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NEW YORK STATE DEPARTMENT OF TRANSPORTATION

SUBJECT: REVISION TO STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES

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DEPUTY CHIEF ENGINEER (STRUCTURES)

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

i		No change.
ii		Revised due to revisions.
xvii		Revised due to revisions.
xviii		No change.
xix		Corrected page numbers.
26 thru 2-3	Art. 1.2.20	Revised Earthquake Material.
26-4	Art. 1.2.21	Moved to accommodate Art. 1.2.20.
31		No change.
32		Removed on Adjacent Prestressed Concrete Box Beams. (covered by Art. 1.3.1 (b))
33		Revised paragraph 3.
34		No change.
63		No change.
64		Blank page.
113		Blank page.
114		Blank page.
117	Art. 1.6.5 A	Changed Stress at Service Load.
118		No change.
149	Art. 1.6.13	Corrected V_c = .
150		No change.
169	Art. 1.7.5	Corrected spelling in Table (CARBON).
170		No change.
173		No change.
174	Art. 1.7.12	Revised second paragraph. (to agree with AASHTO)
179		No change.
180 thru 180-1	Art. 1.7.21	Revised entire artic .
180-2		Blank page.
181		Deleted old portion of Art. 1.7.21. No change in balance of page.
182		No change.
260-25		No change.
260-26		Added missing formula (f).
260-27	Art. 1.7.172	Corrected first R formula.
260-28		No change.
268-1		No change.
268-2 thru 268-3		Added Art. 1.8.10 (from 1977 Interim Specs)
268-4		Added blank page

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DESIGN

Article

1.2.16 – UPLIFT

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

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

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

1.2.17 – FORCE OF STREAM CURRENT, FLOATING ICE AND DRIFT

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

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

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

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

1.2.18 – BUOYANCY

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

1.2.19 – EARTH PRESSURE

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

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

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

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

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

1.2.20 - EARTHQUAKE STRESSES

In regions where earthquakes may be anticipated, structures shall be designed to resist earthquake motions by considering the relationship of the site to active faults, the seismic response of the soils at the site, and the dynamic response characteristics of the total structure in accordance with the following criteria.

A. Equivalent Static Force Method:

For structures with supporting members of approximately equal stiffness, an equivalent horizontal force (EQ) may be applied to the structure. The distribution of the force shall consider the stiffness of the superstructure and supporting members, abutment restraint, and the deflected position of the structure.

1. EQ = CFW

EQ = The equivalent static horizontal force applied at the center of gravity of the structure.
F = Framing Factor
F = 1.0 for structures where single columns or piers resist the horizontal forces.
F = 0.8 for structures where continuous frames resist horizontal forces applied along the frame.
W = The total dead load weight of the structure.

2. C = ARS/Z

C = Combined Response Coefficient.
For information of evaluation of this factor, refer to Item 2, 1975 AASHTO Interim Specifications - Bridges. In lieu of a more detailed analysis, the coefficient "C" may be assumed to be 0.20 A.
A = Maximum expected acceleration at bedrock at the site (in g's). Design accelerations may be obtained from the Seismic Design Map of New York State (Fig. 1.2.20.A) assigning the following values:

Zone A	A = 0.05
Zone B	A = 0.075
Zone C	A = 0.10

A more exact determination of Maximum Expected Acceleration at bedrock may be obtained from Anticipated Rock Acceleration Map of New York State, (Fig. 1.2.20B).

g = 32.2 ft/sec.^{2} (9.81 m/sec.^{2})
R = Normalized rock response.
S = Soil amplification spectral ratio.
Z = Reduction for ductility and risk assessment.

$$T = 0.32 \; \frac{W}{P} \text{ or } \frac{W \text{ (in kg)}}{P \text{ (in N)}}$$

T = The period of vibration of the structure (sec.)
P = Total uniform force required to cause a one-inch (.025 m) maximum horizontal deflection of the whole structure.
The period of vibration may also be computed using dynamic analysis techniques.

B. Response Spectrum Method:

For complex structures, a response spectrum dynamic approach should be used for seismic analysis.

C. Special Cases:

Structures adjacent to active faults, sites with unusual geologic conditions, unusual structures, and structures having a fundamental period greater than 3.0 sec. will be considered special cases. These structures will be required to be designed using current seismicity. soil response and dynamic analysis techniques.

D. Design of Restraining Features:

Restraining features to limit the displacement of the superstructure - i.e., hinge ties, shear blocks, etc. - shall be designed for the following force:

EQ = 0.50 A x contributing DL minus column shears due to EQ.

"Contributing DL" is determined by examining the entire frame. For example, a simple span fixed at one end and sliding at the other will have the entire superstructure as the "Contributing DL" for longitudinal forces at the fixed abutment, while one-half of the superstructure DL will act at each abutment for transverse forces.

For a frame, such as a two-span structure, the full length of the bridge should be used as the contribution length in the longitudinal direction. The resulting force can be reduced by deducting the shear in the column due to earthquake.

For hinge restrainers, use 0.50 A x DL of the smaller of the two frames and deduct the column shears due to EQ.

