.

(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.

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 SCARS *****

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 PILES

(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 WARRENT, 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.

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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 UNDER-MINING. 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 BURRENTS 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 RE-STRAINED 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 THICK-

LOAD FACTOR DESIGN

1.5.14—GENERAL

(A) Application

These specifications are intended for use in the design of simple and continuous structures of moderate (to 200') span length. Large or unusual structures may require special study and detailed consideration of effects that can otherwise be neglected or assigned arbitrary values in the design of structures to which these specifications are intended to apply.

(b) Other Specifications

All applicable provisions of the AASHO Specifications shall apply unless specifically modified herein.

1.5.15—NOTATION

(A) Loads and Forces

B = Buoyancy
CF = Centrifugal force
D = Dead load
EQ = Earthquake
F = Longitudinal force
I = Live load impact
ICE = Ice pressure
L = Live load
LF = Longitudinal force from live load
M = Moment to be used for design of compression member
M_b = Column moment capacity under balanced conditions
M_{cr} = Moment required to crack a concrete section
M_{max} = Maximum dead load moment for section under consideration
M_u = Moment capacity of the section $\geqslant$ applied design load moment at a section
M_{uo} = Theoretical moment strength of a section
M_{ux} = Moment capacity in the direction of the x axis
M_{uy} = Moment capacity in the direction of the y axis
M_x = Design bending moment component in the direction of the x axis
M_y = Design bending moment component in the direction of the y axis
M_I = Value of smaller end moment on compression member calculated from a conventional elastic frame analysis, positive if member is bent in single curvature, negative if bent in double curvature

(B) Dimensions and Constants

A_b = Loaded area
A_b' = Maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area
A_g = Gross area of column section
A_s = Area of tension reinforcement
A_s' = Area of compression reinforcement
A_{sf} = Area of reinforcement to develop compressive strength of overhanging flanges in I- and T-sections
A_{st} = Total area of longitudinal reinforcement = $A_s + A_s'$ = total vertical reinforcement in columns

A_v = Area of shear reinforcement within a distance s
a = Depth of equivalent rectangular stress block = k_1c
a_s = Area of an individual bar, sq. in.
b = Width of compression face of flexural member, or member subject to flexure
b' = Width of web in I- and T-sections. In tapered webs, the average width or 1.2 times the minimum width, whichever is smaller
b_o = Periphery of critical section for slabs and footings
c = Distance from extreme compression fiber to neutral axis
c_b = Distance from extreme compression fiber to neutral axis for balanced conditions
C_m = a factor relating the actual moment diagram to an equivalent uniform moment diagram
D = Nominal diameter of bars; also, overall diameter of circular section
d = Distance from extreme compression fiber to centroid of tension reinforcement
d' = Distance from extreme compression fiber to centroid of compression reinforcement
E_c = Modulus of elasticity of concrete
E_s = Modulus of elasticity of steel
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
F = Moment magnification factor
f'_c = Specified compression strength of concrete
f_h = Tensile stress developed by a standard hook, psi
f_r = Modulus of rupture of concrete
f_y = Specified yield strength of reinforcement
h = Unsupported length of compression member
I_{cr} = Moment of inertia of the transformed cracked section
I_{eff} = Effective moment of inertia for computation of deflection
I_g = Moment of inertia of gross concrete section about the centroidal axis, neglecting the reinforcement
K = Constant for standard hook
k = Effective length factor in design of slender columns
k_1 = 0.85 for strengths, f'_c, up to 4000 psi, and shall be reduced at a rate of 0.05 for each 1000 psi of strength in excess of 4000 psi
L_a = Additional embedment length at support or at point of inflection, in.
L_d = Development length, in.
L_e = Equivalent embedment length, in.
n = E_s/E_c
p = A_s/bd
p' = A_s'/bd
p_b = Reinforcement ratio producing balanced conditions
p_f = $A_{sf}/b'd$
p_w = $A_s/b'd$
q = $A_sf_y/bdf'_c = pf_y/f'_c$
R_m = Ratio of maximum design dead load moment to maximum design total load moment, always positive
r = Radius of gyration of the concrete gross section in the direction of bending

r_b = Ratio of area of bars cut off to total area of bars at the section
s = Shear reinforcement spacing in a direction parallel to the longitudinal reinforcement
t = Flange thickness in I- and T-sections; also overall depth of section
t_{min} = Recommended minimum thickness for constant depth members
y_t = Distance from centroidal axis of gross section, neglecting the reinforcement, to extreme fiber in tension
ϵ_u = Maximum usable strain at the extreme concrete compression fiber, assumed equal to 0.003
ϵ_y = Yield strain of reinforcement corresponding to the yield strength, f_y
φ = Capacity modification factor

1.5.16–MATERIALS PROPERTIES

(A) Concrete

(1) The design strength, f'_c of the concrete shall be specified and the specified strength shall be indicated on the plans. The specified strength of the concrete shall be a basis for acceptance, and each class of concrete shall be represented by a sufficient number of tests.[1] For structures designed in accordance with these specifications, the average of any three consecutive strength tests of the laboratory-cured specimens representing each class of concrete (at least two specimens shall be made for each test) shall be equal to or greater than the specified strength, f'_c, and not more than 10 percent of the strength tests shall have values less than the specified design strength, but no test shall show an average strength less than 85 percent of the specified compressive strength f'_c.

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

(B) Reinforcement

(1) Reinforcing bars shall conform to one of the following specifications, except that yield strength shall correspond to that determined by tests on full sized bars and that reinforcing bars with a specified yield strength, f_y, exceeding 60,000 psi are not permitted under these specifications.

(a) "Specifications for Deformed Billet-Steel Bars for Concrete Reinforcement" (AASHO M31, ASTM A615). If #14 or #18 bars meeting these specifications are to be bent, they shall also be capable of being bent, 90 deg. at a minimum temperature of 60 F. around a ten-bar-diameter pin without cracking transverse to the axis of the bar.

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[1] In the Construction Specifications, or Special Provisions, there shall be included the requirements for the minimum number of tests and specimens for each test at a given age of the concrete, the maximum number of cubic yards of structural concrete for each test, and the least number of tests for each day's concreting. The age for strength tests shall be 28 days, or where specified, the earlier age at which the concrete is to receive its full load or maximum stress. Strength Control Procedures shall be preferably in accordance with ACI 214-65, "Recommended Practice for Evaluation of Compression Tests Results of Field Concrete." Refer also the following specifications: "Method of Sampling Fresh Concrete" (AASHO T141, ASTM C172); "Method of Making and Curing Concrete Compression and Flexure Test Specimens in the Field" (AASHO T23, ASTM C31); "Method of Test for Compressive Strength of Molded Concrete Cylinders" (AASHO T22, ASTM C39).

(b) "Specifications for Rail-Steel Deformed Bars for Concrete Reinforcement" (AASHO M42, ASTM A616). If bars meeting these specifications are to be bent, they shall also meet the bending requirements of AASHO M31, ASTM A615 for Grade 60.

(c) "Specifications for Axle-Steel Deformed Bars for Concrete Reinforcements" (AASHO M53, ASTM A617).

(2) The modulus of elasticity of steel reinforcement, E_s, may be taken as 29,000,000 psi.

1.5.17–LOADS AND LOAD FACTOR EQUATIONS

(A) Loads

The forces in the structure shall be determined by considering the elastic behavior of the structure under loads specified in Section 2, Loads, AASHO Specifications, Articles 1.2.4 and 1.2.16 excepted.

(B) Load Factor Equations

The following Load Groups represent various combinations of loads and forces to which a structure may be subjected. Each part of such structure, or the foundation on which it rests, shall be proportioned for all combinations of such of these forces as are applicable to the particular site or type.

The maximum section required shall be used.

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

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

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

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

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

$$\text{Group III} = 1.30\left[D + (L + I) + CF + 0.3W + WL + F + LF\right]$$

1.5.18–STRENGTH PROVISIONS

(A) Assumptions

(1) The strength design of members of flexure and axial loads shall be based on the assumptions given in this section, and on satisfaction of the applicable conditions of equilibrium and compatibility of strains.

(2) Strain in the reinforcing steel and concrete shall be assumed directly proportional to the distance from the neutral axis.

(3) The maximum usable strain at the extreme concrete compression fiber shall be assumed equal to 0.003.

(4) 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.

(5) Tensile strength of the concrete shall be neglected in flexural calculations of reinforced concrete.

(6) The relationship between the concrete compressive stress distribution and the concrete strain may be assumed to be a rectangle, trapezoid, parabola, or any other shape which results in prediction of strength in substantial agreement with the results of comprehensive tests.

(7) The requirements of Article 1.5.18(A)(6) may be considered satisfied by an equivalent rectangular concrete stress distribution which is defined as follows: A concrete stress of 0.85 f'_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 $a = k_1 c$ 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 k_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.

(8) 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.

1.5.19—CAPACITY MODIFICATION FACTORS

(A) The usable load capacities of the members shall be the calculated capacities of the members modified according to the provisions of this article.

(B) The computed theoretical capacity shall be modified by a capacity modification factor φ as follows:

For flexure . . . . $\varphi = 0.90$
For shear . . . . $\varphi = 0.85$
For spirally reinforced compression members . . . . $\varphi = 0.75$
For tied compression members . . . . $\varphi = 0.70$
For bearing on concrete . . . . $\varphi = 0.70$
Development lengths specified in Article 1.5.29 do not require a φ factor.

