TO:

153. MICHAEL J. SHEEHAN, JR.
Bldg. 5, 6th Floor

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R. N. Kamp
DEPUTY CHIEF ENGINEER (STRUCTURES)

The attached sheets are revisions to Standard Specifications for Highway Bridges. These pages should be inserted immediately in your manual.

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Page

82-6	- Formulas (6-1) and (6-2) rewritten. Formula (6-3) corrected and rewritten.
82-7	- Formulas (6-4) and (6-7) rewritten. - Formulas (6-5), (6-7) and (6-8) corrected and rewritten.
82-8	- Formula (6-10) partially rewritten. - Formula (6-11) corrected and rewritten.
82-9	- Formulas (6-12) and (6-13) rewritten.
82-10	- Formulas (6-15), (6-16), (6-17), (6-18), (6-19) rewritten.
82-11	- Formulas (6-20) and (6-21) rewritten.
82-12	- Formulas (6-22), (6-23), (6-24), (6-25) and (6-26) rewritten.
82-13	- Formulas (6-27) and (6-28) rewritten.
82-14	- Formula (6-29) rewritten.
82-15	- Formula (6-30) rewritten.
85	- No change.
85-1	- Article 1.6.5 revised to agree with AASHTO.
85-2	- Blank page.
86	- Article 1.6.5 deleted from this page
115	- Article 1.7.8 - Sub-item "Concrete", revised.
116	- Top four lines deleted.

1.5.31—DESIGN ASSUMPTIONS

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

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

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

(3) Stress in reinforcement below the specified yield strength, f_y, for the grade of steel used shall be taken as E_s times the steel strain. For strains greater than that corresponding to f_y, the stress in the reinforcement shall be considered independent of strain and equal to f_y.

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

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

(6) The requirements of Article 1.5.31(A)(5) may be considered satisfied by an equivalent rectangular concrete stress distribution which is defined as follows: A concrete stress of $0.85f_c'$ shall be assumed uniformly distributed over an equivalent compression zone bounded by the edges of the cross section and a straight line located parallel to the neutral axis at a distance $a = \beta_1 c$ from the fiber of maximum compressive strain. The distance c from the fiber of maximum strain to the neutral axis is measured in a direction perpendicular to that axis. The fraction β_1 shall be taken as 0.85 for strengths f_c' up to 4000 psi and shall be reduced continuously at a rate of 0.05 for each 1000 psi of strength in excess of 4000 psi.

1.5.32—FLEXURE

(A) Maximum Reinforcement of Flexural Members

(1) For flexural members, the reinforcement ratio, ρ, provided shall not exceed 0.75 of that ratio, ρ_b, which would produce balanced conditions for the section under flexure.

(2) Balanced conditions exist at a cross section when the tension reinforcement reaches its specified yield strength, f_y, just as the concrete in compression reaches its assumed ultimate strain of 0.003.

(B) Rectangular Sections With Tension Reinforcement Only

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

$$M_u = \varphi\left[A_s f_y d\left(1 - 0.6\rho\frac{f_y}{f_c'}\right)\right] \quad (6\text{-}1)$$

$$= \varphi\left[A_s f_y\left(d - \frac{a}{2}\right)\right] \quad (6\text{-}2)$$

where

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

(2) The balanced reinforcement ratio, ρ_b, for rectangular sections with tension reinforcement only is given by:

$$\rho_b = \frac{0.85\beta_1 f_c'}{f_y} \cdot \frac{87,000}{87,000 + f_y} \quad (6\text{-}3)$$

(C) I- and T-Sections With Tension Reinforcement Only

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

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

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

where

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

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

(3) The balanced reinforcement ratio, ρ_b, for I- and T-sections with tension reinforcement only is given by:

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

where

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

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

(D) Rectangular Sections With Compression Reinforcement

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

$$M_u = \varphi\left[(A_s - A'_s)f_y\left(d - \frac{a}{2}\right) + A'_s f_y(d - d')\right] \qquad (6\text{-}6)$$

where

$$a = \frac{(A_s - A'_s)f_y}{0.85f'_c\, b}$$

and the following condition shall be satisfied:

$$\frac{(A_s - A'_s)}{bd} \geq 0.85\beta_1 \frac{f'_c d'}{f_y d} \cdot \frac{87,000}{87,000 - f_y} \qquad (6\text{-}7)$$

