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ENGINEERING INSTRUCTION

NEW YORK STATE DEPARTMENT OF TRANSPORTATION

SUBJECT: STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, REVISION TO

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The attached pages are revisions to the Standard Specifications for Highway Bridges Manual dated April 1976. These pages include corrections, revisions and 1976 AASHTO Interim Specifications.

Page		
13	No change	
14	Art. 1.2.5(G)	Added new sub-article
27	Art. 1.2.22	Fifth paragraph - revised
28	Art. 1.2.22	Moved text to next page Added Group X to table Revised Working Stress Design to Service Load Design Added footnote
29	Art. 1.2.22	Revised text
30	Art. 1.2.22	Added new page - Commentary
41	No change	
42	Art. 1.3.4(A)	Added Limitations to "S" in formulas
43	Art. 1.3.4(A)	Corrected formula at top of page
44	No change	
47	Art. 1.4.1	Revised
48	No change	
49	Art. 1.4.4(B-1)	Revised first paragraph
50	Art. 1.4.4(B-1)	Revised first paragraph
	Art. 1.4.4(C-1)	Revised (b) and (c) Added asterisk (*) to (d) Added (e)
51	Page rewritten	No change in text
52	Page rewritten	No change in text
53	Page rewritten	No change in text
54	No change	
57	Art. 1.4.5(L)	Added new sub-article
57-1	Art. 1.4.6	Added new page - no change in text
57-2	Added new blank page	
58	No change	
65	No change	
66	Art. 1.5.2	Correction to $\sqrt{f'_c}$
85	Art. 1.5.23(D)	Corrected formula
86	No change	
91	No change	

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Page		
92	Art. 1.5.26(B)	Revised Deleted footnote
101	No change	
102	Art. 1.5.33(B)	Corrected formula - A_s to A_g
103	Art. 1.5.33(B)	Rewrote last formula for clarification
104	Art. 1.5.33(C)	Corrected formula - A_s to A_g
107	Art. 1.5.35(B)	Corrected formula (6-21)
108	Art. 1.5.35(C)	Corrected formula (6-23) Corrected all formulas - changed large "V" to small "v"
111	Art. 1.5.35(F)	Rewrote formula at top of page to indicate small "v"
112	Art. 1.5.38(B)	Revised
112-1	Art. 1.5.39	Added new page - no change in text
112-2	Art. 1.5.40	Revised page number - no change in text
119	Art. 1.6.7	Rewrote formula for clarification
120	No change	
121	No change	
122	Art. 1.6.7(B)	Corrected third formula
139	Art. 1.6.11	Corrected both formulas
140	No change	
159	Art. 1.7.1	Reversed headings under A 441
160	Art. 1.7.1	Corrected π^3 to π^2 in first formula for F_a near bottom of page
163	Art. 1.7.3	Revised title of article and text
164	Art. 1.7.3	Continuation of revisions
165	Art. 1.7.3	Revised Fig. No. in "Built-up section
166	Art. 1.7.3	Added last "Situation" under "Groove Welds"
166-1	Art. 1.7.3	Added new page for continuation of article
166-2	Art. 1.7.3	Added new page for continuation of article
166-3	Art. 1.7.3	Added new page for continuation of article
166-4	Added new blank page	
167	Art. 1.7.3	Revised Sketch No. 4
168	No change	
185	Art. 1.7.38(B)	Deleted first paragraph
186	No change	
195	No change	
196	Art. 1.7.71	Corrected formulas for I and J
219	Art. 1.7.105(D)	Corrected formula "w/t = "
220	No change	
229	No change	
230	Art. 1.7.117	Added last sentence
233	Art. 1.7.124(A)	Revised M_2 limitations in formula
234	No change	
237	No change	
238	Art. 1.7.124(F)	Added limitation to formula "D/t_w "
241	No change	
242	Art. 1.7.127	Moved top three lines from page 243 to bottom of page

Page		
243	Art. 1.7.127(B)	Revised Added Commentary
244	No change	
245	Art. 1.7.129(E)	Corrected value of "b/t" in first line
246	No change	
251	No change	
252	Art. 1.7.135(A)	Revised formula
261	Art. 1.8.1	Revised material specifications
262	No change	
265	No change	
266	Art. 1.8.3(A)	Revised chemical specifications
	Art. 1.8.3(B)	Revised minimum yield
	Art. 1.8.3(C)	Revised chemical specifications
	Revised text at bottom of page and moved to page 267.	
267	Retyped due to change in page 266	
268	Art. 1.8.8	Revised
268-1	Art. 1.8.9	Added new Article - new page
268-2	Added new blank page	
269	Art. 1.9.6	Revised
270	Added new blank page	
275	Art. 1.10.2(D)	Revised "ℓ/d" to "ℓ/r" in defining F_c'
276	No change	

<u>REVISIONS TO TABLE OF CONTENTS</u>

I	No change.
II	Corrected to agree with revisions.
III	No change.
IV	Corrected to agree with revisions.
VII	Revised to agree with revisions.
VIII	No change.
IX	Revised to agree with revisions.
X	No change.
XV	Revised to agree with revisions.
XVI	No change.

1.2.4. – OVERLOAD PROVISION*

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

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

1.2.5 – HIGHWAY LOADINGS

A. General

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

B. H. Loadings

The H loadings are illustrated in Figures 1.2.5A and 1.2.5B. They consist of a two-axle truck or the corresponding lane loading. The H loadings are designated H followed by a number indicating the gross weight in tons of the standard truck.

C. HS Loadings

The HS loadings are illustrated in Figures 1.2.5B and 1.2.5C. They consist of a tractor truck with semi-trailer or of the corresponding lane loading. The HS loadings are designated by the letters HS followed by a number indicating the gross weight in tons of the tractor truck. The variable axle spacing has been introduced in order that the spacing of axles may approximate more closely the tractor trailers now in use. The variable spacing also provides a more satisfactory loading for continuous spans, in that heavy axle loads may be so placed on adjoining spans as to produce maximum negative moment.

D. Classes of Loading

Highway loadings shall be of five classes: H 20, H 15, H 10, HS 20 and HS 15. Loadings H 15 and H 10 are 75 per cent and 50 per cent, respectively, of loading H 20. Loading HS 15 is 75 per cent of loading HS 20. If loading of weights other than those designated are desired, they shall be obtained by proportionately changing the weight shown for both the standard truck and the corresponding lane loads.

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

E. Designation of Loadings

The policy of affixing the year to loadings to identify them was instituted with the publication of the 1944 edition in the following manner:

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

The affix remains unchanged until such time as the loading specification is revised. The same policy for identification shall be applied, for future reference, to loading previously adopted by the American Association of State Highway Officials.

F. Minimum Loading*****

In general HS20 live load shall be used for the design of structures on all highways.

In addition for structures on the Main line of trunk highways, the national system of interstate highways and other designated expressways a special loading of two 24,000 pound axles spaced 4 feet on centers shall be used if it produces a greater stress than the HS20 live load. This special loading shall be applied in accordance with the provisions of Article 1.2.8.

G. Interstate Highway Bridge Loadings

Bridges supporting Interstate highways shall be designed for HS 20-44 Loading or an Alternate Military Loading of two axles four feet apart with each axle weighing 24,000 pounds or (10,886 kg), whichever produces the greatest stress.

1.2.22–LOADING COMBINATIONS

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

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

where

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

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

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

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

When long span structures are being designed by load factor design, the gamma and beta factors specified for Load Factor Design represent general conditions and should be increased if in the Engineer's judgment, anticipated loads, service conditions or materials of construction are different than anticipated by the specifications.