1.5.20—FLEXURE

(A) Rectangular sections with tension reinforcement only

For rectangular or flanged sections in which the neutral axis lies within the flange, the moment capacity shall be assumed as:

$$M_u = \varphi [A_s f_y d\ (1 - 0.6q)] \qquad (4\text{-}1)$$

$$= \varphi \left[A_s f_y \left(d - \frac{a}{2}\right)\right] \qquad (4\text{-}2)$$

where

$$q = p \frac{f_y}{f'_c} \qquad (4\text{-}3)$$

and

$$a = \frac{A_s f_y}{0.85 f'_c b} \qquad (4\text{-}4)$$

The reinforcement ratio, p, shall not exceed 0.50 of the ratio, p_b, which produces balanced conditions at ultimate stage given by:

$$p_b = \frac{0.85\, k_1 f'_c}{f_y} \times \frac{87{,}000}{87{,}000 + f_y} \qquad (4\text{-}5)$$

(B) I- and T-sections

(1) When the flange thickness equals or exceeds the depth to the neutral axis, a/k_1, the section may be designed by Equation (4-1), with, a, computed as for a rectangular beam with a width equal to the overall flange width given by AASHO Article 1.5.5(A).

(2) When the flange thickness is less than a/k_1 the design moment M shall not exceed that given by the moment capacity of the section assumed as

$$M_u = \varphi\,[(A_s - A_{sf})\, f_y\, (d - \frac{a}{2}) + A_{sf}\, f_y\, (d - 0.5t)] \qquad (4\text{-}6)$$

where

$$A_{sf} = 0.85\,(b - b')\,\frac{t f'_c}{f_y} \qquad (4\text{-}7)$$

and

$$a = \frac{(A_s - A_{sf})\, f_y}{0.85\, f'_c b'} \qquad (4\text{-}8)$$

The reinforcement ratio, p_w shall not exceed 0.50 of the quantity $(p_b + p_f)$, where p_b is given by equation (4-5).

(C) Rectangular sections with compression reinforcement

The moment capacity of rectangular sections, or flanged sections in which the neutral axis lies within the flange, with compression reinforcement shall be assumed as:

$$M_u = \varphi\,[(A_s - A_s')\, f_y\, (d - \frac{a}{2}) + A_s' f_y\, (d - d')] \qquad (4\text{-}9)$$

where

$$a = \frac{(A_s - A_s')\, f_y}{0.85\, f'_c b} \qquad (4\text{-}10)$$

and the following condition shall exist

$$\frac{(A_s - A_s')}{bd} \geqslant 0.85\, k_1\, \frac{f'_c d'}{f_y d}\, \frac{87{,}000}{87{,}000 - f_y} \qquad (4\text{-}11)$$

When the value of $\frac{(A_s - A_s')}{bd}$

is less than the value given by Equation (4-11), so that the compression steel stress is less than the yield strength, f_y, or when effects of compression steel are neglected, the calculated moment capacity shall not exceed that given by Equations (4-1) and (4-2), except when a general analysis is made on the basis of the assumptions given in Article 1.5.18(A). The quantity

$$\frac{(A_s - A_s')}{bd} = (p - p')$$

shall not exceed 0.50 of the value of p_b, given by Equation (4-5).

(D) Other cross sections

(1) For other cross sections and for cases of nonsymmetrical bending, the moment capacity, $M_u = \varphi M_{uo}$, shall be computed by a general analysis based on the assumptions given in Article 1.5.18(A).

(2) The amount of tension reinforcement shall be so limited that the steel ratio, p, does not exceed 50 percent of that corresponding to balanced conditions as defined by Article 1.5.20(A).

(3) The moment capacity of the reinforced section, when cracked, shall be at least 1.5 times the moment which produces cracking of the transformed, uncracked section. This requirement, which limits the minimum tension reinforcement to be provided in the section shall apply to all sections of Article 1.5.20. The modulus of rupture of the concrete shall be used for calculating the resisting moment of the uncracked section.

1.5.21—SHEAR

(A) Shear stress

(1) The nominal design shear stress in reinforced concrete members shall be computed by:

$$v_u = \frac{V_u}{bd} \tag{4-12}$$

For design, the maximum design shear V_u shall be considered as that at the section a distance, d, from the face of the support. Wherever applicable, effects of torsion shall be added and effects of inclined flexural compression in variable depth members shall be included.

(2) For beams of I- and T-sections, b' shall be substituted for b in Equation (4-12).

(3) The shear stress capacity of the concrete, v_{uc}, shall not exceed $2\varphi\sqrt{f'_c}$ at a distance, d, from the face of the support.[3] If the reinforcement ratio p is less than 1.2 percent then the shear stress capacity of the concrete shall be governed by

$$v_{uc} = (0.8 + 100\,p)\ \varphi\sqrt{f'_c}\ .$$

[3] More detailed calculation of the allowable shear stresses should be made for members subject to axial tension or compression. For these conditions and for members of lightweight concrete, refer to the ACI Building Code for concrete shear capacity formulas.

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 Article 1.5.21(A).

(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 as specified in the remainder of this section.

(2) The periphery shear stress shall be computed by

$$v_u = \frac{V_u}{b_o d} \qquad (4\text{-}14)$$

in which V_u and b_o are taken at the critical section specified in Article 1.5.21(E)(1)(b). The periphery shear stress, v_u, shall not exceed the shear stress capacity of the concrete $v_{uc} = 4\varphi\sqrt{f'_c}$ unless shear reinforcement is provided in accordance with Article 1.5.21(E)(3), in which case, v_u, shall not exceed $6\varphi\sqrt{f'_c}$.

(3) When v_u, exceeds $v_{uc} = 4\varphi\sqrt{f'_c}$ shear reinforcement shall be provided in accordance with Articles 1.5.21(B) to 1.5.21(D), except that the design yield strength, f_y, for the shear reinforcement shall be 50 percent of that prescribed in Article 1.5.16(B). Shear reinforcement consisting of bars, rods or wires shall not be considered effective in members having an effective depth of less than 10 inches.

1.5.22—COLUMNS

(A) General

(1) All columns shall be designed to resist the combined bending and axial loads that result from the various combinations of loads and forces given in Article 1.5.17. All members subjected to a compression load shall be designed for the eccentricity, e, corresponding to the maximum moment that can accompany this loading condition, but not less than 1 inch, or 0.05t for spirally reinforced compression members, or 0.10t for tied compression members, about either principal axis.

(2) The area of longitudinal reinforcement preferably shall not be less than 1 percent, nor more than 8 percent of the gross concrete area of the column section. In a column which, for any reason, has a larger cross-section than required by the loads and moments determined in accordance with the provisions of Article 1.5.17, the minimum amount of longitudinal steel specified above may be reduced provided that in no case shall less longitudinal steel be used than that required by the minimum sized column necessary to support the loads and moments defined above designed with one percent of longitudinal steel.

(B) Column Section Capacities

(1) Concentric Loading

The axial load capacity of a column section subjected to pure compression, P_o, is:

(C) Slenderness effects in columns[4]

The influence of slender columns on behavior of the structure may be taken into account by the following approximate procedures.

(1) The unsupported length, h, 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 capitals or haunches are present, the unsupported length shall be measured to the lower extremity of the capital or 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 kh/r is less than $34 - 12\ M_1/M_2$. For compression members not braced against sidesway, the effects of slenderness may be neglected when kh/r is less than 22. For all compression members with kh/r greater than 100, a more exact analysis than that prescribed herein shall be made. M_1 = value of smaller end moment on compression member calculated from a conventional elastic frame analysis, positive if member is bent in single curvature, negative if bent in double curvature. M_2 = value of larger end moment on compression member calculated from a conventional elastic frame analysis, always positive.

(5) Compression members shall be designed using the design axial load from a conventional frame analysis and a magnified moment M defined by

$$M = FM_2 \qquad (4\text{-}19)$$

where

$$F = \frac{C_m}{1 - P_u/\varphi P_c} \geqslant 1.0 \qquad (4\text{-}20)$$

and

$$P_c = \frac{\pi^2 EI}{(kh)^2} \qquad (4\text{-}21)$$

[4] See "Design of Slender Concrete Columns" by James G. MacGregor, John E. Breen, and Edward O. Pfrang, Journal of the American Concrete Institute, January 1970, pp. 6-28 for a comprehensive discussion of these provisions for designing slender columns. This article includes nomographs for determining the effective length factor, k.

(B) When the supporting surface is wider than the loaded area on all sides, the permissible bearing stress on the loaded area may be multiplied by $\sqrt{A'_b/A_b}$, but not more than 2.

(C) When the supporting surface is sloped or stepped, A'_b 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 or more horizontal.

1.5.24—SERVICE LOAD REQUIREMENTS

(A) Service Load Stresses

(1) 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 for concrete is a straight line under service loads. Service load stresses vary as the distance from the neutral axis except for deep beams.
(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).

(2) In doubly reinforced beams and slabs, an effective modular ratio of $2E_s/E_c$ shall be used to transform the compression reinforcement for stress computations.

1.5.25—FATIGUE

(A) Concrete

The range of compressive stress in the concrete caused by a single passage of live load plus impact and centrifugal force, at service load level, shall be limited to $0.5f'_c$ at points of contraflexure,[5] and at sections where stress reversals occur.

(B) Reinforcement

The range of stress in straight reinforcement caused by a single passage of live load plus impact at service load level, shall be limited to 21,000 psi.[6] Bends in primary reinforcement shall be avoided at sections having a high range of stress.

1.5.26—FLEXURAL STRESS LIMITATIONS

(A) General

(1) The steel stress range shall comply with the fatigue provisions of Article 1.5.25(B).[6]

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[5]Concrete Roadway Slabs excluded.