(2) When the value of $(A_s - A_s')/bd$ is less than the value given by Eq. (6-7), so that the stress in the compression reinforcement is less than the yield strength, f_y, or when effects of compression reinforcement are neglected, the moment strength may be computed by the equations in (B)(1), except when a general analysis is made based on stress and strain compatibility using the assumptions given in Article 1.5.31.

(3) The balanced reinforcement ratio, ρ_b, for rectangular sections with compression reinforcement is given by:

$$\rho_b = \frac{0.85\beta_1 f'_c}{f_y} \cdot \frac{87,000}{87,000 + f_y} + \rho' \frac{f'_s}{f_y} \qquad (6\text{-}8)$$

where

f'_s = stress in compression reinforcement

$$= 87,000\left(1 - \frac{d'}{d} \cdot \frac{87,000 + f_y}{87,000}\right) \leq f_y$$

(E) Other Cross Sections

For other cross sections the design moment strength, $M_u = \phi M_t$, shall be computed by a general analysis based on stress and strain compatibility using the assumptions given in Article 1.5.31. The requirements of Article 1.5.32(A) shall also be satisfied.

1.5.33—COMPRESSION MEMBERS WITH OR WITHOUT FLEXURE

(A) General Requirements

(1) The design of cross sections subject to combined flexure and axial load shall be based on stress and strain compatibility using the assumptions given in Article 1.5.31. Slenderness effects shall be included according to the requirements of Article 1.5.34.

(2) All members subjected to a compression load shall be designed for an eccentricity, e, equal to the greater of

(a) that corresponding to the maximum design moment which accompanies this compression load, or
(b) $0.05h$ for spirally reinforced compression members, or $0.10h$ for tied compression members, about either axis, or
(c) 1 in. about either axis.

(B) Compression Member Strengths

The following provisions may be used as a guide to define the range of the load-moment interaction relationship for members subjected to combined flexure and axial load.

(1) Pure compression

The axial design load strength in pure compression, P_o, may be computed by:

$$P_o = \varphi\left[0.85f'_c(A_g - A_{st}) + A_{st}f_y\right] \qquad (6\text{-}9)$$

Concentric loading is a hypothetical loading condition since all members subjected to a compression load shall be designed for eccentricities not less than the value given in Article 1.5.33(A)(2).

(2) Pure flexure

the assumptions given in Article 1.5.31, or the applicable equations for flexure given in Article 1.5.32 may be used to compute the design moment strength, M_u, in pure flexure.

(3) Balanced conditions

Balanced conditions for a cross section are defined in Article 1.5.32(A)(2). For a rectangular section with reinforcement in one or two faces and located at approximately the same distance from the axis of bending, the balanced load, P_b, and balanced moment, M_b, may be computed by:

$$P_b = \varphi\left[0.85f'_c ba_b + A'_s f'_s - A_s f_y\right] \qquad (6\text{-}10)$$

and

$$M_b = P_b e_b = \varphi\left[0.85f'_c ba_b\left(d - d'' - \frac{a_b}{2}\right) + A'_s f'_s (d - d' - d'') + A_s f_y d''\right] \qquad (6\text{-}11)$$

where

$$a_b = \left(\frac{87{,}000}{87{,}000 + f_y}\right)\beta_1 d$$

and

$$f'_s = 87,000\left(1 - \frac{d'}{d}\right)\left(\frac{87,000 + f_y}{87,000}\right) \leq f_y$$

(4) Combined flexure and axial load

The design strength under combined flexure and axial load shall be based on stress and strain compatibility using the assumptions given in Article 1.5.31. The strength of a cross section is controlled by tension when the axial design load strength, P_u, is less than P_b (or e is greater than e_b). The strength of a cross section is controlled by compression when the axial design load strength, P_u, is greater than P_b (or e is less than e_b).