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

Revised September 1976

TABLE 1.2.22

Table of Coefficients γ and β

	Column No.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
			β FACTORS													
	Group	γ	D	L+I	CF	E	B	SF	W	WL	LF	F	R+S+T	EQ	ICE	%
SERVICE LOAD DESIGN	I	1.0	1	1	1	β_E	1	1	0	0	0	0	0	0	0	100
	II	1.0	1	0	0	1	1	1	1	0	0	0	0	0	0	125
	III	1.0	1	1	1	β_E	1	1	0.3	1	1	1	0	0	0	125
	IV	1.0	1	1	1	β_E	1	1	0	0	0	0	1	0	0	125
	V	1.0	1	0	0	1	1	1	1	0	0	0	1	0	0	140
	VI	1.0	1	1	1	β_E	1	1	0.3	1	1	1	1	0	0	140
	VII	1.0	1	0	0	1	1	1	0	0	0	0	0	1	0	133
	VIII	1.0	1	1	1	1	1	1	0	0	0	0	0	0	1	140
	IX	1.0	1	0	0	1	1	1	1	0	0	0	0	0	1	150
	X*	1.0	1	1	0	β_E	0	0	0	0	0	0	0	0	0	100
LOAD FACTOR DESIGN	I	1.3	β_D	1.67	1.0	β_E	1	1	0	0	0	0	0	0	0	Not applicable
	IA	1.3		2.20	0	0	0	0	0	0	0	0	0	0	0	
	II	1.3		0	0	β_E	1	1	1	0	0	0	0	0	0	
	III	1.3		1	1		1	1	0.3	1	1	1	0	0	0	
	IV	1.3		1	1		1	1	0	0	0	0	1	0	0	
	V	1.25		0	0		1	1	1	0	0	0	1	0	0	
	VI	1.25		1	1		1	1	0.3	1	1	1	1	0	0	
	VII	1.3		0	0		1	1	0	0	0	0	0	1	0	
	VIII	1.3		1	1		1	1	0	0	0	0	0	0	1	
	IX	1.20		0	0		1	1	1	0	0	0	0	0	1	
	X*	1.50	1	1.67	0	β_E	0	0	0	0	0	0	0	0	0	0

*Group X applies only to culverts.

Notes for Table 1.2.22

For Service Load Design

% (Column 15) Percentage of Basic Unit Stress

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

β_E = 0.70 for Reinforced Concrete Boxes; 0.83 for all other culverts. See culvert loading specifications Article 1.2.2(A).

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

For Load Factor Design

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

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

β_E = 1.0 for vertical earth pressure

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

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

for Column Design

β_D = 1.0 for flexural and tension members

Added September 1976

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.

1.2.22

Group X provides service load and load factor alternatives to culvert design.

The service load values for β_e are consistent with the values currently specified in AASHTO.

The load factor design for flexible culverts is included in this section, and is outlined in greater detail in Article 1.8.9. This duplication is justified since it is our intent to focus attention on this basic change. The discussion by the AASHTO Task Force on Flexible Culverts confirmed the advisability of this approach.

The load factor design for rigid culverts can be implemented even though earth structures are subject to more uncertainties in installation, loading, and deformation characteristics than are bridges. The service load analysis applies a safety factor of 2.5 to the analysis of rigid culverts. Our research demonstrates that this factor can be reduced. For example an underdesigned, dummy RCP was able to withstand approximately twice the maximum fill height shown in our design tables despite the fact that the observed soil density was higher than that assumed for design.

1.3.4 – DISTRICUTION OF WHEEL LOADS ON TIMBER FLOORING

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

A. Flooring Transverse

In direction of span:

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

Normal to direction of span:

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

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

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

Thus

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

$$R_x = .034P$$

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

or

$$t = \frac{3R_x}{2F_v}$$ whichever is greater

where

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

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

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

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

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

$$n = \frac{1000}{\sigma PL}\left(\frac{\overline{R_y}}{R_D} + \frac{\overline{M_y}}{M_D}\right)$$

where

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

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

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

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

or

$$\overline{R_y} = \frac{P}{25}(s - 20) \quad \text{for } s > 50 \text{ in.}$$

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

$$\overline{M_y} = \frac{Ps}{1600}(s - 10) \quad \text{for } s \leq 50 \text{ in.}$$

or

$$\overline{M_y} = \frac{Ps}{20}\left(\frac{s-30}{s-10}\right) \quad \text{for } s > 50 \text{ in.}$$

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

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

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

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

where

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

B. Flooring Longitudinal

In direction of span:
Point Loading

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

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

C. Continuous Flooring

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

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

A. Distribution of Concentrated Loads for Bending Moment and Shear

For freely supported or continuous slab spans of composite wood-concrete construction the wheel loads shall be distributed over a transverse width of 5 feet for bending moment and a width of 4 feet for shear.

For composite T-beams of wood and concrete the effective flange width shall not exceed that given in Article 1.7.99. Shear connectors shall be capable of resisting both vertical and horizontal movement.

B. Distribution of Bending Moments in Continuous Spans

Both positive and negative moments shall be distributed in accordance with the following Table:

Maximum bending moments - per cent of simple span moment

Span	Maximum Uniform Dead Load Moments				Maximum Live Load Moments			
	Wood Subdeck		Composite Slab		Concentrated Load		Uniform Load	
	Pos.	Neg.	Pos.	Neg.	Pos.	Neg.	Pos.	Neg.
Interior	50	50	55	45	75	25	75	55
End	70	60	70	60	85	30	85	65
2-span*	65	70	60	75	85	30	80	75

Impact should be considered in computing stresses for concrete and steel, but neglected for wood.

C. Design

The combination in a structural member of two elements having different mechanical properties requires the formulation of a design premise. Such a formulation as follows is based on the elastic properties of the materials:

EC/EW = 1 for slab in which the net concrete thickness is less than half the overall depth of the composite section

EC/EW = 2 for slab in which the net concrete thickness is a least half the overall depth of the composite section

ES/EW = 18.75 (for Douglas Fir and Southern Pine)

in which

EC = modulus of elasticity of concrete

EW = modulus of elasticity of wood

ES = modulus of elasticity of steel

*Continuous beam of 2 equal spans.

Section 4 – SUBSTRUCTURES AND RETAINING WALLS

1.4.1 – ALLOWABLE STRESSES

Concrete, prestressed concrete, steel or timber substructures and retaining walls shall be designed for the unit stresses indicated in Section 5, Section 6, Section 7, or Section 10.

1.4.2 – BEARING POWER OF FOUNDATION SOILS DETERMINATION OF BEARING POWER

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

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

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

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

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

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

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

1.4.3 – ANGLES OF REPOSE

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

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

1.4.4 – BEARING VALUE OF PILING

A. General

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

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

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

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

1. Structural Columns

Piles shall be designed as structural columns. Timber piles shall be designed in accordance with Article 1.10.2, using the allowable unit stresses given in Article 1.10.1 for lumber and in the following table for Round Timber 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 Section 1.5, prestressed concrete piles in accordance with Section 1.6, steel piles in accordance with Section 1.7, and concrete-filled pipe piles in accordance with Section 1.5, except that the allowable unit stresses may be increased 20 percent provided the shell thickness is not less than 1/4 inch or (.006 m). The area of shell shall be included in determining the value of p, (percentage of reinforcement). Where corrosion may be expected, 1/16 inch (1.6 mm) shall be deducted from the shell thickness to allow for reduction in section by corrosion. The allowable stresses of Section 1.5, 1.6, 1.7, and 1.10 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 stresses specified in Sections 1.5, 1.6, 1.7, and 1.10 shall be decreased 20 percent.