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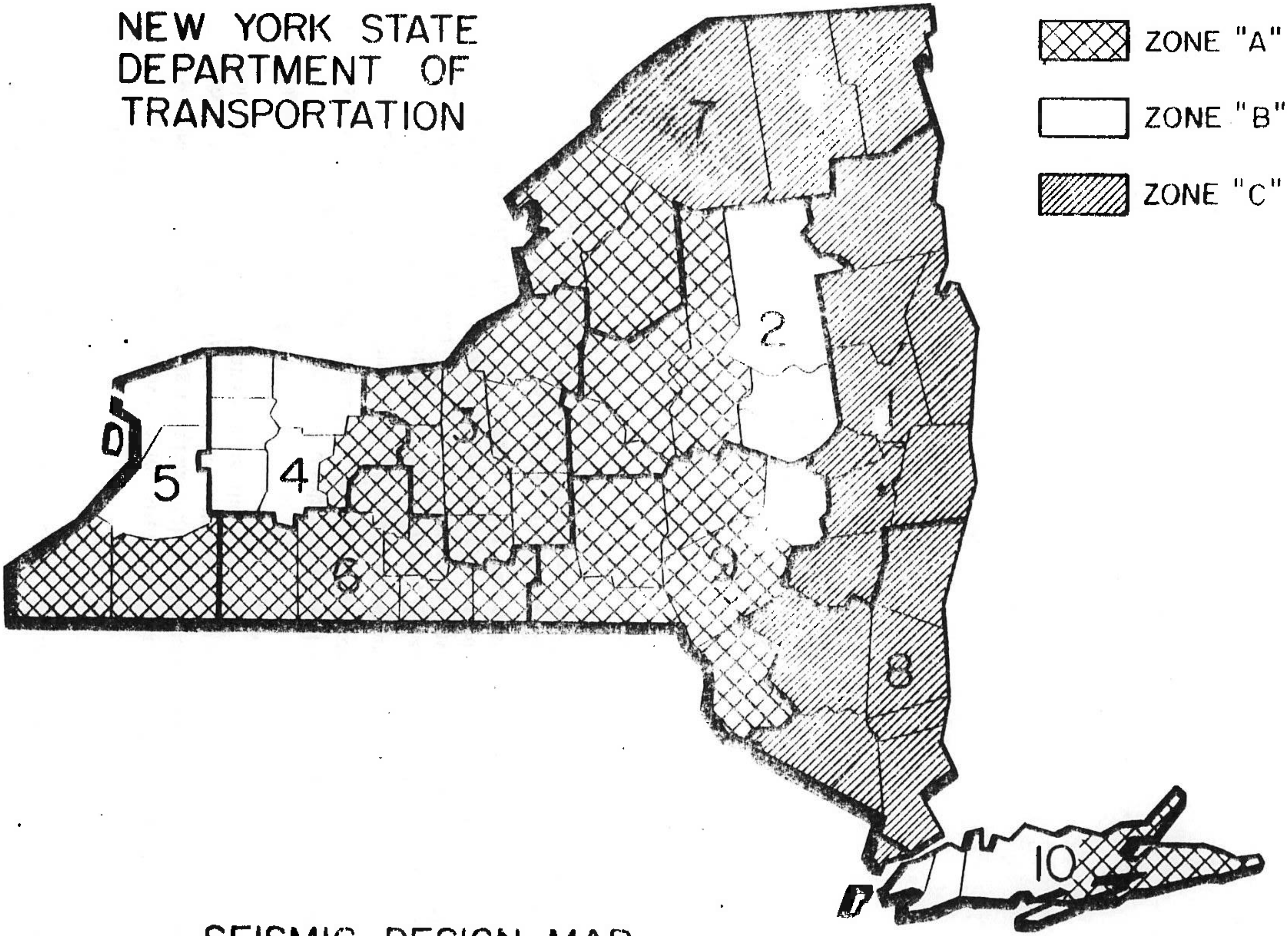
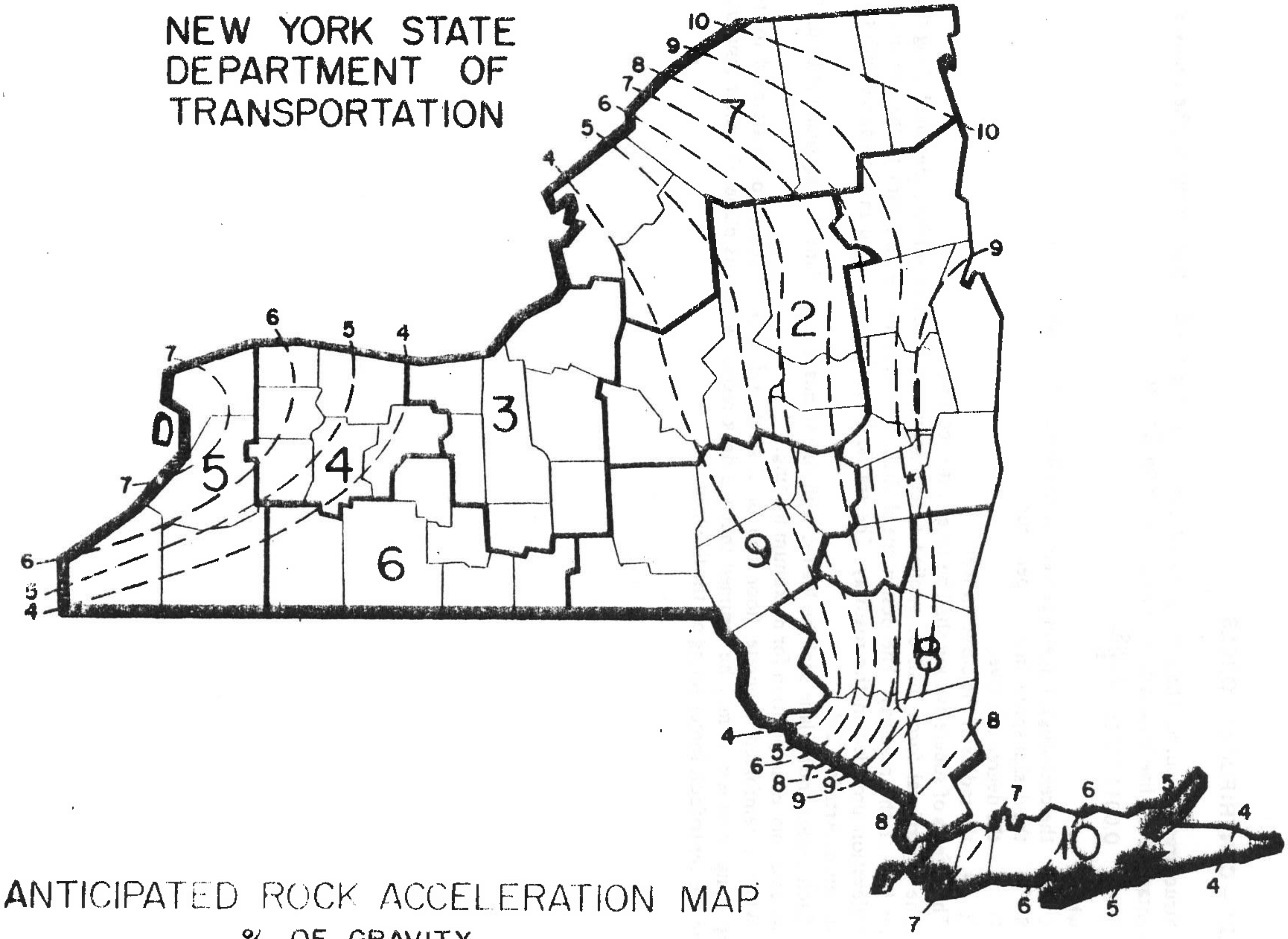


SEISMIC DESIGN MAP

Fig. 1.2.20.A



ANTICIPATED ROCK ACCELERATION MAP

% OF GRAVITY

NOT EXPECTED TO BE EXCEEDED IN 50 YEARS

Fig. 1.2.20.3

1.2.21 – CENTRIFUGAL FORCES

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

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

where

C = the centrifugal force in percent of the live load, without impact.
S = the design speed, in miles per hour.
D = the degree of curve.
R = the radius of the curve, in feet.