The combined axial load and moment strength shall be multiplied by the appropriate capacity reduction factor, ϕ, for spirally reinforced or tied compression members as given in Article 1.5.30(A). The value of ϕ may be increased linearly from the value for compression members to the value for flexure as the axial design load strength, P_u, decreases from $0.10f_c'A_g$ or P_b, whichever is smaller, to zero.

(C) Biaxial Loading

In lieu of a general section analysis based on stress and strain compatibility for a loading condition of biaxial bending, the design strength of non-cicrular members subjected to biaxial bending may be computed by the following approximate expressions:

$$P_{uxy} = \frac{1}{(1/P_{ux}) + (1/P_{uy}) - (1/P_o)} \qquad (6\text{-}12)$$

when the applied axial design load,

$$P_u \geqslant 0.1f_c'A_g$$

or

$$\frac{M_x}{M_{ux}} + \frac{M_y}{M_{uy}} \leq 1 \qquad (6\text{-}13)$$

when the applied axial design load,

$$P_u < 0.1f_c'A_g$$

1.5.34—SLENDERNESS EFFECTS IN COMPRESSION MEMBERS

(A) General Requirements

(1) The design of compression members shall be based on forces and moments determined from an analysis of the structure. Such an analysis shall take into account the influence of axial loads and variable moment of inertia on member stiffness and fixed-end moments, the effect of deflections on the moments and forces, and the effects of the duration of the loads.

(2) In lieu of the procedure described in paragraph (1), the design of compression members may be based on the approximate procedure given in Article 1.5.34(B).

(B) Approximate Evaluation of Slenderness Effects

(1) The unsupported length, l_u, of a compression member shall be taken as the clear distance between slabs, girders, or other members capable of providing lateral support for the compression member. Where haunches are present, the unsupported length shall be measured to the lower extremity of the haunch in the plane considered.

(2) The radius of gyration, r, may be taken equal to 0.30 times the overall dimension in the direction in which stability is being considered for rectangular compression members, and 0.25 times the diameter for circular compression members. For other shapes, r may be computed for the gross concrete section.

(3) For compression members braced against sidesway, the effective length factor, k, shall be taken as 1.0, unless an analysis shows that a lower value may be used. For compression members not braced against sidesway, the effective length factor, k, shall be determined with due consideration of cracking and reinforcement on relative stiffness, and shall be greater than 1.0.

(4) For compression members braced against sidesway, the effects of slenderness may be neglected when kl_u/r is less than $34 - 12M_1/M_2$. For compression members not braced against sidesway, the effects of slenderness may be neglected when kl_u/r is less than 22. For all compression members with kl_u/r greater than 100, an analysis as defined in Article 1.5.34(A)(1) shall be made. M_1 = value of smaller design end moment on compression member calculated from a conventional elastic analysis, positive if member is bent in single curvature, negative if bent in double curvature. M_2 = value of larger design end moment on compression member calculated from a conventional elastic analysis, always positive.

(5) Compression members shall be designed using the applied axial design load, P_u, from a conventional elastic analysis and a magnified moment, M_c defined by:

$$M_c = \delta M_2 \tag{6-14}$$

where

$$\delta = \frac{C_m}{1 - \dfrac{P_u}{\varphi P_c}} \geq 1.0 \tag{6-15}$$

and

$$P_c = \frac{\pi^2 EI}{(k\ell_u)^2} \tag{6-16}$$

In lieu of a more precise calculation, EI may be taken either as

$$EI = \frac{\dfrac{E_c I_g}{5} + E_s I_s}{1 + \beta_d} \tag{6-17}$$

or conservatively

$$EI = \frac{\dfrac{E_c I_g}{2.5}}{1 + \beta_d} \tag{6-18}$$

where β_d is the ratio of maximum design dead load moment to maximum design total load moment, always positive. For members braced against sidesway and without transverse loads between supports, C_m may be taken as

$$C_m = 0.6 + 0.4\frac{M_1}{M_2} \tag{6-19}$$

but not less than 0.4.