2. Required Subsurface Investigations

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

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

1. Point-Bearing Piles

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

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

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

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

(b) For concrete piles, $0.33f'_c$ times the gross cross sectional area of the concrete.

*(c) For concrete-filled piles, $0.40f'_c$ times the total actual area of the concrete and steel.

*(d) For steel H-piles and unfilled tubular steel piles, 9,000 psi (62.053 MPa) over the cross sectional area of the pile tip, not including the area of any pile tip reinforcement.

(e) For prestressed concrete piles fully embedded in soils providing lateral support, a stress of $0.33f'_c-0.27\ f_{ce}$ times the gross cross sectional area of the concrete where f_{ce} is the concrete stress in the pile due to prestressing, after all losses.

*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.

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

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

3. Required Subsurface Investigations

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

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

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

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

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

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

1. Point-Bearing Piles

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

2. Friction Piles

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

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

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

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

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

F. Uplift*****

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

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

G. Group Pile Loading

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

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

where

E = the efficiency or the decimal fraction of the single pile value to be used for each pile in the group.

n = the number of piles in each row.

m = the number of rows in each group.

d = the average diameter of the pile.

s = center to center spacing of piles.

Tan ϕ = d/s

ϕ is numerically equal to the angle expressed in degrees.

..on Piles

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

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

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

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

where

E = the efficiency or the decimal fraction of the single pile value to be used for each pile in the group.

n = the number of piles in each row.

m = the number of rows in each group.

d = the average diameter of the pile.

s = center to center spacing of piles.

Tan ϕ = d/s

ϕ is numerically equal to the angle expressed in degrees.

1.4.5 – PILES

A. General

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

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

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

B. Limitation of Use

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

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

C. Design Loads

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

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

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

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

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

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

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

E. Batter Piles*****

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

The maximum batter shall be 1 horizontal to 3 vertical.

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

F. Buoyancy

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

G. Concrete Piles (Precast)

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

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

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

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

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

4. Column Action

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

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

K. Steel Pile and Steel Pile Shell Protection

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

L. Prestressed Concrete Piles

Prestressed concrete piles, which are generally octagonal, square, or circular shall be of approved size and shape. Air entrained concrete shall be used in piles which are subject to freezing and thawing or wetting and drying. Concrete in prestressed piles shall have a minimum compressive strength (f'_c) of 5000 p.s.i. or (34.474 MPa) at 28 days.

In general, prestressed concrete piles may be solid or hollow. For hollow piles precautionary measures should be taken to prevent breakage due to internal water pressure during driving, ice pressure in trestle piles and gas pressure due to decomposition of material used to form the void.

Main reinforcement shall be spaced and stressed so as to provide a compressive stress on the pile after losses f_{ce}, generally not less than 700 p.s.i. or (4.826 MPa), to prevent cracking during handling and installation. Piles shall be designed to resist stresses developed during handling and installation. Piles shall be designed to resist stresses developed during handling as well as service load conditions. Bending stresses shall be investigated for all conditions of handling, taking into account the weight of the pile plus 50 percent allowance for impact, with tensile stresses limited to $5\sqrt{f'_c}$ or ($.415\sqrt{f'_c}$ in MPa).

The full length of vertical reinforcement shall be enclosed with spiral reinforcement. For piles up to 24 inches or (0.610 m) in diameter, spiral wire shall be No. 5 (U.S. Steel Wire Gage). Spiral reinforcement at the ends of these piles shall have a pitch of 3 inches or (0.076 m) for approximately sixteen turns. In addition, the top 6 inches or (0.152 m) of pile shall have five turns of spiral winding at 1-inch or (0.025 m) pitch. For the remainder of the pile, the vertical steel shall be enclosed with spiral reinforcement with not more than 6-inch pitch. For piles having a diameter greater than 24 inches or (0.610 m), wire shall be No. 4 (U.S. Steel Wire Gage). Spiral reinforcement at the end of these piles shall have a pitch of 2 inches or (0.051 m) for approximately sixteen turns. In addition, the top 6 inches or (0.152 m) shall have 4 turns of spiral winding at 1½ inches or (0.038 m). For the remainder of the pile, the vertical steel shall be enclosed with spiral reinforcement with not more than 4-inch or (0.102 m) pitch. The reinforcement shall be placed at a clear distance from the face of the prestressed pile of not less than 2 inches or (0.051 m).

Large diameter hollow cylinder piles shall be of approved size and shape. The wall thickness for cylinder piles shall not be less than 5 inches or (0.127 m). The grouting of post-tensioning tendons shall be in accordance with Article 2.4.33(I).

1.4.6 – FOOTINGS

A. Depth*****

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

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

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

B. Anchorage*****

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

C. Distribution of Pressure

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

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D. Spread Footings

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

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

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

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

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

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

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

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

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

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

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

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

Section 5 – REINFORCED CONCRETE DESIGN

1.5.1 – APPLICATION

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

1.5.2 – NOTATIONS AND DEFINITIONS

A. Notations

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

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

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

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

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

A_g = gross area of section, sq. in.

A_s = area of tension reinforcement, sq. in.

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

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

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

A_v = area of shear reinforcement within a distance s.

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

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

A_1 = loaded area–Articles 1.5.26 and 1.5.36.

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

b = width of compression face of member.

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

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

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

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

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

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

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

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

d_b = nominal diameter of bar or wire, in.

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

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

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

= M_b/P_b.

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

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

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

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

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

f_c' = specified compressive strength of concrete, psi.

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

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

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

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

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

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

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

f_y = specified yield strength of reinforcement, psi.

h = overall thickness of member, in.

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

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

ANALYSIS AND DESIGN–GENERAL CONSIDERATIONS

1.5.23 – ANALYSIS METHODS

A. General

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

B. Expansion and Contraction

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

C. Stiffness

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

D. Modulus of Elasticity

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

E. Thermal and Shrinkage Coefficients

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

F. Span Length

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

G. Computation of Deflections

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

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

where

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

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

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

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

SERVICE LOAD DESIGN

1.5.25 – GENERAL REQUIREMENTS

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

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

1.5.26 – ALLOWABLE SERVICE LOAD STRESSES

A. Concrete

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

1. Flexure

   Extreme fiber stress in compression, f_c . . . . $0.40f'_c$

   Extreme fiber stress in tension for plain concrete, f_t . . . . $0.21 f_r$

   Modulus of rupture, f_r from tests, or if data are not available:

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

   "Sand-lightweight" concrete . . . . $6.3\sqrt{f'_c}$

   "All-lightweight" concrete . . . . $5.5\sqrt{f'_c}$

2. Shear*

   Beams: Shear carried by concrete, v_c . . . . $0.95\sqrt{f'_c}$

   Maximum shear carried by concrete plus shear reinforcement, v . . . . $v_c + 4\sqrt{f'_c}$

   Slabs and Footings (peripheral shear):**

   Shear carried by concrete, v_c . . . . $1.8\sqrt{f'_c}$

   Maximum shear carried by concrete plus shear reinforcement, v . . . . $3\sqrt{f'_c}$

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

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

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

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

**See Article 1.5.26(F).

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

B. Reinforcement

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

Grade 40 or Grade 50 reinforcement. . . . . 20,000 psi (137.895 MPa)
Grade 60 reinforcement . . . . 24,000 psi (165.494 MPa)

Fatigue Stress Limit – The range between a maximum tension stress and minimum stress in straight reinforcement caused by live load plus impact shall not exceed the value given in Article 1.5.38(B). Bends in primary reinforcement shall be avoided in regions of high stress.