[6]Applicable primarily to bridge deck slabs and short span slab bridges where the dead load to total load moment ratio is less than approximately 0.25.

(B) Dead load deflections at falswork removal

Unless a more comprehensive analysis is made, immediate dead load deflections upon removal of falsework shall be computed by the usual methods of formulas for elastic deflections, using the modulus of elasticity for concrete specified in Article 1.5.16(A). The effective moment of inertia shall be taken as the following, but not greater than I_g:

$$I_{eff} = (M_{cr}/M_{max})^3 I_g + [1 - (M_{cr}/M_{max})^3] I_{cr} \qquad (5\text{-}1)$$

where

$$M_{cr} = f_r I_g / y_t$$

and

M_{max} = Maximum dead load moment for section under consideration

$$f_r = 7.5\sqrt{f'_c}$$

For continuous spans, the effective moment of inertia may be taken as the average of the values obtained from Equation (5-1) for the critical positive and negative moment sections.

(C) Long-time deflections caused by dead loads, creep and shrinkage

Unless a more comprehensive analysis is made, for purposes of determining falsework camber, the dead load deflection computed in (B) may be multiplied by a factor chosen from Table 1.5.27A. The long-time deflections, thus calculated, might be expected to occur over a period of about three years.

TABLE 1.5.27A

	$A'_s = 0$	$A'_s = ½A_s$	$A'_s = A_s$
Climate of high humidity	2.5	1.8	1.5
Climate of average humidity	3.0	2.2	1.8
Climate of low humidity	3.5	2.5	2.0

1.5.28–OVERLOAD

Structures proportioned by this specification will sustain without damage the following overload:

Members designed for Group I loading = D + 5/3 (L + I)
Members designed for Group IA loading = D + 2.2 (L + I)

1.5.29–DEVELOPMENT OF REINFORCEMENT

(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.

L_a at a support shall be the sum of the embedment length beyond the center of the support and the equivalent 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 or 12D, whichever is greater. The value M_{uo}/V_u 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 1.5.29(A)(1) and 1.5.29(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, 12D, 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.

(E) Development length of deformed bars in tension.

The development length, L_d in inches, of deformed bars in tension shall be computed as the product of the basic development length of (1) and the applicable modification factor or factors of (2) and (3), but L_d shall be not less than 12 in.

(1) The basic development length shall be:

For # 11 or smaller bars[10] . . . . $0.04a_sf_y/\sqrt{f'_c}$
but not less than[11] . . . . $0.0004Df_y$
For # 14 bars[12] . . . . $0.085f_y/\sqrt{f'_c}$
For # 18 bars[12] . . . . $0.11f_y/\sqrt{f'_c}$
For deformed wire . . . . $0.03Df_y/\sqrt{f'_c}$

(2) For top reinforcement[13] the basic development length shall be multiplied by a factor of 1.4.

(3) The basic development length, modified by the appropriate requirement of (2) may be multiplied by the applicable factor or factors for:

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

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[10]The constant carries the unit of l/in.

[11]The constant carries the unit of in^2/lb.

[12]The constant carries the unit of in.

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

Reinforcement in a flexural member
in excess of that required . . . . (A_s required/A_s provided)

Bars enclosed within a spiral which is not less than 1/4 in. diameter and not more than 4 in. pitch 0.75

(F) Development length of deformed bars in compression

(1) The development length L_d for bars in compression shall be computed as $0.02 f_y D / \sqrt{f'_c}$ but shall not be less than $0.0003 f_y D$ 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 1/4 in. in diameter and not more than 4 in. pitch.

(G) Development length of bundled bars

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

(H) Standard hooks in tension

(1) Standard hooks shall be considered to develop a tensile stress in bar reinforcement $f_h = K \sqrt{f'_c}$ where K is not greater than the values in Table 1.5.29.

TABLE 1.5.29

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

(2) An equivalent embedment length L_e shall be computed using the provisions of Article 1.5.29(E)(1) by substituting f_h for f_y and L_e for L_d.

(3) Hooks shall not be considered effective in adding to the compressive resistance of reinforcement.

(I) 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.

(J) Mechanical anchorage

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

(K) Anchorage of shear reinforcement

(1) Shear reinforcement shall be carried as close to the compression and tension surfaces of the member as cover requirements and the proximity of other steel will permit, and in any case the end of single leg, simple U-, or multiple U-stirrup, shall be anchored by one of the following means:

(a) A standard hook plus an effective embedment of 0.5 L_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).

(b) Embedment above or below the middepth, d/2, of the beam on the compression side for a full development length L_d but not less than 24 bar diameters.

(c) Bending around the longitudinal reinforcement through at 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 deformed longitudinal bars.

(d) Between the anchored ends, each bend in the continuous portion of a transverse simple U- or multiple U-stirrup shall enclose a longitudinal bar.

(e) Pairs of U-stirrups or ties so placed as to form a closed unit shall be considered properly spliced when the laps are $1.7L_d$. In members at least 18 in. deep, such splices having $a_s f_y$ not more than 9000 lb per leg may be considered adequate if the legs extend the full available depth of the member.

COMMENTARY

Article 1.5.16(B)—Reinforcement

The proposed change in reinforcement specifications reflects the provisions of the 1971 ACI Code. These provisions are considerably different from those in Section 2.A.3(c) of the BPR Criteria, or of the 1963 ACI Building Code. In a similar manner to the 1971 ACI Building Code, no reduction in stress value is stipulated in the proposed specifications for use in the absence of proof testing.

The reason for the Grade 60 exemption from a proof test beyond that required by AASHO Specifications was based on a review of an extensive series of stress-strain tensile tests on the full-range of bar sizes in Grade 60, sampled from all types of producing mills in all areas of the country. The tests were concluded in 1969 and were under the sponsorship of the American Iron and Steel Institute. Although average strengths of 0.003, 0.0035 and 0.005 strains were well above the minimum specified yield strength, f_y, normal scatter permitted study of some results in which the bars just met f_y at the AASHO prescribed strain of 0.005. Stresses at 0.003 or 0.0035 were generally closer to f_y than the underweight-understrength tolerances permitted at 0.005 strain under the outdated ASTM specifications used in the 1963 ACI Code, but not allowed in current specifications for reinforcing bars. The 1970 AASHO specifications changed the basis of computing yield strength from actual area (permitting bars to be underweight 3-1/2 to 6 percent for individual bars, to nominal area effectively upgrading required strengths 3-1/2 to 6 percent. It was concluded that no exception to the AASHO Specifications is required for bars with yield strength, f_y of 60,000 psi or less. The results of these tests, along with a description of testing procedures as conducted by an independent testing laboratory, are available for review by the AASHO Bridge Committee.

Article 1.5.18—Strength Provisions

The strength requirements proposed for reinforced concrete bridges are shown in Figure 1 for Group I loading on the basis of 60,000 psi yield point reinforcement. Superimposed on this drawing are the effects of three serviceability requirements that will be discussed later. (1. Fatigue stress range of 21,000 psi; 2. 36,000 psi maximum steel stress; and 3. a 10 percent increase in negative moment reinforcement for decks not provided with a waterproof membrane protective system.)

The Group I load factor equation results in the following total stress ranges for 40,000 and 50,000 psi reinforcement:

Yield Point	D/T = 0	D/T = 1.0
40,000 psi	16,600 psi	27,700 psi
50,000 psi	20,700 psi	34,600 psi

It is apparent that structures with low D/T ratios using 40,000 psi yield point reinforcement would have lower stresses under the proposed load factor specifications than those presently permitted by AASHO. However, some economy would be gained with 40,000 psi yield point reinforcement for higher D/T ratios, and the proposed specifications may be used with either 40,000 or 50,000 psi yield point reinforcement if desired.