For all other cases C_m shall be taken as 1.0.

(6) When design of compression members is governed by the minimum eccentricities specified in Article 1.5.33(A)(2), M_2 in Eq. (6-14) shall be based on the specified minimum eccentricity with conditions of curvature determined by either of the folloiwing:

(a) When the actual computed eccentricities are less than the specsified minimum, the computed end moments may be used to evaluate the conditions of curvature.

(b) If computations show that there is no eccentricity at both ends of the member, conditions of curvature shall be based on a ratio of M_1/M_2 equal to one.

(7) When compression members are subject to bending about both principal axes, the moment about each axis shall be amplified by δ, computed from the corresponding conditions of restraint about that axis.

(8) In structures which are not braced against sidesway, the flexural members shall be designed for the total magnified end moments of the compression members at the joint.

(9) When a group of compression members on one level comprise a bent, or when they are connected integrally to the same superstructure, and collectively resist the sidesway of the structure, the value of δ shall be computed for the member group. P_u and P_c shall be taken as the summation of P_u and P_c for all members in the group. In designing each member in the group, δ shall be taken as the larger of (a) the value computed for the group as a whole, or (b) the value computed for the individual compression member assuming its ends to be braced against sidesway.

1.5.35—SHEAR

(A) Shear Strength

(1) The design shear stress, v_u, shall be computed by:

$$v_u = \frac{V_u}{\varphi b_w d} \tag{6-20}$$

where b_w shall be taken as the width of web and d shall be taken as the distance from the extreme compression fiber to the centroid of the longitudinal tension reinforcement.

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

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

(3) The shear stress carried by the concrete, v_c, shall be calculated according to Article 1.5.35(B). When v_u exceeds v_c, shear reinforcement shall be provided according to Article 1.5.35(C). Whenever applicable, the effects of torsion* shall be added.

(B) Permissible Shear Stress

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

(2) The shear stress carried by the concrete, v_c, may be computed by:

$$v_c = 1.9\sqrt{f_c'} + 2500\,\rho_w \frac{V_u d}{M_u} \tag{6-21}$$

but v_c shall not exceed $3.5(f_c')^{1/2}$ The quantity $V_u d/M_u$ shall not be taken greater than 1.0, where M_u is the applied design moment occurring simultaneously with V_u at the section considered.

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

(3) For members subjected to axial compression, v_c may be computed by:

$$v_c = 2\left(1+0.0005\frac{N_u}{A_g}\right)\sqrt{f'_c} \qquad (6\text{-}22)$$

The quantity N_u/A_g shall be expressed in psi.

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

$$v_c = 2\left(1+0.002\frac{N_u}{A_g}\right)\sqrt{f'_c} \qquad (6\text{-}23)$$

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

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

(a) When f_{ct} is specified, the shear stress, v_c, shall be modified by substituting $f_{ct}/6.7$ for $(f_c')^{1/2}$, but the value of $f_{ct}/6.7$ used shall not exceed $(f_c')^{1/2}$.

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

(C) Design of Shear Reinforcement

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

$$A_v = \frac{(v_u - v_c)b_w s}{f_y} \qquad (6\text{-}24)$$

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

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

$$A_v = \frac{(v_u - v_c)b_w s}{f_y(\sin\alpha + \cos\alpha)} \qquad (6\text{-}25)$$

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

$$A_v = \frac{(v_u - v_c)b_w d}{f_y \sin\alpha} \qquad (6\text{-}26)$$

in which $(v_u - v_c)$ shall not exceed $3(f_c')^{1/2}$.

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

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

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

(4) When $(v_u - v_c)$ exceeds $4(f_c')^{1/2}$, the maximum spacings given in Article 1.5.10(C) shall be reduced by one-half.

(5) The value of $(v_u - v_c)$ shall not exceed $8(f_c')^{1/2}$.

(D) Shear-friction

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

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

(3) The shear stress, v_u, shall not exceed $0.2f_c'$ nor 800 psi.