1.5.27 – FLEXURE

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

a. A section plane before bending remains plane after bending; strains vary as the distance from the neutral axis.
b. The stress-strain relation of concrete is a straight line under service loads within the allowable service load stresses. Stresses vary as the distance from the neutral axis except, for deep flexural members with overall depth span ratios greater than 2/5 for continuous spans and 4/5 for simple spans, a nonlinear distribution of stress should be considered.
c. The steel takes all the tension due to flexure.
d. The modular ratio, $n = E_s/E_c$, may be taken as the nearest whole number (but not less than 6). Except in calculations for deflections, the value of "n" for lightweight concrete shall be assumed to be the same as for normal weight concrete of the same strength.
e. In doubly reinforced flexural members, an effective modular ratio of $2E_s/E_c$ shall be used to transform the compression reinforcement for stress computations. The compressive stress in such reinforcement shall not be greater than the allowable tensile stress.

1.5.28 – COMPRESSION MEMBERS WITH OR WITHOUT FLEXURE

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

D. Rectangular Sections With Compression Reinforcement

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

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

where

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

and the following condition shall be satisfied:

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

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, " b," for rectangular sections with compression reinforcement is given by:

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

where

f'_s = stress in compression reinforcement

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

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 Article 1.5.31. The requirements of Article 1.5.32(A) shall also be satisfied.

1.5.33 – COMPRESSION MEMBERS WITH OR WITHOUT FLEXURE

A. General Requirements

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

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

   a. that corresponding to the maximum design moment which accompanies this compression load, or

   b. 0.05h for spirally reinforced compression members, or "0.10h" for tied compression members, about either axis, or

   c. 1 in. about either axis.

B. Compression Member Strengths

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

1. Pure Compression

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

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

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

2. Pure Flexure

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

3. Balanced Conditions

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

$$P_b = \phi \left[0.85 f'_c b a_b + A'_s f'_s - A_s f_y\right] \tag{6-10}$$

and

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

where

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

and

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

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", decreased from "$0.10 f'_c A_g$ or "P_b", whichever is smaller, to zero.

C. Biaxial Loading

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

$$P_{uxy} = \frac{1}{\left(\frac{1}{P_{ux}}\right) + \left(\frac{1}{P_{uy}}\right) + \left(\frac{1}{P_o}\right)} \tag{6-12}$$

when the applied axial design load,

$$P_u \geqq 0.1 f'_c A_g$$

or

$$\frac{M_x}{M_{ux}} + \frac{M_y}{M_{uy}} \leqq 1 \tag{6-13}$$

when the applied axial design load,

$$P_u < 0.1 f'_c A_g$$

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, "lu", 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.

B. Permissible Shear Stress

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

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

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

but "v_c" shall not exceed "$3.5\sqrt{f'_c}$". The quantity "$V_u d/M_u$" shall not be taken greater than 1.0, where "M_u" is the applied design moment occurring simultaneously with "V_u" at the section considered.

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

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

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

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

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

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

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

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

   but the value of

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

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

C. Design of Shear Reinforcement

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

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

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

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

$$A_v = \frac{(v_u - v_c)\, b_w\, s}{f_y\,(\sin c + \cos a)} \tag{6-25}$$

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

$$A_v = \frac{(v_u - v_c)\, b_w\, d}{f_y\, \sin a} \tag{6-26}$$

in which $(v_u - v_c)$ shall not exceed $3\sqrt{f'_c}$.

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

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

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

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

5. The value of $(v_u - v_c)$ shall not exceed $8\sqrt{f'_c}$.

2. The peripheral shear stress shall be computed by:

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

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, $v_c = 4\sqrt{f'_c}$ unless shear reinforcement is provided in accordance with (3), in which case "v_u" shall not exceed $6\sqrt{f'_c}$.

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

1.5.36 – PERMISSIBLE BEARING STRESS

A. Bearing stress in concrete on loaded area, "f_b", shall not exceed "$0.85\phi f'_c$, except as provided below.

B. When the supporting surface is wider on all sides than the loaded area, the permissible bearing stress on the loaded area may be multiplied by

$$\sqrt{\frac{A_2}{A_1}} \quad \text{but not more than 2.}$$

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

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

1.5.37 – SERVICEABILITY REQUIREMENTS

A. Application

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

B. Service Load Stresses

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

1.5.38 – FATIGUE STRESS LIMITS

A. Concrete

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

B. Reinforcement

The range between a maximum tension stress and minimum stress in straight reinforcement caused by live load plus impact at service load shall not exceed:

$$f_r = 21 - 0.33 f_{min.} + \frac{8r}{h}$$

or

$$f_r = \left(144.790 - 0.33 f_{min.} + 55.12 \left(\frac{r}{h}\right)\right) \text{ in SI}$$

where:

f_r = stress range, ksi or (MPa)

$f_{min.}$ = algebraic minimum stress level, tension positive, compression negative, ksi or (MPa)

$\frac{r}{h}$ = ratio of base radius to height of rolled on transverse deformation; when actual value is not known, use 0.3

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, "fy", for tension reinforcement exceeds 40,000 psi, cross sections of maximum positive and negative moment shall be so proportioned that the calculated stress in the reinforcement at service load, "f_s", in kips per sq. in., does not exceed the value computed by:

$$f_s = \frac{z}{\sqrt[3]{d_c A}} \qquad (6\text{-}30)$$

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

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

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

The quantity "z" in Eq. (6-30) shall not exceed 170 kips per in. for members in moderate exposure conditions and 130 kips per in. for members in severe exposure conditions. Where members are exposed to very aggressive exposure or corrosive environments, such as deicer chemicals, the denseness and nonporosity of the protecting concrete should be considered, or other protection, such as a waterproof protecting system, should be provided in addition to satisfying Eq. (6-30).

1.5.40 – CONTROL OF DEFLECTIONS

A. General

Flexural members of bridge structures shall be designed to have adequate stiffness to limit deflections or any deformations which may adversely affect the strength or serviceability of the structure at service load.

B. Superstructure Depth Limitations

The minimum thicknesses stipulated in Table 1.5.40 are recommended unless computation of deflection indicates that lesses thickness may be used without adverse effects.

TABLE 1.5.40 – Recommended Minimum Thickness for Constant Depth Members*

Superstructure Type	Minimum Thickness** (in feet)
Bridge slabs with main reinforcement parallel or perpendicular to traffic	$\frac{S+10}{30}$ but less than 0.542
T-Girders	$\frac{S+9}{18}$
Box-Girders	$\frac{S+10}{20}$

*When variable depth members are used, table values may be adjusted to account for change in relative stiffness of positive and negative moment sections.
**Recommended values for continuous spans; simple spans should have about 10 percent greater thickness.
S = span length as defined in Article 1.5.23(F), in feet.

1.6.7 – LOSS OF PRESTRESS

A. Friction Losses

Friction losses in post-tensioned steel shall be based on experimentally determined wobble and curvature coefficients, and shall be verified during stressing operations. The values of coefficients assumed for design, and the acceptable ranges of jacking forces and steel elongation shall be shown on the plans. These friction losses shall be calculated as follows:

$$T_o = T_x e^{(KL + \mu a)}$$

When $(KL + \mu\alpha)$ is not greater than 0.3, the following equation may be used:

$$T_o = T_x(1 + KL + \mu a)$$

The following values for K and μ may be used when experimental data for the materials used are not available:

Type of Steel	Type of Duct	K	μ
Wire or Ungalvanized Strand	Bright Metal Sheathing	0.0020	0.30
	Galvanized Metal Sheathing	0.0015	0.25
	Greased or Asphalt-Coated and Wrapped	0.0020	0.30
	Galvanized Rigid	0.0002	0.25
High-Strength Bars	Bright Metal Sheathing	0.0003	0.20
	Galvanized Metal Sheathing	0.0002	0.15

Friction losses occur prior to anchoring but should be estimated for design and checked during stressing operations. Rigid ducts shall have sufficient strength to maintain their correct alignment without visible wobble during placement of concrete. Rigid ducts may be fabricated with either welded or interlocked seams. Galvanizing of the welded seam will not be required.