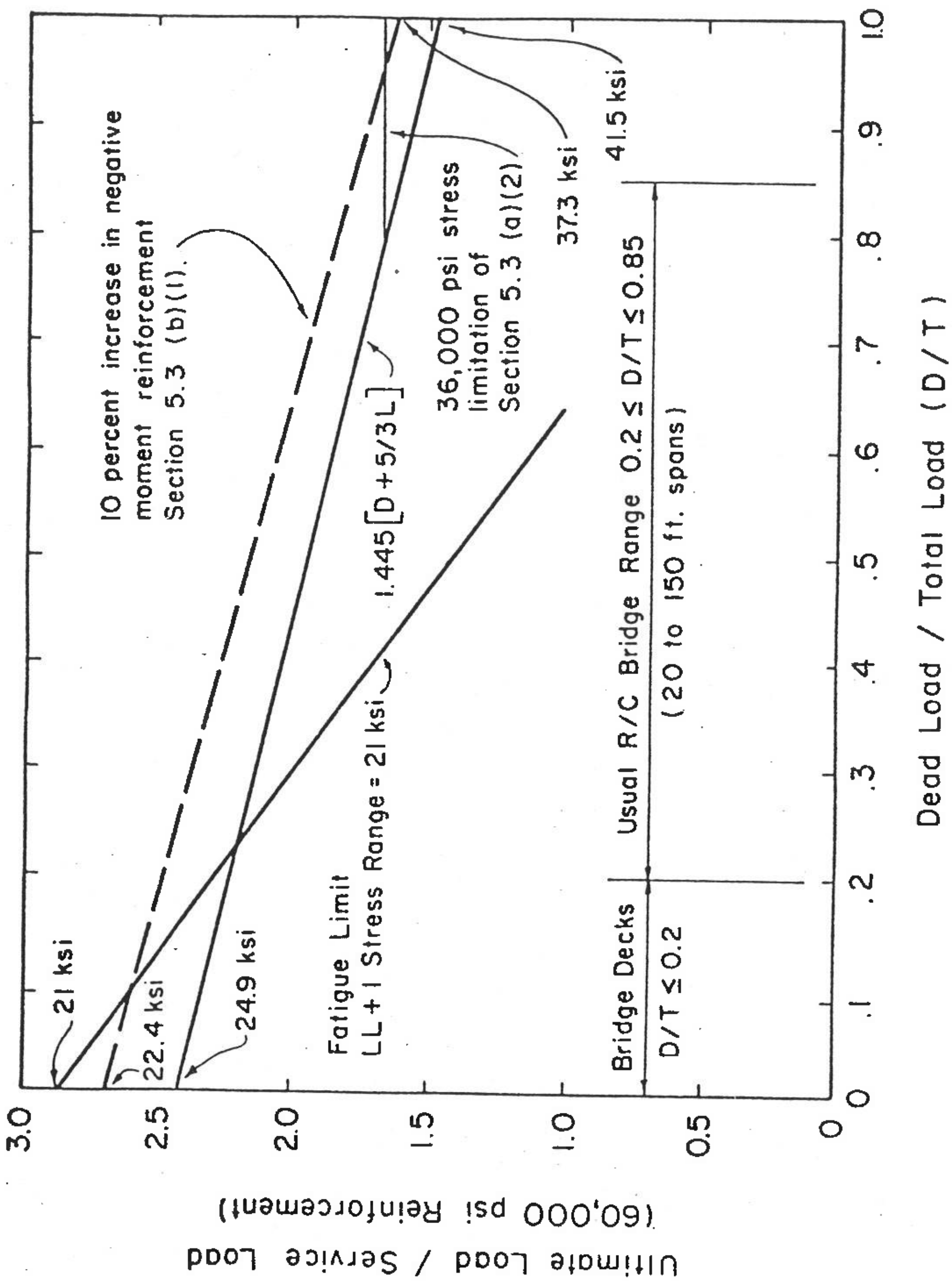
In regard to strength provisions, it should be noted that the moment redistribution provisions in Sections 3.A.3 of the BPR Criteria have been eliminated in the proposed specifications. Application of moment redistribution to reduce negative moment reinforcement was considered inappropriate under the proposed specifications from the standpoint of both strength and serviceability.

Article 1.5.22—Columns

Column designs by the ultimate strength-load factor method generally will result in smaller column sections, or less reinforcement, or both, than designs by the AASHO procedure. Figure 2 shows interaction diagrams for one comparison—a 5′ diameter column with 1 percent of 40 ksi yield strength reinforcement. The straight line representing allowable column loads as calculated by the AASHO procedure (labeled AASHO Design 1) clearly is inadequate for the various AASHO Group Loads plotted in the Figure. In fact, a 6′ diameter column with 1 percent of 40 ksi yield strength reinforcement is necessary (labeled AASHO Design 2). However, the curved line representing Load Factor Design 1 shows that the 5′ diameter column with 1 percent of 40 ksi yield strength reinforcement satisfies all factored loading groups.

A few specific values comparing the Load Factor and the AASHO designs for the 5′ diameter column are summarized in the following table:

Eccentricity Ratio	Section Capacity Ratio $\frac{\text{Section capacity by load factor method}}{\text{Allowable load by AASHO procedures}}$	Load Ratio $\frac{\text{Factored Load}}{\text{AASHO Load}}$
e/t = 0.1	3.2	1.42 (Group I)
e/t = 0.23	2.9	1.63 (Group III)
e/t = 0.3	2.4	1.73 (Group II EQ & VII)



3.0
2.5
2.0
1.5
1.0
0.5
0
0
.1
.2
.3
.4
.5
.6
.7
.8
.9
1.0
21 ksi
22.4 ksi
24.9 ksi
10 percent increase in negative moment reinforcement Section 5.3 (b)(1).
1.445[D+5/3L]
36,000 psi stress limitation of Section 5.3 (a)(2)
37.3 ksi
41.5 ksi
Fatigue Limit LL+I Stress Range = 21 ksi
Bridge Decks D/T ≤ 0.2
Usual R/C Bridge Range 0.2 ≤ D/T ≤ 0.85 (20 to 150 ft. spans)
Dead Load / Total Load (D/T)
Ultimate Load / Service Load (60,000 psi Reinforcement)

FIG. 1

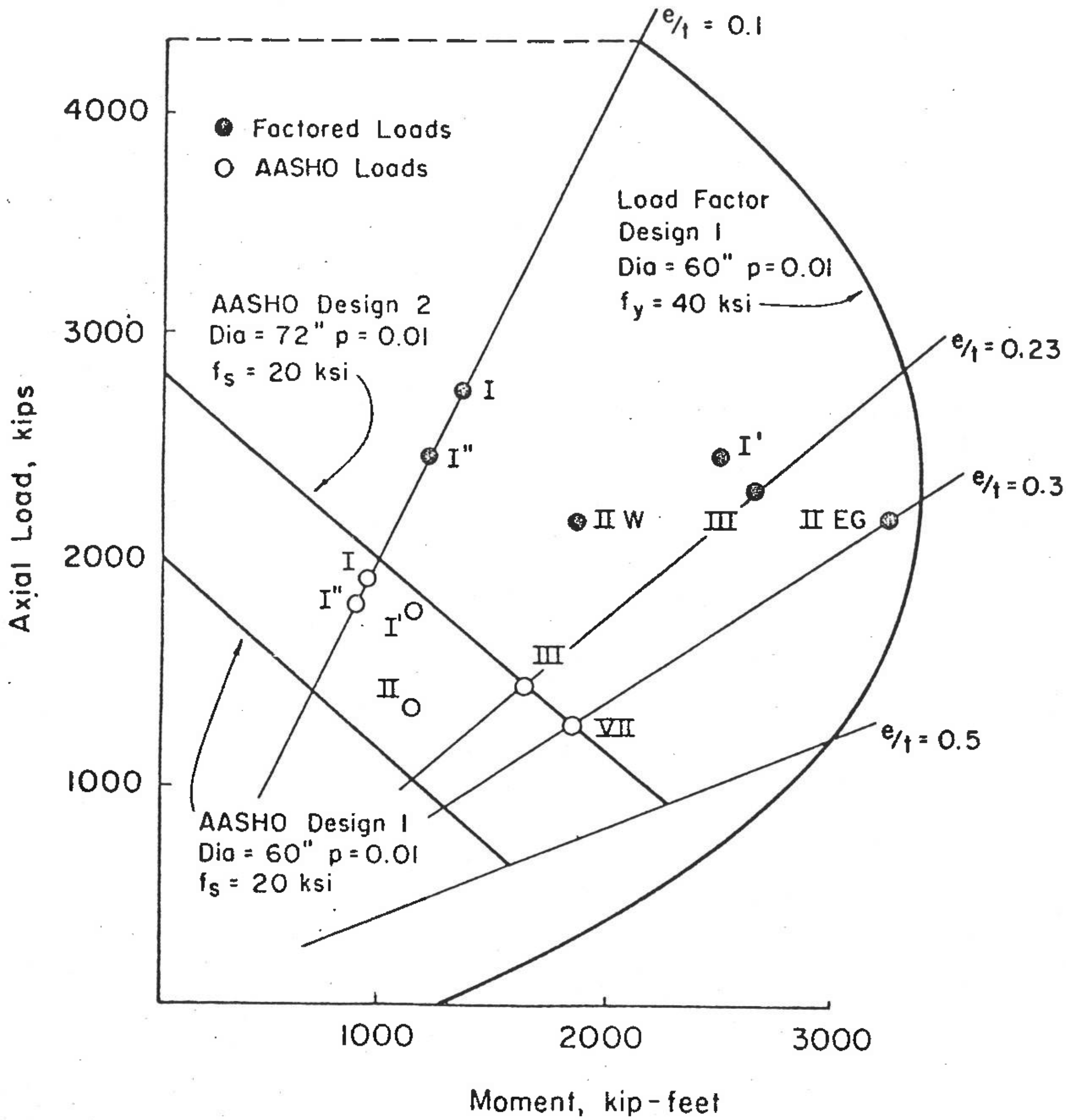

e/t = 0.1
4000
3000
2000
1000
Factored Loads
AASHO Loads
Load Factor
Design I
Dia = 60" p = 0.01
fy = 40 ksi
AASHO Design 2
Dia = 72" p = 0.01
fs = 20 ksi
e/t = 0.23
e/t = 0.3
e/t = 0.5
I
I"
I'
II W
III
II EG
I
I"
I'
II
III
VII
AASHO Design I
Dia = 60" p = 0.01
fs = 20 ksi
Axial Load, kips
1000
2000
3000
Moment, kip-feet

FIG. 2

Attention is invited to two features indicated by the figure and the table. First, the load ratio (i.e., ratio of factored load to AASHO load) varies from 1.42 to 1.73. This ratio for the Group I loads is primarily influenced by the overall load factor 1.30 since columns characteristically have high D/T ratios. The load ratio for the Group III loads is merely the result of multiplying the overall load factor for Group III by the allowable percent overstress for AASHO Group III, i.e., 1.30x1.25 = 1.63. The third load ratio compares Load Factor Group II EQ with AASHO Group VII. This comparison also results from multiplying the overall load factor for Group II by the allowable percent overstress for AASHO Group VII, i.e., 1.30x1.33 = 1.73.