(4) The required area of reinforcement, A_{vf}, shall be computed by

$$A_{vf} = \frac{V_u}{\varphi f_y \mu} \qquad (6\text{-}27)$$

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

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

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

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

(E) Horizontal Shear Design for Composite Concrete Flexural Members

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

(2) Full transfer of horizontal shear forces may be assumed when all of the following are satisfied: (a) the contact surfaces are clean and intentionally roughened, (b) minimum ties are provided in accordance with paragraph (6), (c) web members are designed to resist the entire vertical shear, and (d) all shear reinforcement is anchored into all intersecting components.

When all of the above are not satisfied, horizontal shear shall be fully investigated.

(3) The horizontal shear stress, v_{dh}, may be computed at any cross section as

$$v_{dh} = \frac{V_u}{\varphi b_v d} \qquad (6\text{-}28)$$

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

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

(a) When ties are not provided, but the contact surfaces are clean and intentionally roughened, permissible $v_h = 80$ psi.

(b) When the minimum tie requirements of paragraph (6) are provided and the contact surfaces are clean but not intentionally roughened, permissible $v_h = 80$ psi.

(c) When the minimum tie requirements of paragraph (6) are provided and the contact surfaces are clean and intentionally roughened, permissible $v_h = 350$ psi.

(d) When v_{dh} exceeds 350 psi. design for horizontal shear shall be made in accordance with Article 1.5.35(D).

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

(6) Ties for horizontal shear

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

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

(7) Measure of roughness

Internal roughness may be assumed only when the contact surface is roughened. clean, and free of laitance. Roughness shall have a full amplitude of approximately 1/4 in.

(F) Special Provisions for Slabs and Footings

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

(a) The slab or footing acting as a wide beam. with a critical section extending in a plane across the entire width and located at a distance d from the face of the concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with Articles 1.5.35(A) through (C).

(b) Two-way action for the slab or footing, with a critical section perpendicular to the plane of the slab and located so that its periphery is a minimum and approaches no closer than $d/2$ to the periphery of the concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with paragraphs (2) and (3).

(2) The peripheral shear stress shall be computed by:

$$v_u = \frac{V_u}{\varphi b_o d} \tag{6-29}$$

in which V_u and b_o are taken at the critical section defined in (b). The peripheral shear stress, v_u, shall not exceed the shear stress carried by the concrete. $v_c = 4(f_c')^{1/2}$, unless shear reinforcement is provided in accordance with (3), in which case v_u shall not exceed $6(f_c')^{1/2}$.

(3) Shear reinforcement consisting of bars or wires anchored in accordance with Article 1.5.21 may be provided. For design of such shear reinforcement, shear stresses shall be investigated at the critical section defined in (b) and at successive sections more distant from the support; and the shear stress, v_c, carried by the concrete at any section shall not exceed $2(f_c')^{1/2}$. Where v_u exceeds v_c, the shear reinforcement shall be provided according to Article 1.5.35(C).

1.5.36—PERMISSIBLE BEARING STRESS

(A) Bearing stress in concrete on loaded area, f_b, shall not exceed $0.85\phi f_c'$, except as provided below.

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

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

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

1.5.37—SERVICEABILITY REQUIREMENTS

(A) Application

For flexural members designed with reference to load factors and strengths by LOAD FACTOR DESIGN, stresses at service load shall be limited to satisfy the requirements for fatigue in Article 1.5.38, and the requirements for distribution of reinforcement in Article 1.5.39. The requirements for deflection control in Article 1.5.40 shall also apply.

(B) Service Load Stresses

For investigation of service load stresses to satisfy the requirements of Articles 1.5.38 and 1.5.39, the straight-line theory of stress and strain in flexure shall be used and the assumptions given in Article 1.5.27 shall apply.

1.5.38—FATIGUE STRESS LIMITS

(A) Concrete

The maximum compressive stress in the concrete shall not exceed $0.5f_c'$ at sections where stress reversals occur caused by live load plus impact at service load. This stress limit shall not apply to concrete deck slabs.

(B) Reinforcement

The range between a maximum and minimum stress in straight reinforcement caused by live load plus impact at service load shall not exceed 21,000 psi.* Bends in primary reinforcement shall be avoided in regions of high stress range.