The effects of superelevation shall be taken into account.

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

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

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

Section 3 — DISTRIBUTION OF LOADS

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

A. Position of Loads for Shear

In calculating end shears and end reactions in transverse floor beams and longitudinal beams and stringers, no longitudinal distribution of the wheel load shall be assumed for the wheel or axle load adjacent to the end at which the stress is being determined.

Lateral distribution of the wheel load shall be that produced by assuming the flooring to act as a simple span between stringers or beams. For loads in other positions on the span, the distribution for shear shall be determined by the method prescribed for moment, except that the calculation of horizontal shear in rectangular timber beams shall be in accordance with Article 1.10.2.

B. Bending Moment in Stringers and Longitudinal Beams

In calculating bending in longitudinal beams or stringers, no longitudinal distribution of the wheel loads shall be assumed. The lateral distribution shall be determined as follows:

1. Interior Stringers and Beams*****

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

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

Kind of Floor	Bridge Designed for one traffic lane	Bridge Designed for two or more traffic lanes
Timber:		
Plank	S/4.0	S/3.75
Strip 4" thick or multiple layer floors over 5" thick	S/4.5	S/4.0
Strip 6" or more thick	S/5.0 If S exceeds 5′ use footnote[2].	S/4.25 If S exceeds 6.5′ use footnote[2].
Concrete:		
On Steel or Precast Concrete Stringers	S/7.0 If S exceeds 10′ use footnote[2].	S/5.5 If S exceeds 14′ use footnote[2].
On Concrete T-Beams	S/6.5 If S exceeds 6′ use footnote[2].	S/6.0 If S exceeds 10′ use footnote[2].
On Timber Stringers	S/6.0 If S exceeds 6′ use footnote[2].	S/5.0 If S exceeds 10′ use footnote[2].
Concrete box girders See footnote[1]	S/8.0 If S exceeds 12′ use footnote[2].	S/7.0 If S exceeds 16′ use footnote[2].
On Steel Box Girders	(See Art. 1.7.103)	
On Prestressed Concrete Spread Box Beams	See Art. 1.6.24(A)	
Steel Grid:		
(less than 4" thick)	S/4.5	S/4.0
(4" or more)	S/6.0 If S exceeds 6.0′ use footnote[2].	S/5.0 If S exceeds 10.5′ use footnote[2].

S = average stringer spacing in feet.

Note 1 The sidewalk live load (see Article 1.2.11) shall be omitted for interior and exterior box girders designed in accordance with the wheel load distribution indicated herein.

Note 2 In this case the load on each stringer shall be the reaction of the wheel loads, assuming the flooring between the stringers to act as a simple beam. However, the provisions of Article 1.2.6 do not apply.

2. Outside Roadway Stringers and Beams

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

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

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

When the outside roadway beam or stringer supports the sidewalk live load as well as traffic live load and impact, the allowable stress in the beam or stringer may be increased 25 percent for the combination of dead load, sidewalk live load, traffic live load, and impact, providing the beam is of no less carrying capacity than would be required if there were no sidewalks. When combination of sidewalk live load as well as traffic live load plus impact governs the design and the structure is to be designed by load factor method, 1.25 may be used as the Beta factor in place of 1.67. For straight stingers, in no case shall a fascia stringer have less load carrying capacity than an interior stringer. For stringers of equal strength, fascia stringer camber shall be equal to that provided for interior stringers.

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

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

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

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

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

COMMENTARY: ART. 1.3.1(B)(2)(a), ART. 1.3.2(B)

Certain percentages of overstress allowed in Article 1.3.1(B)(2)(a) and Article 1.3.2(B) are based on the remote possibility of this maximum combination of load occurring at the same time. The proposed Beta factors for load factor design in both cases are set on the basis of the assumption that the combined loading with live load twice as much as dead load will approximately produce the permitted percentage of overstress which are specified for service load design.

(b) Concrete Box Girders

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

The wheel load distribution to the exterior girder shall be

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

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

3. Total Capacity of Stringers

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

C. Bending Moment in Floor Beams (Transverse)

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

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

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

S = spacing of beams in feet.

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

E. Splices and Joints

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

F. Bracing

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

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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 Engineer's judgment, it is reasonable to expect that this strength will be obtained consistently. Still higher concrete strengths may be considered on an individual area basis. In such cases, the engineer shall satisfy himself completely that the controls over materials and fabrication procedures will provide the required strengths, the provisions of this chapter are equally applicable to prestressed concrete structures or components designed with lower concrete strengths.

A. Prestressing Steel

Temporary stress before loss due to creep and shrinkage . . . . $0.7f'_s$
Stress at service load (*) after losses . . . . $0.8f^*_y$

(Overstressing to $0.75f'_s$ for pretensioned members and 0.80 for post-tensioned members may be permitted for short periods of time provided that the stress, after transfer to the concrete in pretensioning or seating of anchorage in post-tensioning, does not exceed $0.70f'_s$)

B. Concrete

1. Temporary stresses before losses due to creep and shrinkage

Compression
Pretensioned members . . . . $0.60f'_{ci}$
Post-tensioned members . . . . $0.55f'_{ci}$

Tension
Precompressed Tensile Zone. (An area that is initially under compression during prestressing, but may be in tension under load). No temporary allowable stresses are specified. See Article 1.6.6(B)(2) for allowable stresses after losses.

Other Areas
In tension areas with no pretensioned strands, grouted post-tensioned strands or nonprestressed reinforcement.

200 psi or $3\sqrt{f'_{ci}}$

Where the calculated tensile stress exceeds this value, bonded reinforcement shall be provided to resist the total tension force in the concrete computed on the assumption of an uncracked section. The maximum tensile stress shall not exceed

$$7.5\sqrt{f'_{ci}}$$

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*Service load consists of all loads contained in Section 1.2.1 but does not include overload provisions.

2. Stress at service load after losses have occurred:

Compression 0.4

Tension in the precompressed tensile zone

(a) For members with bonded reinforcement . . . . $6\sqrt{f}$

For severe corrosive exposure conditions, such as coastal areas . . $3\sqrt{f}$

(b) For members without bonded reinforcement . . . .