B. Prestress Losses

1. Loss of prestress due to all causes, excluding friction, may be determined by the fc ing method.* The method is based on normal weight concrete and one of the follo types of prestressing steel: 250 or 270 ksi, (1724 or 1862 MPa), seven-wire, stress-reli strand; 240 ksi (1655 MPa) stress-relieved wires; or 145 to 160 ksi (1000 to 1103 M smooth or deformed bars. For data regarding the properties and effects of lighweight ag gate concrete and low-relaxation tendons, refer to documented tests or see authori suppliers.

TOTAL LOSS

$$\Delta f_s = SH + ES + CR_c + CR_s$$

where Δf_s = total loss excluding friction in psi. (MPa)
SH = loss due to concrete shrinkage in psi. (MPa)
ES = loss due to elastic shortening in psi. (MPa)
CR_c = loss due to creep of concrete in psi. (MPa)
CR_s = loss due to relaxation of prestressing steel in psi. (MPa)

(a) SHRINKAGE

Pretensioned Members

$$SH = 17{,}000 - 150\ RH$$
$$= (117.21 - 1.034\ RH) \text{ in MPa}$$

Post-tensioned Members

$$SH = 0.80\ (17{,}000 - 150\ RH)$$
$$= 0.80\ \ 117.21 - 1.034\ RH \text{ in MPa}$$

where RH = average annual ambient relative humidity in percent
(See Figure 1.6.7)

*Should more exact prestress losses be desired, data representing the materials to be used, the methods of curing, the ambient service condition and any pertinent structural details should be determined for use in accordance with a method of calculating prestress losses that is supported by appropriate research data.

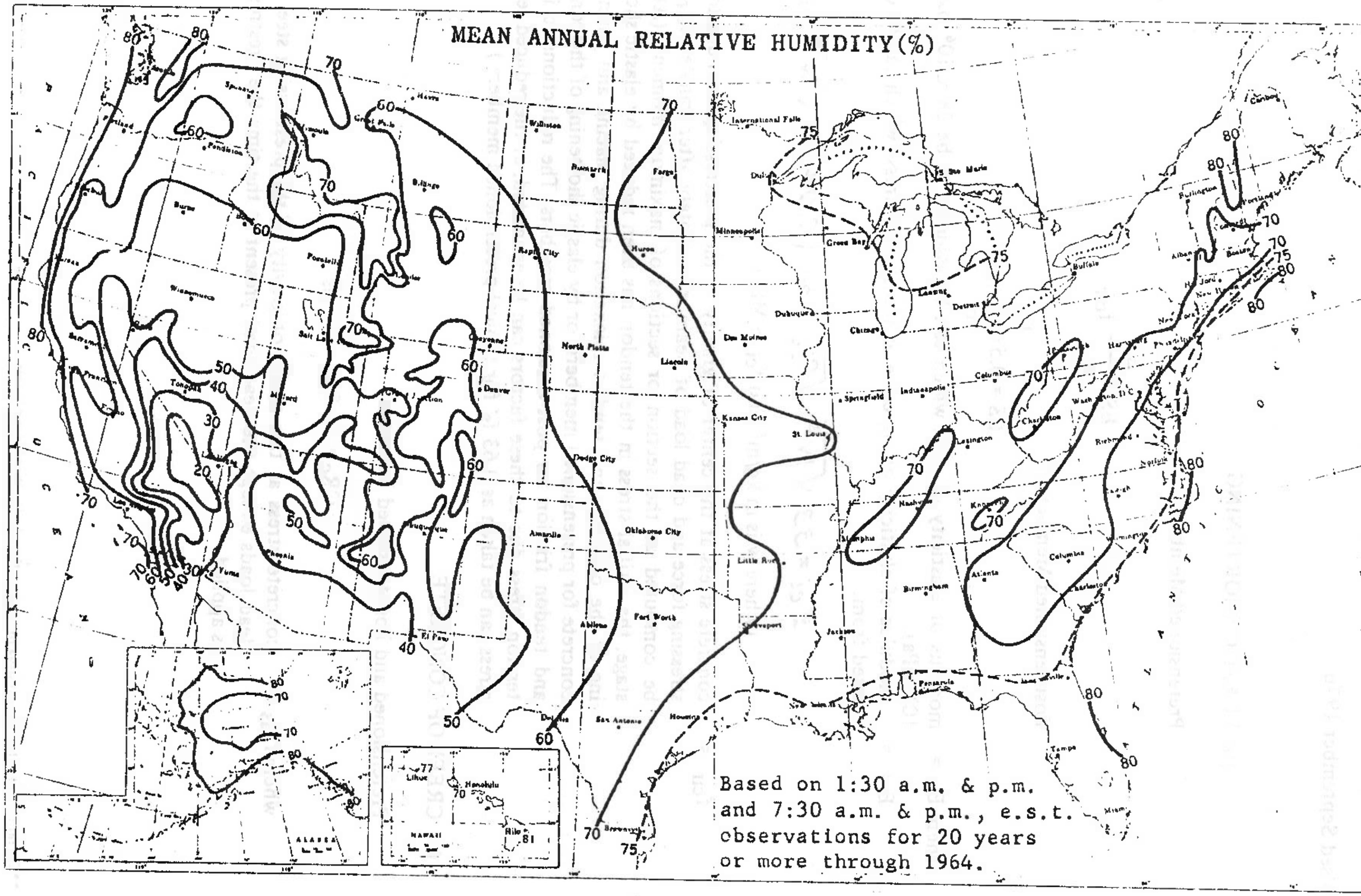
MEAN ANNUAL RELATIVE HUMIDITY (%)
Based on 1:30 a.m. & p.m. and 7:30 a.m. & p.m., e.s.t. observations for 20 years or more through 1964.

Figure 1.6.7

(b) ELASTIC SHORTENING

Pretensioned Members

$$ES = \frac{E_s}{E_{ci}} f_{cir}$$

Post-tensioned Members**

$$ES = 0.5\frac{E_s}{E_{ci}} f_{cir}$$

where E_s = modulus of elasticity of steel which can be assumed to be 28 x 10^6 psi (.193 x 10^6 MPa).

E_{ci} = modulus of elasticity of concrete at transfer of stress (which can be calculated from:

$$E_{ci} = 33\sqrt{W^3}\sqrt{f'_{ci}} \quad \text{or} \quad \left(0.0428\sqrt{W^3}\sqrt{f'_{ci}}\right)$$

(where w is in kg/m³ and f_{ci} in MPa)

f_{cir} = concrete stress at the center of gravity of the prestressing steel due to prestressing force and dead load of beam immediately after transfer. f_{cir} shall be computed at the section or sections of maximum moment. (At this stage, the initial stress in the tendon has been reduced by elastic shortening of the concrete and tendon relaxation during placing and curing the concrete for pretensioned members, or by elastic shortening of the concrete and tendon friction for post-tensioned members. The reductions to initial tendon stress due to these factors can be estimated, or the reduced tendon stress can be taken as 0.63 f_s' for typical pretensioned members.)

(c) CREEP OF CONCRETE

Pretensioned and Post-tensioned Members

$$CR_c = 12\, f_{cir} - 7\, f_{cds}$$

where f_{cds} = concrete stress at the center of gravity of the prestressing steel due to all dead loads except the dead load present at the time the prestressing force is applied.