1.5.39—DISTRIBUTION OF FLEXURAL REINFORCEMENT

Tension reinforcement shall be well distributed in the zones of maximum tension. When the design yield strength, f_y, for tension reinforcement exceeds 40,000 psi, cross sections of maximum positive and negative moment shall be so proportioned that the calculated stress in the reinforcement at service load, f_s, in kips per sq. in., does not exceed the value computed by:

$$f_s = \frac{z}{\sqrt[3]{d_c A}} \tag{6-30}$$

but f_s shall not be greater than $0.6f_y$, where

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

d_c = thickness of concrete cover measured from the extreme tension fiber to the center of the bar located closest thereto, in.

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* Applicable primarily to concrete deck slabs and short span slab bridges where the dead load to total load ratio is less than approximately 0.25.

$p^* = A^*_{s}/bd$, RATIO OF PRESTRESSING STEEL.

$p' = A'_{s}/bd$, RATIO OF COMPRESSION REINFORCEMENT.

Q = STATICAL MOVEMENT OF CROSS SECTIONAL AREA, ABOVE OR BELOW THE LEVEL BEING INVESTIGATED FOR SHEAR, ABOUT THE CENTROID.

SH = CONCRETE SHRINKAGE LOSS.

s = LONGITUDINAL SPACING OF THE WEB REINFORCEMENT.

t = AVERAGE THICKNESS OF THE FLANGE OF A FLANGED MEMBER.

T_o = STEEL STRESS AT JACKING END.

T_x = STEEL STRESS AT ANY POINT X.

v = ULTIMATE HORIZONTAL SHEAR STRESS.

V_c = SHEAR CARRIED BY CONCRETE.

V_u = SHEAR DUE TO ULTIMATE LOAD AND EFFECT OF PRESTRESSING.

μ = FRICTION CURVATURE COEFFICIENT.

α = TOTAL ANGULAR CHANGE OF PRESTRESSING STEEL PROFILE IN RADIANS FROM JACKING END TO POINT X.

1.6.3--DESIGN THEORY

MEMBERS SHALL MEET THE ULTIMATE STRENGTH AND ALLOWABLE STRESS REQUIREMENTS AS SPECIFIED.

DESIGN SHALL BE BASED ON ULTIMATE STRENGTH AND BEHAVIOR AT SERVICE CONDITIONS FOR ALL LOAD STAGES THAT MAY BE CRITICAL DURING THE LIFE OF THE STRUCTURE FROM THE TIME OF PRESTRESSING.

1.6.4--BASIC ASSUMPTIONS

THE FOLLOWING ASSUMPTIONS ARE MADE FOR DESIGN PURPOSES:

(1) STRAINS VARY LINEARLY OVER THE DEPTH OF THE MEMBER THROUGHOUT THE ENTIRE LOAD RANGE.

(2) BEFORE CRACKING, STRESS IS LINEARLY PROPORTIONAL TO STRAIN.

(3) AFTER CRACKING, TENSION IN THE CONCRETE IS NEGLECTED.

1.6.5 — LOAD FACTORS

Load factors are multiples of the design load applied to the structure to ensure its safety. The computed ultimate capacity shall not be less than the largest value obtained from formulas 6.1, 6.2, 6.3 and 6.4. Members subject to combinations of loads and forces shall be designed for the combined effect.

$$\text{Group I} = \frac{1.30}{\phi} \times \left[D + \frac{5}{3}(L + I) \right] \qquad (6\text{-}1)$$

For all loadings less than H2O, 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.

$$\text{Group IA} = \frac{1.30}{\phi} \times [D + 2.2\,(L+I)] \qquad (6\text{-}2)$$

$$\text{Group II} = \frac{1.30}{\phi} \times [D+W+F+SF+B+S+T] \qquad (6\text{-}3)$$

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.

$$\text{Group III} = \frac{1.30}{\phi} \times [D + (L+I) + CF + 0.3W + WL + F + LF] \qquad (6\text{-}4)$$

Except for the ϕ factors listed below, the symbols in the above formulas represent the moments, shears or forces caused by the loads and the effects described in Article 1.2.22.