Tension in Other Areas

Tension in other areas is limited by the allowable temporary stresses specified i Article 1.6.6(B)(1).

3. Cracking stress (refer to Article 1.6.10)

Modulus of rupture from tests or if not available:

For normal weight concrete . . . . $7.5\sqrt{f'_c}$

For sand-lightweight concrete . . . . $.6.3\sqrt{f'_c}$

For all other lightweight concrete . . . . $5.5\sqrt{f'_c}$

4. Anchorage Bearing Stress:

Post-tensioned anchorage at service load. . . . . 3000 psi

(But not to exceed $0.9f'_{ci}$)

1.6.13 – SHEAR (*)

Prestressed concrete members shall be reinforced for diagonal tension stresses. Shear reinforcement shall be placed perpendicular to the axis of the member.

The area of web reinforcement shall be

$$A_v = \frac{(V_u - V_c)\,s}{2 f_{sy}\, jd}$$

but not less than

$$A_v = \frac{100\, b' s}{f_{sy}}$$

where f_{sy} shall not exceed 60,000 psi

where $V_c = 0.06 f'_c b' j\, d$ but not more than $180 b' jd$.

Web reinforcement may consist of:

1. Stirrups perpendicular to the axis of the member
2. Welded wire fabric with wire located perpendicular to the axis of the member.

The spacing of web reinforcement shall not exceed three-fourths the depth of the member.

The critical sections for shear in simply supported beams will usually not be near the ends of the span where the shear is a maximum but at some point away from the ends in a region of high moment.

For the design of the web reinforcement in simply supported members carrying moving loads, it is recommended that shear be investigated only in the middle half of the span length. The web reinforcement required at the quarter points should be used throughout the outer quarters of the span.

For continuous bridges whose individual spans consist of precast prestressed girders, web reinforcement shall be designed for the full length of interior spans and for the interior 3/4 of the exterior span.

(*) The method for design of web reinforcement presented in ACI 318-71 is an acceptable alternate but web reinforcement shall not be less than $A_v = 100\, b' s/f_{sy}$.

1.6.14 – COMPOSITE STRUCTURES

A. General

Composite structures in which the deck is assumed to act integrally with the beam shall be interconnected in accordance with B, C and D of this Article to transfer shear along contact surfaces and to prevent separation of elements.

B. Shear Transfer

Full transfer of the ultimate horizontal shear forces may be assumed when contact surfaces are clean and intentionally roughened, minimum vertical ties are provided in accordance with D of this Article, all stirrups are fully anchored into all intersecting components, and the web members are designed to resist the entire vertical shear. Otherwise, ultimate horizontal shear stress shall be calculated and limited according to C and D of this Article.

C. Shear Capacity

In lieu of the requirements of B of this Article, ultimate horizontal shear stress may be computed by the formula $v=V_uQ/Ib$. To resist the computed shear stress, the following values of shear capacity shall be assumed at the contact surfaces:

When the minimum steel tie requirements of (D) of this Article are met. . . 75 psi (.517MPa)

When the minimum steel tie requirements of (D) of this Article are met and the contact surface of the precast element is clean and intentionally roughened . . . . . . 300 psi (2.068 MPa)

In addition to the above values, for each percent of the contact surface provided by stirrup and vertical tie reinforcement crossing the joint in excess of the percentage provided by the minimum requirements of (D) of this Article . . . . 150 psi (1.034 MPa)

D. Vertical Ties

All web reinforcement shall extend into cast-in-place decks. The minimum total area of vertical ties per linear foot of span shall be not less than the area of two No. 3 bars spaced at 12 in. Web reinforcement may be used to satisfy the vertical tie requirement. The spacing of vertical ties shall not be greater than four times the average thickness of the composite flange and in no case greater than 24 in.

E. Shrinkage Stresses

In structures with a cast-in-place slab on precast beams, the differential shrinkage tends to cause tensile stresses in the slab and in the bottom of the beams. Because the tensile shrinkage develops over an extended time period, the effect on the beams is reduced by creep. Differential shrinkage may influence the cracking load and the beam deflection profile. When these factors are particularly significant the effect of differential shrinkage should be added to the effect of loads.

1.7.5 – BOLTS

In proportioning bolts, the nominal diameter shall be used, except as otherwise noted.

The effective bearing area of a bolt shall be its diameter multiplied by the thickness of the metal on which it bears. In metal less than 3/8 inch thick, countersunk bolts, turned bolts, or ribbed bolts shall not be assumed to carry stress. In metal 3/8 inch thick and over, one-half the depth of countersink shall be omitted in calculating the bearing area.

Allowable unit stresses in pounds per square inch for bolts shall be as listed in the table below:

Type of Bolt	Tension	Bearing	Shear	
			Friction Type Connection	Bearing Type Connection
(A) Low carbon steel bolts Turned bolts (ASTM A-307) and ribbed bolts	13,500*	20,000		11,000
(B) High strength bolts High strength steel bolts (ASTM A-325)	36,000	40,000**	13,500	20,000***

All bolts except high strength bolts, shall have single self-locking nuts or double nuts.

Joints required to resist shear between their connected parts are designated as either friction type or bearing type connections. Shear connections subjected to stress reversal shall be friction type except for secondary members.

Bolts in girder field splices shall be friction type.

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

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

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

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

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

where f_t = tensile stress due to applied loads

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

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

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

*Based on area at the root of thread.

**Does not apply to friction type connections.

***The allowable shear value of bolts for bearing type connections in steel with a yield point less than 42,000 psi shall be reduced by 20% when the end of the splice material is more than 24 inches from the end of the connected member, as measured along the gage line of the bolts.

Where shown in the design drawings and at other locations approved by the designer, oversize, short-slotted, and long-slotted holes may be used with high strength bolts 5/8-in. (15.9 mm) and larger in diameter proportioned to meet the allowable unit stresses, except as hereinafter restricted:

1. Oversize or slotted holes shall be used on secondary members only.
2. Oversize holes are 3/16-in. (4.8 mm) larger than bolts 7/8-in. (22.2 mm) and less in diameter, 1/4-in. (6.4 mm) larger than bolts 1-in. in diameter (25.4 mm), and 5/16-in. (7.9 mm) larger than bolts 1.-1/8-in. (28.6 mm) and greater in diameter. They may be used in any or all plies of friction-type connections. Hardened washers shall be installed over exposed oversize holes.
3. Short-slotted holes are 1/16-in. (1.6 mm) wider than the bolt diameter and have a length which does not exceed the oversize diameter provisions of subparagraph 1 by more than 1/16-in. (1.6 mm). They may be used in any or all plies of friction-type connections. Hardened washers shall be installed over exposed short slotted holes.
4. Long-slotted holes are 1/16-in. (1.6 mm) wider than the bolt diameter and have a length more than allowed in subparagraph 2, but not more than 2-1/2 times the bolt diameter.