Second, the figure illustrates that column designs by the load factor method are more likely to be controlled by load groups involving lateral loads than is the case with the AASHO methods. The shape of the interaction curve is quite different from the sloping straight line of the AASHO procedure. The load factor procedure calls for design for at least a minimum eccentricity ratio of 0.1 (e/t) for tied columns, which is not an AASHO requirement.

Article 1.5.22(A)—General

The general column provisions of the proposed specification covers minimum eccentricities in agreement with the 1971 ACI Building Code, and the standard limitations on column reinforcement ratio.

Article 1.5.22(B)—Column Section Capacities

Biaxial Bending is taken from the BPR Criteria provisions with a few minor modifications. The 1971 ACI Building Code does not give rules for handling biaxial bending. It was felt that such rules were desirable in this proposed specification since biaxial bending is encountered more frequently by bridge designers than by building designers.

Article 1.5.22(C)—Slenderness Effects in Columns

Slenderness effects in columns is taken verbatim from the 1971 ACI Building Code. The provisions constitute a major revision in the procedure for designing "long" columns from AASHO procedures or previous ACI provisions.

Article 1.5.25(B)—Fatigue of Reinforcement

The permissible range of stress in straight reinforcement was changed from 20,000 psi to 21,000 psi based on extensive fatigue research conducted under NCHRP Project 4-7 "Fatigue Strength of High-Yield Reinforcing Bars." (Report not yet in print). As indicated in Figure 1, this provision only has application on bridge decks or on very short span bridges where the dead load to total load ratio is less than 0.25.

Article 1.5.26(A)—Flexural Stress Limitations

General steel stress limitations are the 21,000 psi stress range under live load plus impact as a fatigue restriction and a maximum stress for long span reinforced concrete bridges of 36,000 psi. The latter limitation is based on judgment and will rarely be of concern since it will occur only for unsusually long spans. This upper limit on steel stress will provide a minimum overall safety factor of 60,000 / 36,000, or 1.667.

Article 1.5.26(B)—Corrosive Environments

For bridges without a field tested waterproof membrane protective system on the decks, 10 percent additional negative moment reinforcement is required in order to limit the theoretical structural crack width in unprotected decks to a range of about 0.006 to 0.009 in. (as calcuated by the formula in the BPR Criteria).

The 1971 ACI Code does not contain crack width provisions per se. However, Section 10.6 of the ACI Code does contain provisions on "Distribution of flexural reinforcement in Beams and One-Way Slabs." These provisions are intended to encourage distribution of reinforcement throughout a flange or tension area rather than concentration of reinforcement in a small number of large bars. A similar provision is generally considered unnecessary for most bridge designs where the member sizes are often determined by the space required to accommodate the reinforcement and where bars are normally spaced at close intervals.

Direct consideration of crack width limitations has been eliminated in the proposed specifications for reasons as discussed above and because of three possibly more important considerations. These are:

(1) Structural crack width has not been conclusively related to corrosion of reinforcement or deterioration even in specimens exposed to extremely corrosive environments and for crack widths associated with flexural stresses of up to 50,000 psi.[14]

Preliminary indications from a current crack width-corrosion study at the University of Texas at Austin[15] are that "the difference in rusting between a bar stress level of 20 ksi and 36 ksi appears to be only nominal."

(2) "Crack width is inherently subject to wide scatter even in careful laboratory work, and is influenced by shrinkage and other time dependent effects. Great accuracy in crack control computations is not warranted. . ." (quoted from the commentary to the 1971 ACI Building Code). Another quote pertinent to this point, taken from PCA Development Laboratory Bulletin D53, "High Strength Bars as Concrete Reinforcement—Part 2. Control of Flexural Cracking" is "The mechanism of crack formation is such that wide experimental scatter must inherently exist. Crack spacings and widths from 50 percent over to 50 percent under average values are entirely normal."

(3) Exposure tests indicate that concrete quality, adequate compaction, and ample concrete cover may be of greater importance for corrosion protection than crack width at the concrete surface.

For all the reasons cited above, it is believed unwarranted to impose severe restrictions on steel stresses to limit theoretical maximum crack widths. However, in unprotected decks the provisions of 10 percent additional negative moment reinforcement, as proposed, will restrict the maximum structural crack width to a reasonable value. In design calculations, the procedure also has the practical advantage of eliminating a check of steel stress at service load level, which is required by the BPR Criteria.

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[14] "Report 2, Tensile Crack Exposure Tests, Results of Tests of Reinforced Concrete Beams, 1955-1963" by E.C. Roshore, U.S. Army Engineer's Waterways Experiment Station, Corps of Engineers, Vicksburg, Miss., November 1964.

[15] Project No. 3-5-68-112, P.M. Ferguson and J.T. Houston, Project Supervisors, University of Texas at Austin.

Article 1.5.27—Deflections

The BPR Criteria contains stiffness limitations for reinforced concrete bridges based on the vibration characteristics of the structure under live load. Based on calculated deflections and field experience, it is believed that such restrictions are unnecessary for concrete bridges. In place of vibration characteristics or live load deflection limitations the proposed specifications contain recommended superstructure thickness limitations selected to provide acceptable long-time deflections under creep and shrinkage. These thickness limitations as summarized in Table 1.5.27 in Article 1.5.27 (reproduced in Figure 3 for convenience) are nearly

TABLE 1.5.27

RECOMMENDED MINIMUM THICKNESS* FOR CONSTANT DEPTH MEMBERS** (t_{min} in feet)	
Slabs with main reinforcement parallel and transverse to traffic S = effective span in feet	$t_{min} = 0.33 + \frac{S}{30}$ but no less than 0.542 feet
Tee-Beams S = actual span in feet	$t_{min} = 0.5 + \frac{S}{18}$
Box Girders S = actual span in feet	$t_{min} = 0.5 + \frac{S}{20}$

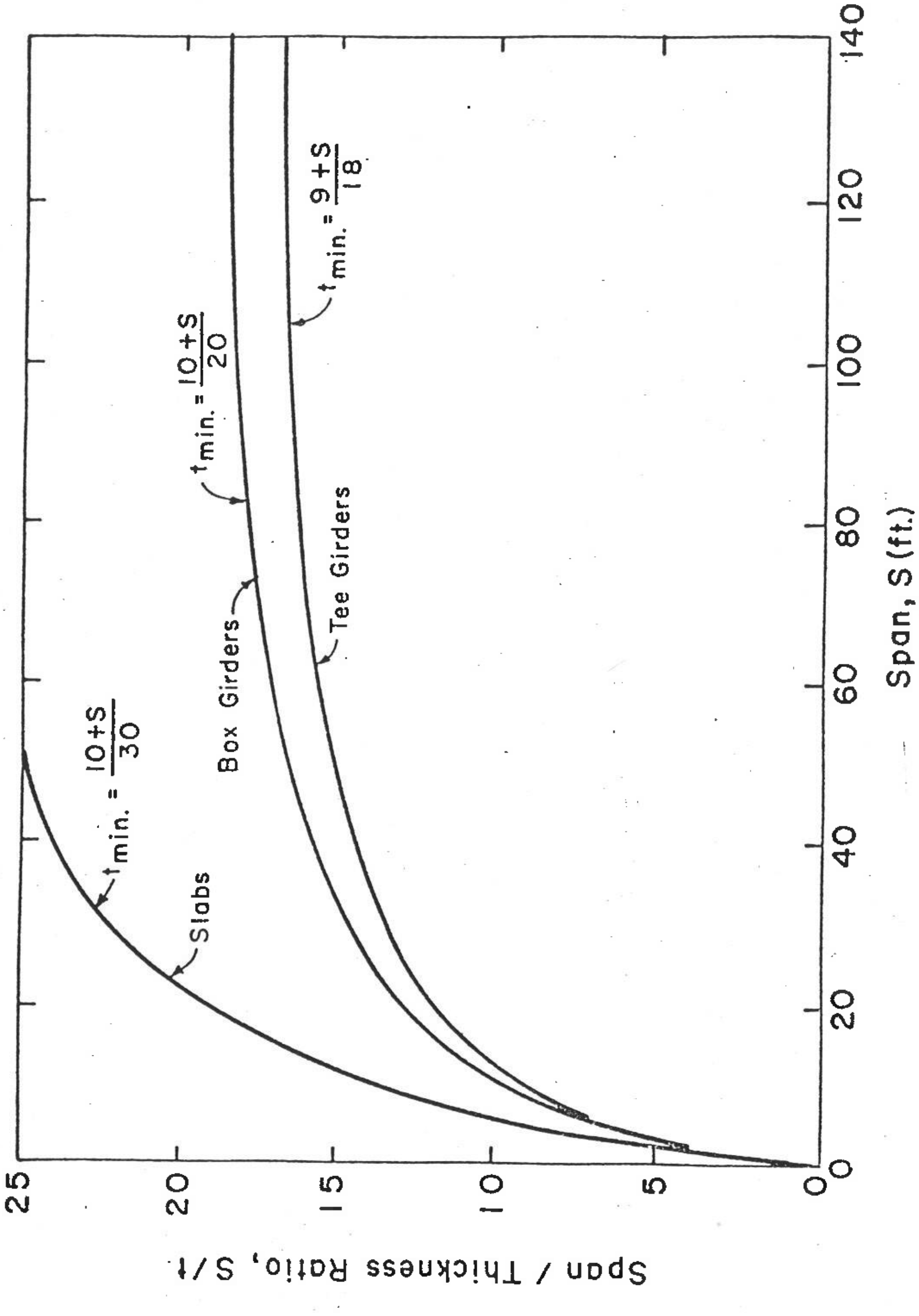
*Recommended values for continuous spans; simple spans should have about 10 percent greater thickness.