ϕ = Factor on section strength =

1.0 for factory produced precast prestressed concrete members
0.95 for post-tensioned cast-in-place concrete members
0.90 for shear

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1.6.6--ALLOWABLE STRESSES ******

THE DESIGN OF PRECAST PRESTRESSED MEMBERS ORDINARILY SHALL BE BASED ON $f'_c = 5000$ PSI. AN INCREASE TO 6000 PSI IS PERMISSIBLE WHERE, IN THE ENGINEERS JUDGMENT, IT IS REASONABLE TO EXPECT THAT THIS STRENGTH WILL BE OBTAINED CONSISTENTLY. STILL HIGHER CONCRETE STRENGTHS MAY BE CONSIDERED ON AN INDIVIDUAL AREA BASIS. IN SUCH CASES, THE ENGINEER SHALL SATISFY HIMSELF COMPLETELY THAT THE CONTROLS OVER MATERIALS AND FABRICATION PROCEDURES WILL PROVIDE THE REQUIRED STRENGTHS. THE PROVISIONS OF THIS CHAPTER ARE EQUALLY APPLICABLE TO PRESTRESSED CONCRETE STRUCTURES OR COMPONENTS DESIGNED WITH LOWER CONCRETE STRENGTHS.

(A) PRESTRESSING STEEL

TEMPORARY STRESS BEFORE LOSS DUE TO CREEP AND SHRINKAGE.............................. $0.70f'_s$

STRESS AT SERVICE LOAD(*) AFTER LOSSES ... $0.80f_y^*$

(OVERSTRESSING TO $0.75f'_s$ FOR PRETENSIONED MEMBERS AND 0.80 FOR POST TENSIONED MEMBERS MAY BE PERMITTED FOR SHORT PERIODS OF TIME PROVIDED THAT THE STRESS, AFTER TRANSFER TO THE CONCRETE IN PRETENSIONING OR SEATING OF ANCHORAGE IN POST TENSIONING, DOES NOT EXCEED $0.70f'_s$)

* SERVICE LOAD CONSISTS OF ALL LOADS CONTAINED IN SECTION 1.2.1 BUT DOES NOT INCLUDE OVERLOAD PROVISIONS.

(B) CONCRETE

(1) TEMPORARY STRESSES BEFORE LOSSES DUE TO CREEP AND SHRINKAGE

COMPRESSION

PRETENSIONED MEMBERS .... $0.60f'_{ci}$

POST-TENSIONED MEMBERS .... $0.55f'_{ci}$

SHALL GOVERN. FOR BOLTED CONNECTIONS THE FASTENER SPECIFICATIONS SHALL GOVERN

B. MALLEABLE CASTINGS

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

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

C. CAST IRON

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

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

1.7.7 -BRONZE OR COPPER-ALLOY

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

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

1.7.8 -BEARING ON MASONRY

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

GRANITE	800
SANDSTONE AND LIMESTONE	400

CONCRETE:

Refer to Articles 1.5.26(A)(3) and 1.5.36.

Revised January, 1975

DETAILS OF DESIGN

1.7.9 -EFFECTIVE LENGTH OF SPAN

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

1.7.10 -DEPTH RATIOS

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

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

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

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

1.7.11 -LIMITING LENGTHS OF MEMBERS

For compression members, the slenderness ratio, KL/r, shall not exceed 120 for main members, or those in which the major stresses result from dead or live load, or both; and shall not exceed 140 for secondary members, or those whose primary purpose is to brace the structure against lateral or longitudinal force, or to brace or reduce the unbraced length of other members, main or secondary.

In determining the radius of gyration, r, for the purpose of applying the limitations of the KL/r ratio, the area of any portion of a member may be neglected provided that the strength of the member as calculated without using the area thus neglected and the strength of the member as computed for the entire section with the KL/r ratio applicable thereto, both equal or exceed the computed total stress that the member must sustain.