In friction-type connections, oversize or slotted holes may be used without regard to direction of loading if one-third more bolts are provided than needed to satisfy the allowable unit stresses, except as herein restricted.

Oversize or slotted holes must be shown on the design drawings.

Long-slotted holes may be used in only one of the connected parts of friction-type connection at an individual faying surface.

Structural plate washers, or a continuous bar not less than 5/16-in. (7.9 mm) in thickness, are required to cover long slots that are in the outer plies of joints. These washers or bars shall have a size sufficient to completely cover the slot after installation.

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 should not be less than 1/25.

For composite girders the ratio of the overall depth of girder (concrete slab, plus steel girder) to the length of span preferably should not be less than 1/25, and the ratio of depth of steel girder alone to length of span preferably should not be less than 1/30.

For trusses the ratio of depth to length of span preferably should not be less than 1/10.

For continuous span depth ratios the span length shall be considered as the distance between the dead load points of contraflexure.

The foregoing requirements as they relate to beam or girder bridges may be exceeded at the discretion of the designer.*

COMMENTARY

The above modifications are intended to remove the mandatory aspects of the present Articles in the 1973 AASHTO Specifications.

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.

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*For considerations to be taken into account when exceeding these limitations, reference is made to "Bulletin No. 19, Criteria for the Deflection of Steel Bridges," available from the American Iron and Steel Institute, Washington, D.C.

The radius of gyration and the effective area for carrying stress of a member containing perforated cover plates shall be computed for a transverse section through the maximum width of perforation. When perforations are staggered in opposite cover plates the cross-sectional area of the member shall be considered the same as for a section having perforations in the same transverse plane.

Actual unbraced length L shall be assumed as follows:

For the top chords of half-through trusses, the length between panel points laterally supported as indicated under Article 1.7.86; for other main members, the length between panel point intersections or centers of braced points or centers of end connections; for secondary members, the length between the centers of the end connections of such member or centers of braced points.

For tension members, except rods, eyebars, cables and plates, the ratio of unbraced length to radius of gyration shall not exceed 200 for main members, 240 for bracing members, and 140 for main members subjected to reversal of stress.

Cross frame members for curved stringers shall be considered main members and shall have a maximum "l/r" of 120.

1.7.12 – DEFLECTION*****

The term "deflection" as used herein shall be the deflection computed in accordance with the assumption made for loading when computing the stress in the member.

Members having simple or continuous spans preferably should be designed so that the deflection due to live load plus impact shall not exceed 1/800 of the span, except on bridges in urban areas used in part by pedestrians whereon the ratio preferably shall be 1/1000.

The deflection of cantilever arms due to live load plus impact preferably should be limited to 1/300 of the cantilever arm except for the case including pedestrian use, where the ratio preferably should be 1/375.

When spans have cross-bracing or diaphragms sufficient in depth or strength to insure lateral distribution of loads, the deflection may be computed for the standard H or HS loading, considering all beams or stringers as acting together and having equal deflection.

The moment of inertia of the gross cross-section area shall be used for computing the deflections of beams and girders. When the beam or girder is a part of a composite member, the live load may be considered as acting upon the composite section.

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

The foregoing requirements as they relate to beam or girder bridges may be exceeded at the discretion of the designer.*

COMMENTARY

The above modifications are intended to remove the mandatory aspects of the present Articles in the 1973 AASHTO Specifications.

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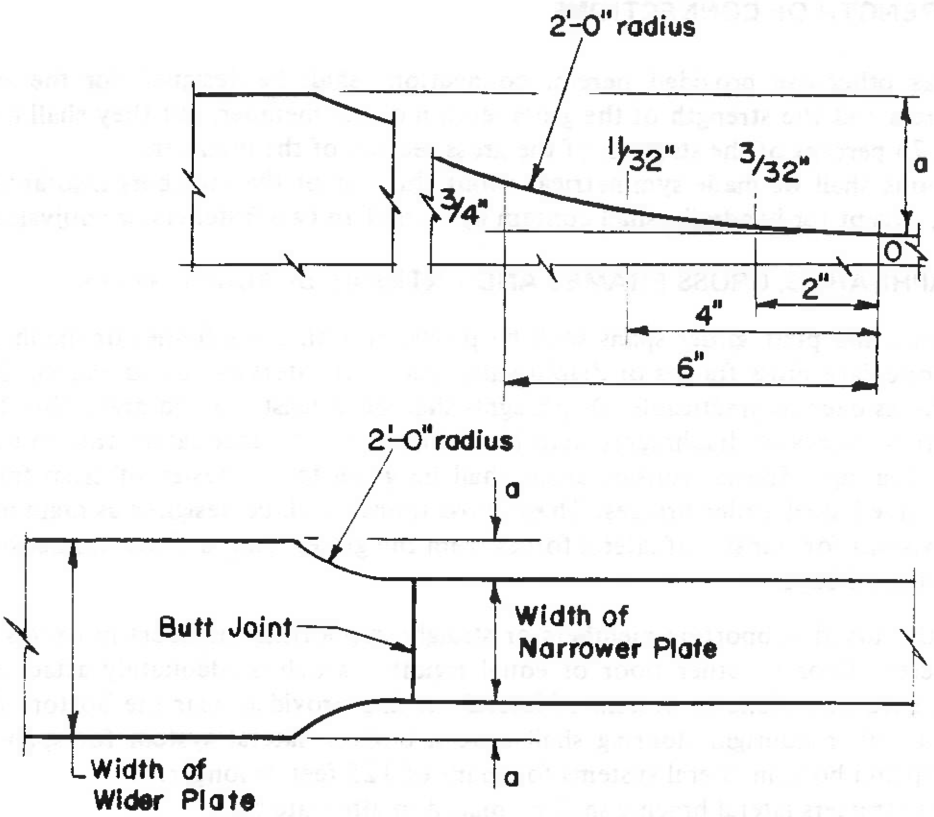
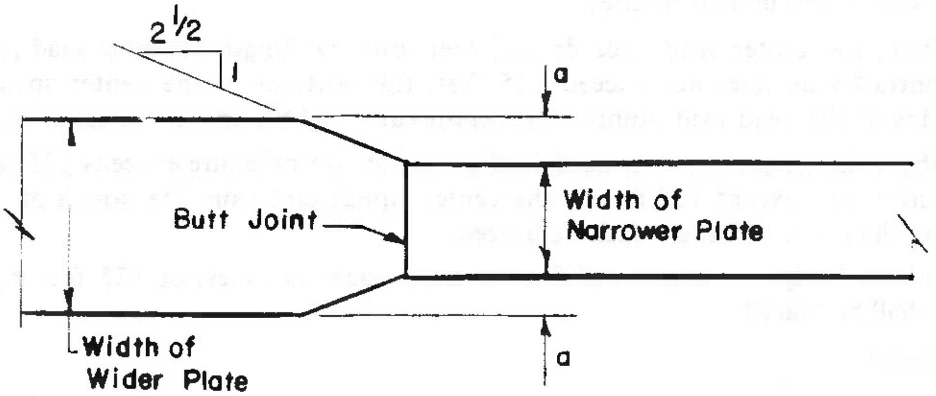
*For considerations to be taken into account when exceeding these limitations, reference is made to "Bulletin No. 19, Criteria for the Deflection of Steel Bridges," available from the American Iron and Steel Institute, Washington, D.C.