**When variable depth members are used, table values may be adjusted to account for change in relative stiffness of positive and negative moment sections.

FIG. 3

identical to the span-depth ratio limitations proposed in the "short form" of the BPR Criteria developed by the California Division of Highways Bridge Department. These thickness recommendations, expressed as span to thickness ratios, are shown graphically in Figure 4.

The calculated live load deflections for the seven bridge designs summarized in the Appendix are shown graphically in Figure 5 (in terms of span/deflection), for both the working stress and load-factor designs. The graph shows span (LL + I Deflection) ratios for the various designs as follows:



Span / Thickness Ratio, S/t
25
20
15
10
5
0
0
20
40
60
80
100
120
140
Span, S (ft.)
Slabs
$t_{min.} = \frac{10+S}{30}$
Box Girders
$t_{min.} = \frac{10+S}{20}$
Tee Girders
$t_{min.} = \frac{9+S}{18}$

FIG. 4

SPAN/CALCULATED LL DEFLECTION

	Working Stress Designs	Load Factor Designs
Slab Bridges	1350 to 1800	1300 to 1650
Tee Beam Bridges	3500 to 3900	2650 to 3500
Box Girder Bridges	2400 to 7000	2100 to 6000

It can be seen from Figure 5 and the above table that the live load plus impact deflections for these concrete bridges are all substantially less than the 1/800 or 1/1000 span deflection limitations specified for steel bridges. Except for slab bridges, the live load deflections obtained are 1/7 to 1/3 those permitted for steel bridge designs. For this reason, no live load deflection limitations are presented in the proposed specifications for reinforced concrete bridges.

In place of the live load deflection provisions, the proposed specification presents recommendations for minimum thickness as a means for limiting long-time deflections. These provisions are presented as "recommendations" since shallower depths might be used in cases where specific consideration is given to limiting long-time deflections. For example, long-time deflections may be reduced substantially by use of compression reinforcement as inferred by Table 1.5.27A in Article 1.5.27.

The method of calculating deflections presented in Article 1.5.27 was used in making the deflection calculations. This method was taken from the 1971 ACI Building Code.

Article 1.5.28—Overload

The specified overloads, D + 5/3 (L + I) equal to double live load on one lane only, and D + 2.2 (L + I) are the same expressions as used in the load factor design equations. Therefore, the designer need not consider overload as a separate item in design. For the specified overloads, the strength provisions of the specification will limit the stress in reinforcement to 69.2 percent of the yield strength. This percentage limit is less than that permitted under overloads by AASHO Article 1.11.1. For emphasis these two requirements are compared in Fig. 6.

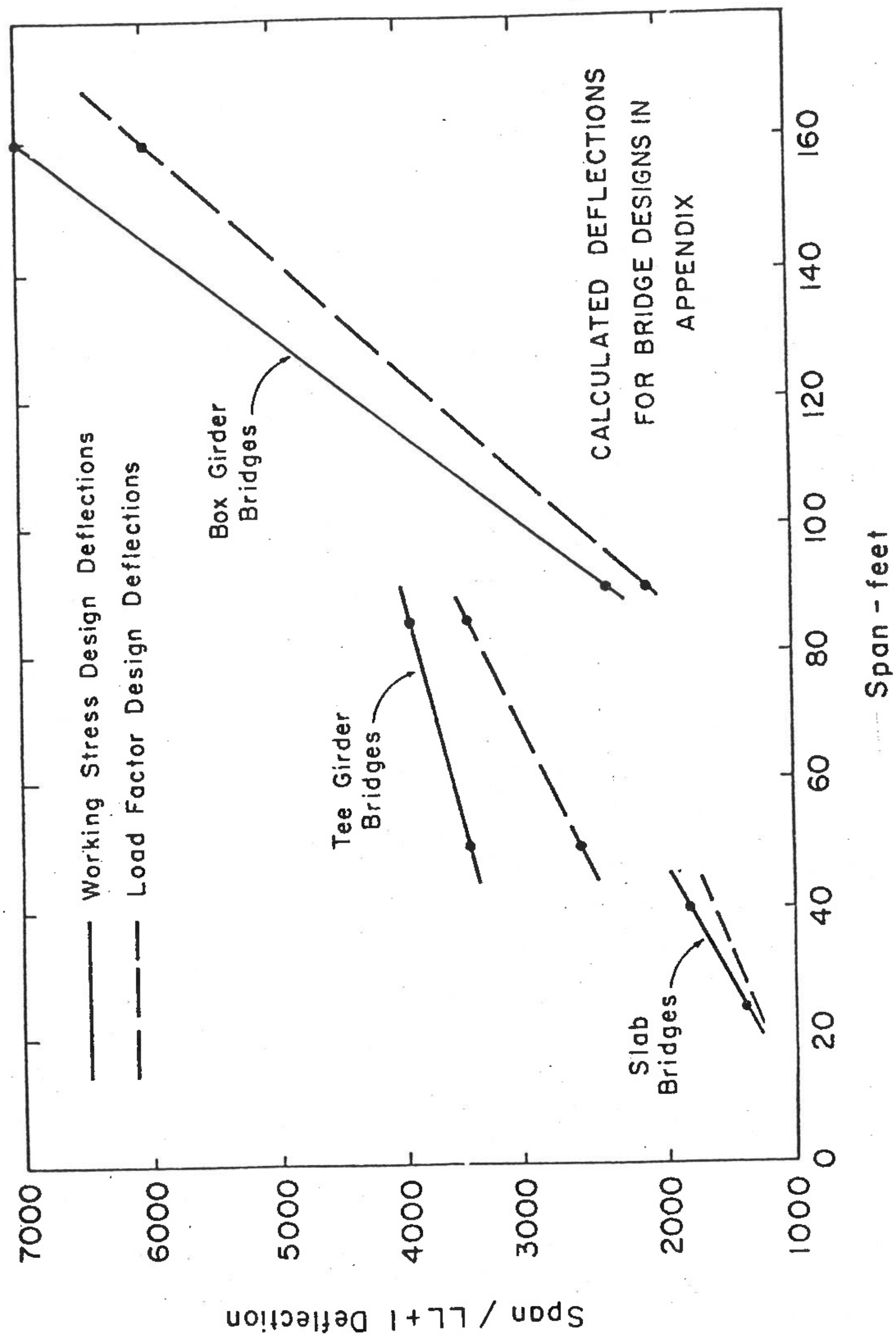

CALCULATED DEFLECTIONS
FOR BRIDGE DESIGNS IN
APPENDIX
Working Stress Design Deflections
Load Factor Design Deflections
Box Girder Bridges
Tee Girder Bridges
Slab Bridges
Span / LL+I Deflection
7000
6000
5000
4000
3000
2000
1000
0
20
40
60
80
100
120
140
160
Span - feet