**Certain tensioning procedures may alter the elastic shortening losses.

1.6.11 – NONPRESTRESSED REINFORCEMENT

Nonprestressed reinforcement may be considered as contributing to the tensile strength of the beam at ultimate strength in an amount equal to its area times its yield point, provided that

$$\frac{p f_{sy}}{f'_c} + \frac{p^* f^*_{su}}{f'_c} - \frac{p f_y}{f'_c}$$ does not exceed 0.3 for rectangular sections, or

$$\frac{A_s f'_{sy}}{b'df'_c} + \frac{A_{sr} f^*_{su}}{b'df'_c} - \frac{A'_s f'_y}{b'df'_c}$$ does not exceed 0.3 for flanged sections.

1.6.12 – CONTINUITY

A. General

Continuous beams and other statically indeterminate structures shall be designed for adequate strength and satisfactory behavior. Behavior shall be determined by elastic analysis, taking into account the reactions, moments, shear and axial forces produced by prestressing, the effects of temperature, creep, shrinkage, axial deformation, restraint of attached structural elements, and foundation settlement.

B. Cast-in-place Post-Tensioned Bridges

The effect of secondary moments due to prestressing shall be included in stress calculations at working load. In calculating ultimate strength moment and shear requirements the secondary moments or shears induced by prestressing (with a load factor of 1.0) shall be added algebraically to the moments and shears due to factored or ultimate dead and live loads.

COMMENTARY

The current provisions of AASHTO 1.6.12(B) were first included in the 1970 interim specifications. The provisions were consistent with similar provisions then under consideration and later adopted for the ACI 318-71 Building Code. However, as pointed out in Professor Alan Mattock's discussion in the September 1970 ACI Journal, and in a January-February 1972 PCI Journal Article by Professor T.Y. Lin and Mr. Keith Thornton, it is necessary to have either full moment redistribution or a concordent tendon (which does not produce secondary moments) in order to correctly ignore secondary moments. In view of the fact that the amount of moment redistribution permitted by the ACI Code is generally quite small, and the fact that no moment redistribution at all is permitted by the AASHTO Specifications, these specifications, while normally quite conservative, are in error. Further, it has been demonstrated by tests that will be discussed below that the effects of secondary moments (and shears) are reflected in the moments in continuous post-tensioned beams throughout loading conditions up to failure.

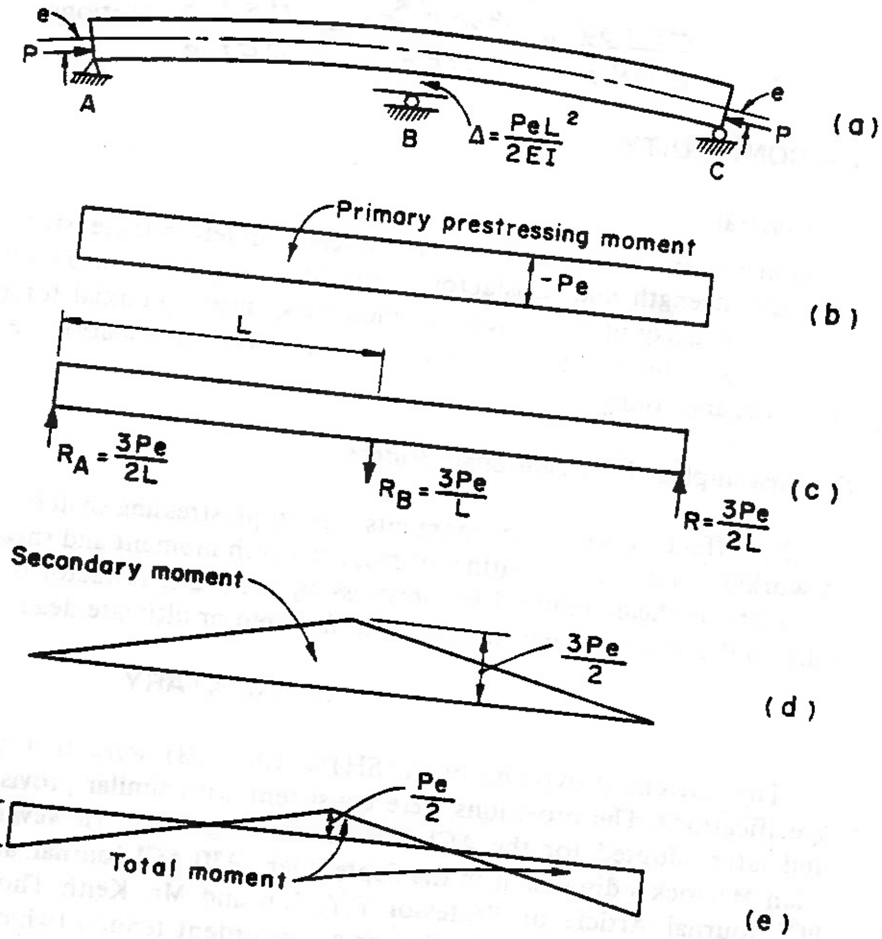
e
P
A
B
Δ = PeL²/2EI
C
e
P
(a)
Primary prestressing moment
-Pe
(b)
L
R_A = 3Pe/2L
R_B = 3Pe/L
R = 3Pe/2L
(c)
Secondary moment
3Pe/2
(d)
-Pe
Pe/2
Total moment
(e)

Figure 1

Section 7 – STRUCTURAL STEEL DESIGN

1.7.1 – ALLOWABLE STRESSES

TABLE 1.7.1

Allowable unit stresses shown in pounds per square inch.

The modulus of elasticity of all grades of steel shall be assumed to be 29,000,000 psi and the coefficient of linear expansion 0.0000065 per degree Fahrenheit.

		Structural Carbon Steel	High Strength Low Alloy Structural Steel			
ASTM Designation		A36	A588	A441		
Thickness of Plates		(5) Up to 8" Incl.	(5) Up to 4" Incl.	Over 1½" To 4" Incl.	Over ¾" To 1½" Incl.	¾" and Under
				Shapes (6)		
				Group 4.5	Group 3	Group 1.2
Minimum tensile strength	Fu	58,000	70,000	63,000	67,000	70,000
Minimum yield point	Fy	36,000	50,000	42,000	46,000	50,000
Axial Tension, net section Tension in extreme fiber of rolled shapes, girders and built-up sections subject to bending, net section Axial compression, gross section: stiffeners of plate girders. Compression in splice material, gross section.	Fb = .55Fy	20,000	27,000	23,000	25,000	27,000