FIGURE 1.7.19



(a) 2'-0" Radius Transition



(b) Straight Tapered Transition

SPLICE DETAILS

1.7.20 – STRENGTH OF CONNECTIONS

Except as otherwise provided herein, connections shall be designed for the average of the calculated stress and the strength of the gross section of the member, but they shall be designed for not less than 75 percent of the strength of the gross section of the member.

Connections shall be made symmetrical about the axis of the members insofar as practicable. Connections, except for handrails, shall contain not less than two fasteners or equivalent weld.

1.7.21 – DIAPHRAHMS, CROSS FRAMES AND LATERAL BRACING *****

Rolled beam and plate girder spans shall be provided with cross frames or diaphragms at each end and intermediate cross frames or diaphragms spaced at intervals not to exceed 25 feet. Cross frames shall be as deep as practicable. Diaphragms shall be at least 1/3 and preferably 1/2 the girder depth. End cross frames or diaphragms shall be proportioned to adequately transmit all the lateral forces to the bearings. Special consideration shall be given to the design of cross frames used on horizontally curved steel girder bridges. These cross frames shall be designed as main members with adequate provisions for transfer of lateral forces from the girder flanges. Cross frames or diaphragms shall be placed in all bays.

Spans with curved supporting members or straight supporting members in excess of 125 feet, having a concrete floor or other floor of equal rigidity, which is adequately attached to the top flanges, shall have one plane or system of lateral bracing provided near the bottom flange. Spans with timber or other nonrigid flooring shall have a bottom lateral system for spans longer than 40 feet and top and bottom lateral systems for spans of 125 feet or longer.

For curved stringers lateral bracing shall be placed in alternate bays.

1. Simple spans in excess of 125 feet shall be braced for their entire length.
2. Two-span continuous structures, where either or both spans exceed 125 feet, shall be braced for their entire length.
3. Three-span continuous structures:
   a. Where the center span exceeds 125 feet, but the length between dead load points of contraflexure does not exceed 125 feet, the portions of the center span on the pier sides of the dead load points of contraflexure, and the entire end spans shall be braced.
   b. Where the length between dead load points of contraflexure exceeds 125 feet, all of the center span except 125 feet in the center equidistant from the points of contraflexure and the entire end spans shall be braced.
4. Continuous bridges consisting of four or more spans in excess of 125 feet: the entire end spans shall be braced.

   In addition:

   a. Where the length of an interior span between dead load points of contraflexure does not exceed 125 feet, the portions of the interior span between the dead load points of contraflexure and the near piers shall be braced.
   b. Where the length of an interior span between dead load points of contraflexure exceeds 125 feet, the entire interior span, except for 125 feet in the center equidistant from the points of contraflexure shall be braced.

In a transverse direction, bracing shall be provided as follows:

1. Up to and including six bays (seven stringers) – outside bays only.
2. Seven or more bays (eight or more stringers) – outside and first interior bays.

Where beams or girders comprise the main members of through spans, such members shall be stiffened against lateral deformation by means of gusset plates or knee braces with solid webs which shall be connected to the stiffeners on the main members and the floor beams. If the unsupported length of the edge of the gusset plate (or solid web) exceeds 60 times its thickness, the plate or web shall have a stiffening plate or angles connected along its unsupported edge.

Through truss spans, deck truss spans and spandrel braced arches shall have top and bottom lateral bracing.

Bracing shall be composed of angles, other shapes or welded sections.

If a double system of bracing is used, both systems may be considered effective simultaneously if the members meet the requirements both as tension and compression members. The members shall be connected at their intersections.

The lateral bracing of compression chords, preferably shall be as deep as the chords and effectively connected to both flanges.

The smallest angle used in bracing shall be 3 by 2½ inches. There shall be not less than 2 fasteners or equivalent weld in each end connection of the angles.

For bridges with skews up to and including 30 degrees, diaphragms or cross frames shall be placed in a continuous line parallel to the skew. For skews over 30 degrees, they shall be placed in a continuous line across the bridge at right angles to the stringers.

For secondary members, the edge of the gusset or connection plate shall be stiffened if the outstanding width of that portion of the plate outside the main member is equal to or greater than the following number of times its thickness.

58 for steel with 36,000 psi Y.P. min.
54 for steel with 42,000 psi Y.P. min.
51 for steel with 46,000 psi Y.P. min.
49 for steel with 50,000 psi Y.P. min.

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1.7.22 – NUMBER OF MAIN MEMBERS ON THROUGH SPANS

Where beams, girders or trusses are used for through spans, the spans preferably shall have only two main members. Such members shall be spaced a sufficient distance apart (center to center) to be secure against overturning by the assumed lateral forces.

1.7.23 – ACCESSIBILITY OF PARTS

The accessibility of all parts of a structure for inspection, cleaning and painting shall be secured by the proper proportioning of members and the design of their details.

1.7.24 – CLOSED SECTIONS AND POCKETS

Closed sections, and pockets or depressions which will retain water, shall be avoided where practicable. Pockets shall be provided with effective drain holes or be filled with waterproofing material.