FIG. 5

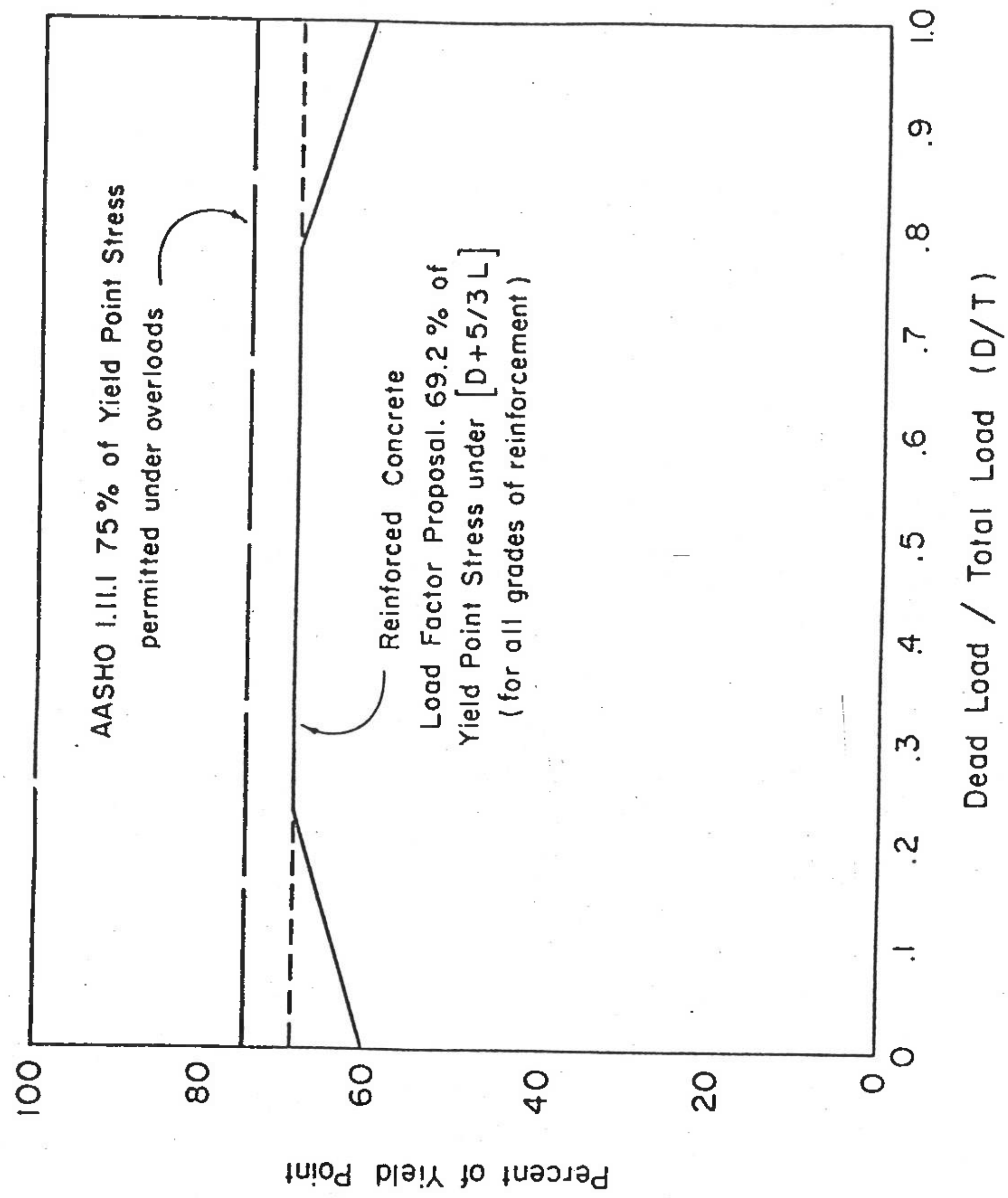

AASHO 1.11.1 75% of Yield Point Stress
permitted under overloads
Reinforced Concrete
Load Factor Proposal. 69.2 % of
Yield Point Stress under [D+5/3 L]
(for all grades of reinforcement)
Percent of Yield Point
100
80
60
40
20
0
.1
.2
.3
.4
.5
.6
.7
.8
.9
1.0
Dead Load / Total Load (D/T)

FIG. 6

ALL BOLTS EXCEPT HIGH STRENGTH BOLTS, SHALL HAVE SINGLE SELF-LOCKING TS 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, , IN(ASTM A325) HIGH STRENGTH BOLTS SHALL NOT EXCEED THE VALUES OBTAINED FROM THE FOLLOWING EQUATION:

$$= 13,500 - .22$$

WHERE = 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:

$$C + (0.555T) = S$$

WHERE C = THE COMPUTED UNIT STRESS IN SHEAR
T = THE COMPUTED UNIT STRESS IN TENSION
S = THE ALLOWABLE UNIT STRESS IN SHEAR

1.7.6-WROUGHT IRON (THIS SECTION DELETED)

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

A. CAST STEEL AND DUCTILE IRON

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

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

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

1.7.117 - CAMBER

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

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

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

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

LOAD FACTOR DESIGN

1.7.118 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.

1.7.119 NOTATION

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

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

1.7.120 LOADS

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

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

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

The maximum loads are the loadings specified in Article 1.7.124.

1.7.121 DESIGN THEORY

The moments, shears and other forces shall be determined by assuming elastic behavior of the structure except as modified in Article 1.7.125(A)(3).

The members shall be proportioned by the methods specified in Articles 1.7.125 through 1.7.136 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 1.7.124.

Service behavior shall be investigated as specified in Articles 1.7.137 through 1.7.139.

1.7.122 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.

1.7.123 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.

1.7.124 MAXIMUM DESIGN LOADS

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

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

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.

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

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

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

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

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

1.7.125 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)

(continued on next page)

(Article 1.7.125(A) continued from page 33)

below shall be considered compact sections and the maximum strength shall be as computed:

$M_u = F_y Z$

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

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

(a) Projecting flange element

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

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

(b) Web thickness

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

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

(c) Lateral bracing

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

or

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

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 1.7.125(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.

*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.

(d) Maximum axial compression

$$P \leq 0.15 F_y A$$

where A is the area of the cross section.

(e) Maximum shear force

$$V \leq 0.55 F_y d t_w$$

(2) Article 1.7.125(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 1.7.125(A) are given in Table 1.

TABLE 1

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

(3) In the design of a continuous beam of compact section complying with the provisions of Article 1.7.125(A)(1), negative moments over supports determined by elastic analysis may be reduced by a maximum of 10%. Such reduction shall be accompanied by an increase in maximum positive moment in the span equal to the average decrease of the negative moments in the span. The reduction shall not apply to negative moments produced by cantilever loading.

(B) *Braced Non-Compact Sections*

For rolled or fabricated I-shaped beams not meeting the requirements of Article 1.7.125(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

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

When

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

(b) Web thickness

$$D/t_w \leq 150$$

where D is the clear unsupported distance between flange components.

(c) Spacing of lateral bracing for compression flange

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

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 1.7.125(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 1.7.125(A) and (B) may be computed by straight line interpolation, except that the web thickness must always satisfy Article 1.7.125 (A)(1)(b).

(D) *Unbraced Sections*

(1) For members not meeting the lateral bracing requirement of Article 1.7.125(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 1.7.125(B)(1)(e) the web shall be provided with transverse stiffeners as specified in Article 1.7.125(E).

(3) Members with axial loads in excess of $0.15F_yA$ should be designed as beam-columns as specified in Article 1.7.135.

(E) *Transversely Stiffened Girders*

(1) For girders not meeting the shear requirements of Articles 1.7.125 (A)(1)(e) and 1.7.125(B)(1)(e) transverse stiffeners are required for the web. For girders with transverse stiffeners but without longitudinal stiffeners the thickness of the web shall meet the requirement:

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

For different grades of steel this limit is:

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

(2) The maximum bending strength of transversely stiffened girders meeting the requirements of Article 1.7.125(E)(1) shall be computed by Articles 1.7.125(B) or 1.7.125(D)(1) as applicable subject to the requirement of Article 1.7.125(E)(4).

(3) The shear capacity of beams and girders with webs fulfilling the requirements of Article 1.7.125(E)(1) shall be computed as:

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

where:

$$V_p = 0.58F_yDt_w$$

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

D = clear, unsupported distance between flange components.

d_o = distance between transverse stiffners.

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

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

(5) Transverse stiffeners shall be spaced at a distance, d_o, according to shear capacity as specified in Article 1.7.125(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 1.7.125(B)(1)(c).

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

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

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

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

where b' is the projecting width of the stiffener.

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

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

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

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

C is computed by Article 1.7.125(E)(3)

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

$$I = d_o\, t_w^3\, J$$

where:

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

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

(F) *Longitudinally Stiffened Girders*

(1) Longitudinal stiffeners shall be required when the web thickness is less than that specified by Article 1.7.125(E)(1) and shall be placed at a distance $D/5$ from the inner surface of the compression flange.

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

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

For different grades of steel, this limit is:

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

(2) The maximum bending strength of longitudinally stiffened girders meeting the requirements of Article 1.7.125(F)(1) shall be computed by Articles 1.7.125(B) or Article 1.7.125(D)(1) as applicable, subject to the requirement of Article 1.7.125(E)(4).

(3) The shear capacity of girders with one longitudinal stiffener shall be computed by Article 1.7.125(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 1.7.125(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:

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

In computing I and r values above, a centrally located web strip not more than $18t_w$ in width shall be considered as a part of the longitudinal stiffener. Transverse stiffeners for girder panels with longitudinal stiffeners shall be designed according to Article 1.7.125(E)(5) except that the depth of subpanels shall be used instead of the total panel depth, D. In addition, the section modulus of the transverse stiffener shall be not less than:

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

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

1.7.126 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 1.7.125(A) through 1.7.125(D) shall be applicable except that in computing the maximum strength by Article 1.7.125(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 1.7.125(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 1.7.125(E)(1) or 1.7.126(B).

For girders with one longitudinal stiffener and transverse stiffeners, the provisions of Article 1.7.125(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:

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

1.7.127 COMPOSITE BEAMS AND GIRDERS

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

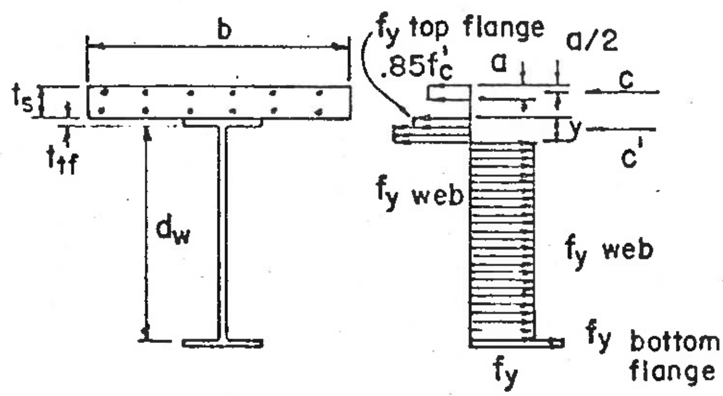
(a) The maximum strength of any section shall not be less than the sum of the computed moments at that section multiplied by the appropriate load factors.

(b) The web of the steel section shall be designed to carry the total external shear and must satisfy the applicable provisions of Articles 1.7.125 and 1.7.126. 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.

1.7.128 POSITIVE MOMENT SECTIONS OF COMPOSITE BEAMS AND GIRDERS

(A) *Compact Sections*

When the steel section satisfies the compactness requirements of Article 1.7.128(A)(2), the maximum strength shall be computed as the resultant moment of the fully plastic stress distribution acting on the section (Figure 1.7.128).



(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) $C = 0.85 f_c{}'bt_s + (A\ Fy)_c$

where b is the effective width of slab,

t_s is the slab thickness,

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

(2) $C = (A\ Fy)_{bf} + (A\ Fy)_{tf} + (A\ Fy)_w$

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

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

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

(3) $C = \Sigma Q_u$

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

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

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

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

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

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

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

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

(2) Composite beams qualify as compact when their steel section meets the requirements of Articles 1.7.125(A)(1)(b) and 1.7.125 (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

1.7.128(A)(2) the maximum strength of the section shall be taken as the moment at first yielding.