		Structural Carbon Steel	High Strength Low Alloy Structural Steel			
ASTM Designation		A36	A588	A441		
Thickness of Plates		(5) Up to 8" Incl.	Up to 4" Incl.	Over 1½" To 4" Incl.	Over ¾" To 1½" Incl.	¾" and Under
				Shapes (6)		
				Group 4, 5	Group 3	Group 1, 2
Compression in extreme fibers of plates, rolled shapes, girders and built-up sections, subject to bending, gross section, when compression flange is: (A) Supported laterally its full length by embedment in concrete.	$F_b = .55F_y$	20,000	27,000	23,000	25,000	27,000
(B)(1) Partially supported or is unsupported.	(2) $F_u = .55F_y \left[1 - \frac{\left(\frac{\ell}{r'}\right)^2 F_y}{4\pi^2 E}\right]$ with $r'^2 = \frac{b^2}{12}$ $F_b = .55F_y \left[1 - \frac{3\left(\frac{\ell}{b}\right)^2 F_y}{\pi^2 E}\right]$	$= 20.000 - 7.5\left(\frac{\ell}{b}\right)^2$	$27.000 - 14.4\left(\frac{\ell}{b}\right)^2$	$23.000 - 10.2\left(\frac{\ell}{b}\right)^2$	$25,000 - 12.2\left(\frac{\ell}{b}\right)^2$	$27.000 - 14.4\left(\frac{\ell}{b}\right)^2$
with $\frac{\ell}{b}$ not greater than:		36	30	34	32	30
Compression in concentrically loaded columns. (3)						
with $C_c = (2\pi^2 E/F_y)^{1/2}$ = ...		126.1	107.0	116.7	111.6	107.0
when $KL/r \leq C_c$	$F_a = \frac{F_y}{F.S.}\left[1 - \frac{(KL/r)^2 F_y}{4\pi^2 E}\right] =$	$16.980 - .53(KL/r)^2$	$23.580 - 1.03(KL/r)^2$	$19,810 - .73(KL/r)^2$	$21,700 - .87(KL/r)^2$	$23,580 - 1.03(KL/r)^2$
when $KL/r \geq C_c$	$F_a = \frac{\pi^2 E}{F.S.(KL/r)^2} = \frac{135.000.740}{(KL/r)^2}$ with F.S. = 2.12					
Shear in beam and girder webs gross section	$F_v = 0.33\ F_y$	12,000	17,000	14,000	15,000	17,000

1.7.2 – ALLOWABLE STRESSES FOR WELD METAL

Unless otherwise specified, the yield point and ultimate strength of weld metal shall be equal to or greater than minimum specified value of the base metal. Allowable stresses on the effective areas of weld metal shall be as follows:

Butt Welds–

The same as the base metal joined, except in the case of joining metals of different yields when the lower yield material shall govern.

Fillet Welds–

F_V = 12,400 psi on base metal with minimum specified yield point of 36,000 psi or 1100 lbs. per inch per 1/8 in. of leg.

F_V = 14,700 psi on base metal with minimum specified yield point of between 40,000 psi and 50,000 psi inclusive or 1300 lbs. per inch per 1/8 in. of leg.

Plug Welds–

F_V = 12,400 psi for resistance to shear stress only.

Where F_V = Allowable basic shear stress.

1.7.3 REPETITIVE LOADING AND TOUGHNESS CONSIDERATIONS

A. Allowable Fatigue Stress

Members and fasteners subject to repeated variations or reversals of stress shall be designed so that the maximum stress does not exceed the basic allowable stresses given in Articles 1.7.1 and 1.7.2 and that the actual range of stress does not exceed the allowable fatigue stress range given in Table 1.7.3A1 for the appropriate type and location of material shown in Table 1.7.3A2 and illustrated in Figure 1.7.3.

The range of stress is defined as the algebraic difference between the maximum stress and the minimum stress. Tension stress is considered to have the opposite algebraic sign from compression stress.

In Table 1.7.3A2 "T" signifies range in tensile stress only; "Rev." signifies a range of stress involving both tension and compression during a stress cycle.

TABLE 1.7.3A1

Category See Table 1.7.3A2	Allowable Range of Stress, F_{sr} (ksi) or (MPa)			
	For 100,000 Cycles	For 500,000 Cycles	For 2,000,000 Cycles	For over 2,000,000 Cycles
A	60 (413.69)	36 (248.21)	24 (165.47)	24 (165.47)
B	45 (310.26)	27.5 (189.60)	18 (124.10)	16 (110.31)
C	32 (220.63)	19 (131.00)	13 (89.63)	10, 12* (68.97), (82.74)*
D	27 (186.16)	16 (110.31)	10 (68.95)	7 (48.26)
E	21 (144.79)	12.5 (86.18)	8 (55.15)	5 (34.47)
F	15 (103.42)	12 (82.74)	9 (62.05)	8 (55.15)

B. Load Cycles

The number of cycles of maximum stress to be considered in the design shall be selected from Table 1.7.3B unless traffic and loadometer surveys or other considerations indicate otherwise.

Allowable fatigue stresses shall apply to those Group Loadings that include live load or wind load.

The number of cycles of stress to be considered for wind loads in combinations with dead loads, except for structures where other considerations indicate a substantially different number of cycles, shall be 100,000 cycles.

*For transverse stiffener welds on girder webs or flanges.

TABLE 1.7.3A2

General Condition	Situation	Kind of Stress	Stress Category	Example of (See Fig. 1.7.3)
Plain Material	Base metal with rolled or cleaned surfaces. Flame cut edges with ANSI smoothness of 1000 or less	T or Rev.	A	1, 2
Built-up	Base metal and weld metal in members without attachments, built-up of plates or shapes connected by continuous full or partial penetration groove welds or by continuous fillet welds parallel to the direction of applied stress	T or Rev.	B	3, 5
	Calculated flexural stress at toe of transverse stiffener welds on girder webs on girder flanges	T or Rev.	C E	4
	Base metal at end of partial length welded over plates having square or tapered ends with or without welds across the ends	T or Rev.	E	5
Groove Welds	Base metal and weld metal at full penetration groove welded splices of rolled and welded section having similar profiles when welds are ground flush and weld soundness established by non-destructive inspection	T or Rev.	B	6, 8

TABLE 1.7.3A2 (continued)

General Condition	Situation	Kind of Stress	Stress Category	Example of (See Fig. 1.7.3)
Grove Welds (Con't.)	Base metal and weld metal in or adjacent to full penetration groove welded splices at transitions in width or thickness, with welds ground to provide slopes no steeper than 1 to 2½ with grinding in the direction of applied stress, and weld soundness established by non-destructive inspection	T or Rev.	B	9, 10
	Base metal at details attached by groove welds subject to transverse and/or longitudinal loading	T or Rev.	E	11
	Above With 1′ - 0″ Rad. Transition	T or Rev.	C	12
	Base metal and weld metal in or adjacent to full penetration groove welded splices, with or without transitions having slopes no greater than 1 to 2½ when reinforcement is not removed and weld soundness is established by non-destructive inspection.	T or Rev.	C	9, 10
Fillet Welded Connections	Base metal adjacent to Stud Shear Connectors	T or Rev.	D	13
	Base metal at details attached by fillet welds	T or Rev.	E	14, 15

TABLE 1.7.3A2 (continued)

Mechanically Fastened Connections	Base metal at gross section of high-strength bolted slip resistant connections except axially loaded joints which induce out-of-plane bending in connection material	T or Rev.	B	16
	Base metal at net section of high-strength bolted bearing-type connections and other mechanically fastened joints	T	B	16
Fillet Welds	Shear stress on throat of fillet welds	Shear	F	7

TABLE 1.7.3B – STRESS CYCLES

Main (Longitudinal) Load Carrying Members

Type of Road	Case	ADTT*	Truck Loading	Lane Loading†
Freeways, Expressways, Major Highways and Streets	I	2500 or more	over 2,000,000	500,000
	II	less than 2500	500,000	100,000
Other Highways and Streets not included in Case I or II	III	. . . .	100,000	100,000

Transverse Members and Details Subjected to Wheel Loads

Type of Road	Case	ADTT*	Truck Loading
Freeways, Expressways, Major Highways and Streets	I	2500 or more	over 2,000,000
	II	less than 2500	2,000,000
Other Highways and Streets	III	. . . .	500,000

* Average Daily Truck Traffic.
† Longitudinal members should also be checked for truck loading.

C. Charpy V-Notch Impact Requirements

Main load carrying member components subjected to tensile stress require supplemental impact properties as described in the Material Specifications.

These impact requirements vary depending on the type of steel, type of construction, welded or mechanically fastened, and the average minimum service temperature to which the structure may be subjected.[1] Table 1.7.3C contains the temperature zone designations.