Details shall be so arranged that the destructive effects of bird life, the retention of dirt, leaves, and other foreign matter will be reduced to a minimum. Where angles are used, either singly or in pairs, they preferably shall be placed with the vertical legs extending downward.

1.7.25 – WELDING, GENERAL*****

Steel base metal to be welded shall conform to the requirements of the AASHTO Standard Specifications for Welding of Structural Steel Highway Bridges, 1977, and subsequent AASHTO Interim Specifications Bridges.

Welding symbols shall conform with the latest edition of the American Welding Society Publication AWS A2.4

Fabrication shall conform to Article 2.10.23.

1.7.26 – MINIMUM SIZE OF FILLET WELDS

The minimum fillet weld size shall be as shown in the following Table except the minimum size fillet weld connecting parts carrying primary stress shall be 5/16 in. in lieu of 1/4 in. shown in the Table. Weld size is determined by the thicker of the two parts joined unless a larger size is required by calculated stress. The weld size need not exceed the thickness of the thinner part joined. Bearings, bracing connection stiffeners, diaphragms and lateral bracing shall have minimum 5/16 in. fillet welds.

Material Thickness of Thicker Part Joined (inches)	Minimum Size of Filler Weld (inches)
To 3/4 inclusive	1/4
Over 3/4 to 1 1/2	5/16
Over 1 1/2 to 2 1/4	3/8
Over 2 1/4 to 6	1/2
Over 6	5/8

The minimum size seal weld shall be 1/4 in. fillet weld.

1.7.27 – MAXIMUM EFFECTIVE SIZE OF FILLET WELDS

The maximum size of a fillet weld that may be assumed in the design of a connection shall be such that the stresses in the adjacent base material do not exceed the values allowed in Article 1.7.1. The maximum size that may be used along edges of connected parts shall be:

1. Along edges of material less than 1/4 inch thick, the maximum size may be equal to the thickness of the material.
2. Along edges of material 1/4 inch or more in thickness, the maximum size shall be 1/16 inch less than the thickness of the material, unless the weld is especially designated on the drawings to be built out to obtain full throat thickness.

1.7.28 – EFFECTIVE WELD AREAS

A. Butt Welds

The effective area shall be the effective weld length multiplied by the effective throat thickness.

1. The effective weld length for any butt weld, square or skewed, shall be the width of the part joined, perpendicular to the direction of stress.
2. The effective throat thickness shall be the thickness of the thinner piece of base metal joined. (No increase is permitted for weld reinforcement).

Added October 1977

CURVED HYBRID GIRDERS

1.7.168 – GENERAL

This section pertains to the design of hybrid I-girders which have a vertical axis of symmetry through the middle-plane of the web plate and which meet the general requirements of Article 1.7.110. The following additional requirements apply for curved hybrid girders.

1.7.169 – ALLOWABLE STRESSES

A. Bending – Noncomposite Girders

For noncomposite girders the bending stress in the web may exceed the allowable stress for the web steel provided that the stress in each flange does not exceed the allowable stress from Article 1.7.158 for the steel in that flange multiplied by the reduction factor

(a)

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

where:

F_y = minimum yield strength of the compression flange (psi)

β = the area of the web divided by the area of the tension flange

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

α' = $\alpha(1+|f_w/f_b|t)$

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

$|f_w/f_b|t$ = the absolute value of the tension flange tip stress due to lateral flange bending divided by the bending stress in the tension flange (c)

If $|f_w/f_b|t \geq \frac{1-\alpha}{\alpha}$, then R = 1.

B. Bending – Composite Girders

In the positive moment region of composite girders the bending stress in the web ma exceed the allowable stress for the web steel provided that the stress in the tension flange do not exceed the allowable stress from Article 1.7.158 for the steel in that flange multiplied b the reduction factor determined from Eq. (a) in Article 1.7.169(A) where α' is given by Eq. (b)

In the negative moment region of continuous span composite girders in which the tensio flange is connected to the concrete slab by shear connectors, the reduction factor shall be calculated using Eq. (a) in Article 1.7.169(A). where α' shall be the appropriate value given below:

when $$\left|\frac{f_w}{f_b}\right|_c \leq \frac{2\psi - 1}{1-\psi} \qquad \alpha' = \alpha \tag{d}$$

when $$\frac{2\psi-1}{1-\psi} < \left|\frac{f_w}{f_b}\right|_c < \frac{\psi}{\alpha(1-\psi)} - 1; \qquad \alpha' = \alpha\left[1 + \left|\frac{f_w}{f_b}\right|_c\right]\frac{1-\psi}{\psi} \tag{e}$$

where α and ψ are defined in Article 1.7.169(A) and

$\left|\frac{f_w}{f_b}\right|_c$ = the absolute value of the compression flange tip stress due to lateral flange bending divided by the bending stress in the compression flange

If $$\left|\frac{f_w}{f_b}\right|_c \geqslant \frac{\psi}{\alpha(1-\psi)} - 1, \qquad \text{then } R = 1. \tag{f}$$

C. Shear

The shear stress in the web shall not exceed the allowable shear stress for the web steel.

D. Fatigue

Design for fatigue shall be according to the requirements of Article 1.7.111(C).

1.7.170 – PLATE THICKNESS REQUIREMENTS

In calculating the minimum thickness of the web plate according to Article 1.7.160, f_b shall be taken as the calculated bending stress in the compression flange divided by the reduction factor. R. determined from the appropriate formula in Article 1.7.169. The limiting width to thickness ratio of the compression flange plate for hybrid girders shall be determined according to Article 1.7.158 using the yield stress of the compression flange material and need not be reduced due to the lower yield strength of the web material.

HEAT-CURVED ROLLED BEAMS AND WELDED PLATE GIRDERS

1.7.171 – SCOPE

This section pertains to rolled beams and welded I-section plate girders heat-curved to obtain a horizontal curvature. Steels that are manufactured to a specified minimum yield point greater than 50,000 psi, shall not be heat curved.

1.7.172 – MINIMUM RADIUS OF CURVATURE

For heat-curved beams and girders, the horizontal radius of curvature measured to the centerline of the girder web shall not be less than 150 feet, and shall not be less than the larger of the values calculated (at any and all cross sections throughout the length of the girder) from the following two equations:

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

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

In these equations, "F_y" is the specified min. yield point in ksi of steel in the girder web, "ψ" is the ratio of the total cross-sectional area to the cross-sectional area of both flanges, "b" is the widest flange width in inches, "D" is the clear distance between flanges in inches, "T" is the web thickness in inches, and "R" is the radius in inches.