(C) *General*

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

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

1.7.129 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 1.7.125 and 1.7.126 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.

1.7.130 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 Articles 1.7.127, 1.7.128 and 1.7.129. 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 1.7.130(E).

(B) *Lateral Distribution*

The live load bending moment for each box girder shall be determined in accordance with Article 1.7.104.

(C) *Web Plates*

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

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

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

(D) *Tension Flanges*

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

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

(E) *Compression Flanges*

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

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

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

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

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

in which c shall be taken as

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

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

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

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

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

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

$\phi = 0.125k^3$ when $n = 1$.

w=width of flange between longitudinal stiffeners or distance from a web to the nearest longitudinal stiffener,

n=number of longitudinal stiffeners,

k=buckling coefficient which shall not exceed 4.

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

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

For greater values of w/t, but not exceeding $6650\sqrt{k}/\sqrt{F_y}$, the buckling stress of the flange, including stiffeners is given by Article 1.7.130 (E)(2) in which c shall be taken as:

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

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

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

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

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

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

where b'= width of any outstanding stiffener element,

t'= thickness of outstanding stiffener element.

(F) *Diaphragms*

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

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

1.7.131 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 1.7.101(A)(3).

(B) *Design of Connectors*

The number of shear connectors shall be determined in accordance with Article 1.7.101(A)(2), and checked for fatigue in accordance with Article 1.7.101(A)(1) and 1.7.101(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.

1.7.132 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 1.7.125 through 1.7.131 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.

1.7.133 NONCOMPOSITE HYBRID GIRDERS

(A) *Compact Sections*

The equation of Article 1.7.125(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 1.7.125(B) for the maximum strength of compact sections shall be replaced by the expression

$$M_u = F_{yf} SR$$

For symmetrical sections,

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

where

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

For unsymmetrical sections,

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

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

(C) *Unbraced Noncompact Sections*

The equation of Article 1.7.125(D)(1) for the maximum strength of unbraced noncompact sections shall be replaced by the expression

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

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

(D) *Transversely Stiffened Girders*

The equation of Article 1.7.125(E)(3) for the shear capacity of transversely stiffened girders shall be replaced by the expression

$$V_u = V_p C$$

The equation for A in Article 1.7.125(E)(5) is not applicable to hybrid girders.

1.7.134 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 1.7.133(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.

1.7.135 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.G. Johnston, "Guide to Design Criteria for Metal Compression Members," John Wiley and Sons, Inc., New York, 1966.

(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:

$$\frac{P}{0.85 A_s F_{cr}} + \frac{M\,C}{M_u\left(1 - \dfrac{P}{A_s F_e}\right)} \leq 1.0$$

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

where

F_{cr} is buckling stress as determined by the Equations of Article 1.7.135(A)(1)

M_u is the maximum strength as determined by Articles 1.7.125(A)(B) or (D)

$F_e = \dfrac{\pi^2 E}{\left(\dfrac{KL_c}{r}\right)^2}$ the Euler buckling stress in the plane of bending,

C is the equivalent moment factor,

$M_p = F_y Z$, the full plastic moment of the section,

Z is the plastic section modulus,

$\dfrac{KL_c}{r}$ is the effective slenderness ratio in the plane of bending.

(2) *Equivalent Moment Factor*

If the ends of the beam-column are restrained from sidesway in the plane of bending by diagonal bracing or attachment to an adjacent laterally braced structure, then the value of equivalent moment factor, C, may be computed by the formula:

$$C = 0.6 + 0.4a, \text{ but not less than } 0.4$$

where a is the ratio of the numerically smaller to the larger end moment. The ratio a is positive when the two end moments act in an opposing sense (i.e., one acts clockwise and the other acts counterclockwise) and negative when they act in the same sense. In all cases, factor C may be taken conservatively as unity.

1.7.136 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 1.7.124. The maximum strengths multiplied by the reduction factors are listed in Table 3.

(2) *Welds*

The ultimate strength of weld metal in groove welds shall be equal to or greater than that of the base metal. The ultimate strength of the weld metal in fillet welds need not match the strength of the base metal. However, the welding procedure and weld metal shall be selected to insure sound welds. The effective weld area shall be taken as defined in Article 1.7.29.

(3) *Bolts and Rivets*

In proportioning fasteners, the nominal diameter shall be used except when a shear plane intersects the threads.

High-strength bolts preferably shall be used for fasteners subject to tension or combined shear and tension.

For combined tension and shear in bearing type connections, bolts and rivets shall be proportioned so that the shear stress does not exceed:

$$F_{vc} \leq \sqrt{F^2 - (0.6f_t)^2}$$

where F_v = shear strength of the fastener, ϕF, as given in Table 3.

f_t = tensile stress due to the applied load.

TABLE 3

Type of Fastener	Strength (ϕF)
Groove Weld(1)	1.00 F_y
Fillet Weld(2)	0.45 f_u
Low-Carbon Steel Bolts	
ASTM A307	
Tension	27 ksi
Shear(3)	25 ksi
Power-Driven Rivets	
ASTM A502	
Shear — Grade 1	25 ksi
Shear — Grade 2	30 ksi
High-Strength Bolts	
ASTM A325	
Tension(5)	76 ksi
Shear (Bearing-Type)(3)(4)(5)	54 ksi

(1)—F_y = yield point of connected material.

(2)—F_u = minimum strength of the welding rod metal but not greater than the tensile strength of the connected parts.

(3)—When a shear plane intersects the bolt threads, the root area shall be used.

(4)—Bearing stresses in bearing-type connections shall not exceed the tensile strength of the connected material.

(5)—For A325 bolts the tensile strength decreases for diameters greater than ⅞ in. The design value listed is for bolts up to ⅞ in. diameter. For diameters greater than ⅞ in. diameter the design value shall be computed as 0.56 F_u for tension and 0.45 F_u for shear where F_u is the ASTM minimum tensile strength of the bolt.

(4) *Friction Joints*

Friction joints shall be designed to prevent slip at the overload in accordance with Article 1.7.137(C). Maximum strength of the bolts need not be considered in the design of such joints.

(B) Connections

(1) *Splices*

Splices may be made with rivets, with high-strength bolts or by the use of welding. Splices, whether in tension, compression, bending or shear, shall be designed for not less than the average of the calculated stress resultant at the point of the splice and the strength of the member at the same point, but in any event not less than 75% of the maximum strength of the member. Where a section changes at a splice, the maximum strength of the splice shall be at least 75% of the smaller section spliced.

The maximum strength of the member shall be determined by the gross section for compression members. For members primarily in bending, the gross section shall be used, except that if more than 15% of each flange area is removed, that amount removed in excess of 15% shall be deducted. For tension members and splice material, the gross section shall be used unless the net section area is less than 85% of the corresponding gross area, in which case that amount removed in excess of 15% shall be deducted.

(2) *Bolts Subjected to Prying Action by Connected Parts*

Bolts required to support applied load by means of direct tension shall be proportioned for the sum of the external load and tension resulting from prying action produced by deformation of the connected parts. The total tension should not exceed the values given in Table 3 of Article 1.7.136.

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.

1.7.137 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 $0.8F_yS$. In the case of moment redistribution under the provisions of Article 1.7.125(A)(3), the above limitation shall apply to the modified moments but not to the original moments.

(B) Composite Beams

For composite beams the moment caused by $D + \frac{5}{3}(L+I)$ shall not exceed 95% of the moment at first yielding in the section. For such beams designed for Group IA loading, the moment caused by $D+2.2(L+I)$ shall not exceed 95% of the moment at first yielding in the section. In computing dead load stresses the presence or absence of temporary supports during the construction shall be considered.

(C) Friction Joints

The shear caused by the loading, $D+\frac{5}{3}(L+I)$ in friction-type high-strength bolted joints shall not exceed 21,000 psi for ASTM 325 bolts.

For combined shear and tension in friction-type joints where applied forces reduce the total clamping force on the friction plane, the maximum shear stress shall not exceed the values obtained from the following equations:
For A325 bolts

$$f_v = 21{,}000\ [1 - f_t/0.53F_u)]$$

where F_u is the tensile strength of the bolt,

f_t is the applied tensile stress.

1.7.138 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 1.7.101(A).

(C) Hybrid Beams and Girders

Hybrid girders shall be designed for fatigue in accordance with Article 1.7.112 (C).

1.7.139 DEFLECTION

The control of deflection of steel or of composite steel and concrete structures shall conform to the provisions of Article 1.7.13.

0.079	18.2	29.8	35.7
0.109	23.4	46.8	53.0
0.138	24.5	49.0	63.7
0.168	25.6	51.3	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
.060	9.0	14.0
.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	ALUMINUN 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

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

$$FF = \frac{D}{(EI)}$$

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

$$D = \frac{r}{k}\sqrt{\frac{24E}{f_u}}$$

$$f_c = f_u - \frac{f_u^2}{48E}\left(\frac{kD}{r}\right)^2, \text{PSI}$$

THE RING COMPRESSION STRESS, AT WHICH BUCKLING BECOMES CRITICAL, IN THE ELASTIC BUCKLING ZONE FOR DIAMETERS GREATER THAN

$$D = \frac{r}{k}\sqrt{\frac{24E}{f_u}}$$

$$f_c = \frac{12E}{\left(\frac{kD}{r}\right)^2}, \text{PSI}$$

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 o.3 to 0.5.

1.9.3 MINIMUM HEIGHT OF COVER

The minimum cover for design loads shall be 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 be not 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.

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