In addition to the above requirements, the radius shall not be less than 1000 feet when the flange thickness exceeds 3 inches or the flange width exceeds 30 inches.

1.7.173 – CAMBER

To compensate for possible loss of camber of heat-curved girders in service as residual stresses dissipate, the amount of camber in inches. Δ, at any section along the length L of the girder shall be equal to:

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

where ΔDL is the camber in inches at any point along the length "L" calculated by usual procedures to compensate for deflection due to dead loads or any other specified loads. "Δm" is the maximum value of "ΔDL" in inches within the leng 'L". "E" is the modulus of elasticity in ksi, "F_y" is the specifi 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 non-symmetrical sections), and "L" is the span length for simple spans or the distance between a simple end support and the dead load contraflexure point; or the distance between points of dead load contraflexure for continuous spans. "L" is measured in inches.

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

Added September 1976

1.8.9 – LOAD FACTOR DESIGN

Load factor design is an alternate method of design for flexible culverts. It is a method of proportioning structural sections for multiples of the design load. The design shall be based on the following loading combination:

$$1.5 \times D + \beta_e E + \beta_l \times (L + I)$$

$$\beta_e = 1.67$$

$$\beta_l = 1.67$$

Where

D = Dead Load
E = Earth Load
L = Live Load
I = Impact
β_e = Long term effective density increase
β_l = Live load coefficient

(Safety factors used in service load design do not apply to load factor design)

COMMENTARY

The proposed changes to the 1973 AASHTO Specifications for Highway Bridges on culvert design are based on the extensive culvert research program by the California Department of Transportation. This research has progressed to the point that revisions to Section 1.2.22, and addition of 1.8.9 are proposed.

It is proposed to add this new article to specifically permit Load Factor Design as an alternate method of design for flexible culverts. TRB publication 510, Soil Mechanics, inlcudes an article which outlines the basic background data for application of an ultimate design concept to flexible culverts. It is based on two flexible culvert research projects recently completed by California.

The factor of safety presently used for longitudinal test seam strength is 4.0. Application of this factor would result in allowable fill heights that would be inconsistent with the observed satisfactory performance of the research projects despite signigicant yielding of the steel. A 1.5 factor is recommended. The increase in peripheral pressures, subsequent to fill completion, was approximately 70 percent. Based on this, a β_e factor of 1.67 is proposed.

Summary

It is recognized that there may be some further qualification of these culvert design criteria when CALTRANS research on a 96 in. or (2.438 m) prestressed concrete pipe, and 84 in. or (2.134 m) concrete pipe, and a 120 in. or (3.048 m) structural steel plate pipe (Rte 210) is completed.

1.8.10 – LONG SPAN STRUCTURAL PLATE STRUCTURES

A. General

Long-Span Structural Plate Structures are defined as:

1. Structural plate structures which exceed the maximum sizes imposed by Article 1.8.2(B).
2. Special shapes of any size which involve a relatively large radius of curvature in crown or side plates. Horizontal ellipses, low profile arches, high profile arches, and inverted pear shapes are the terms describing the special shapes. See Fig. 1.8.10.

B. Designs

1. Long Span Structures shall be designated in accordance with Article 1.8.2 (A). Substitute twice the top arc radius for the Span in the formula.
2. In lieu of meeting requirements of Articles 1.8.2(B) and (C), Long Span Structures may include acceptable special features and meet the geometric and sectional requirements of Table 1.8.3.
3. Acceptable Special Features
   - (a) Continuous longitudinal structural stiffeners connected to the corrugated plates at each side of the top arc.
   - (b) Pre-loaded soil bins on the top of the structure, formed by continuous longitudinal corrugated plates fastened to the structure near the ends of the top arc and connected together across the top arc with flat plate dividers on 6 ft. (1.829m) centers.
   - (c) Reinforcing ribs formed from structural shapes curved to conform to the curvature of the plates, fastened to the structure as required to insure integral action with the corrugated plates, and spaced at such intervals as necessary to increase the moment of inertia of the section to that required by the design.
4. Structures without acceptable special features must comply with Articles 1.8.2 (B) and (C). Substitute twice the top arc radius for the diameter in the formulae.
5. Design for Deflection

   Soil design and placement requirements for long-span structures limit deflection satisfactorily. Therefore. Section 1.8.2 (D) is not applicable. However, construction procedures must be such that severe deflections do not occur during construction.
6. Soil Design

   Granular type soils shall be used as structure backfill (the envelope next to the metal structure). The order of preference of acceptable structure backfill materials is as follows:
   1. Well graded sand and gravel; sharp, rough or angular is possible.
   2. Uniform sand or gravel.
   3. Approved stabilized soil shall be used only under direct supervision of a competent, experienced soils engineer. Plastic soils shall not be used.

The structure backfill material shall conform to one of the following soil classifications from AASHTO Specification M145, Table 2:

For height of fill less than 12 feet (3.658m), A-1, A-3, A-2-4 and A-2-5; For height of fill of 12 feet (3.658m) and more, A-1, A-3. Structure backfill shall be placed and compacted to not less than 90 percent density per AASHTO T180.

The extent of the soil envelope about the barrel shall be great enough to prevent shearing planes from developing in the envelope. To each side of the barrel, the soil envelope or sidefill shall extend a minimum horizontal distance of half the span or two-thirds of the total rise, whichever is greater. It is not necessary to excavate native soil at the sides if the quality of the native soil is already as good as the proposed compacted sidefill. The soil over the top shall also be select and shall be carefully and densely compacted.

C. End Treatment

When headwalls are not used, special attention may be necessary at the ends of the structure. Severe bevels and skews are not recommended. For hydraulic structures, additional reinforcement of the end is recommended to secure the metal edges at inlet and outlet against hydraulic forces. Reinforced concrete or structural steel collars, or tension tiebacks to anchors in soil, partial headwalls and cut off walls below invert elevation are some of the methods which can be used. Square ends may have side plates beveled to a mixumum 2:1 slope. Skew ends up to 15° with no bevel, are permissable. When this is done on spans over 20 feet (6.096m) the cut edge must be reinforced with reinforced concrete or structural steel collar. When full headwalls are used and they are skewed, the offset portion of the metal structure shall be supported by the headwall. A special headwall shall be designed for skews exceeding 15°. A maximum skew shall be limited to 35°.

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