TABLE 1.7.3C

Minimum Service Temperature	Temperature Zone Designation
0° F and above or (−18° C and above)	1
−1° F to −30° F or (−19° C to −34° C)	2
−31° F to −60° F or (−35° C to −51° C)	3

Components requiring mandatory impact properties shall be designated on the drawings and the appropriate zone shall be designated in the contract documents.

COMMENTARY

"Interim 8" of the 1974 Interim Specification has been retitled "Repetitive Loading and Toughness Considerations" and editorially rearranged into (A) Allowable Fatigue Stresses; (B) Load Cycles; and (C) Charpy V-Notch Impact Requirements. (A) and (B) are essentially an editorially rearrangement of the existing specification, (C) is a new section which brings into the Design Specification, portions of the provisions for mandatory Charpy V-Notch Impact Requirements which the Bridge Committee adopted in 1973 and which were subsequently incorporated into the 1974 AASHTO Material Specifications. These provisions are being placed in the Design Specification as they relate to design considerations rather than a material specification.

Reference has been added by footnote in order to give the designer access to a more complete background on the Charpy V-Notch Impact Requirements.

Since toughness considerations are not always germane to repetitive loadings, it is recommended that the 1977 edition of the Specifications be revised to place Article 1.7.3(C) Charpy V-Notch Impace Requirements in the book as a separate article.

[1] The basis and philosophy used to develop these requirements are given in a paper entitled "The Development of AASHTO Fracture-Toughness Requirements for Bridge Steels" by John M. Barsom, February 1975 available from the American Iron and Steel Institute, Washington, D.C.

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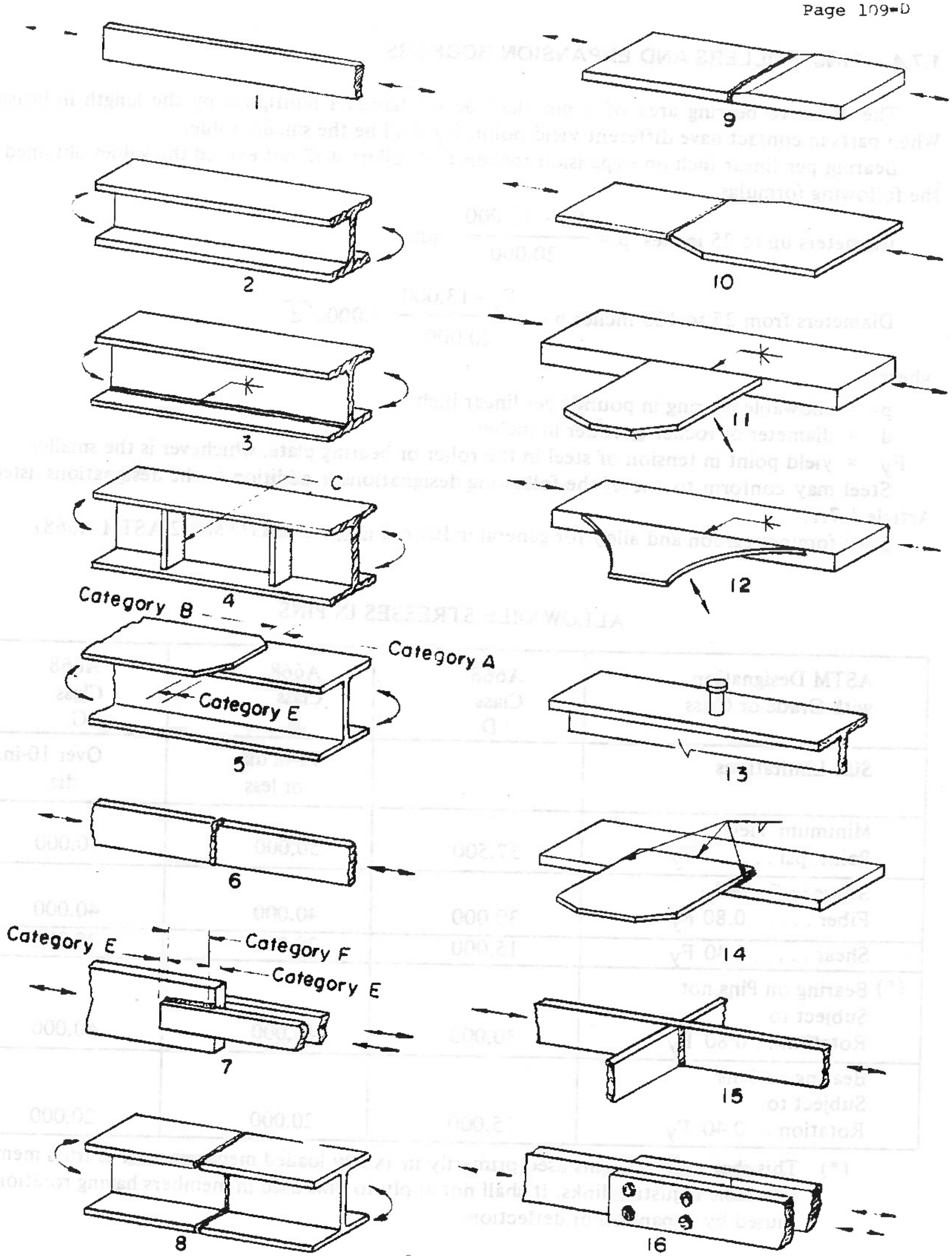

1
2
3
4
C
Category B
Category A
Category E
5
6
Category E
Category F
Category E
7
8
9
10
11
12
13
14
15
16

Fig. 1. 7. 3

1.7.4 – PINS, ROLLERS AND EXPANSION ROCKERS

The effective bearing area of a pin shall be its diameter multiplied by the length in bearing. When parts in contact have different yield point, F_y shall be the smaller value.

Bearing per linear inch on expansion rockers and rollers shall not exceed the values obtained by the following formulas:

Diameters up to 25 inches $p = \dfrac{F_y - 13{,}000}{20{,}000}\ 600d$

Diameters from 25 to 125 inches $p = \dfrac{F_y - 13{,}000}{20{,}000}\ 3{,}000\sqrt{d}$

where

p = allowable bearing in pounds per linear inch
d = diameter of rocker or roller in inches
F_y = yield point in tension of steel in the roller or bearing plate, whichever is the smaller.

Steel may conform to one of the following designations in addition to the designations listed in Article 1.7.1.

Steel forgings carbon and alloy for general industrial use, (AASHTO M102-ASTM A668)

ALLOWABLE STRESSES IN PINS

ASTM Designation with Grade or Class	A668 Class D	A668 Class F	A668 Class G
Size Limitations		10-in dia. or less	Over 10-in. dia.
Minimum Yield Point, psi . . . . . F_y	37,500	50,000	50,000
Stress in Extreme Fiber . . . . . 0.80 F_y	30,000	40,000	40,000
Shear . . . . . 0.40 F_y	15,000	20,000	20,000
(*) Bearing on Pins not Subject to Rotation . . 0.80 F_y	30,000	40,000	40,000
Bearing on Pins Subject to Rotation . . 0.40 F_y	15,000	20,000	20,000

(*) This shall apply to pins used primarily in axially loaded members such as truss members and cable adjusting links. It shall not apply to pins used in members having rotation caused by expansion or deflection.

1.7.38 – EDGE DISTANCE OF FASTENERS****

A. General

The minimum distance from the center of any fastener to the edge of a plate shall be:

For 1 1/8 inch fasteners, 2 inches.
For 1 inch fasteners, 1 3/4 inches.
For 7/8 inch fasteners, 1 1/2 inches
For 3/4 inch fasteners, 1 1/4 inches.
For 5/8 inch fasteners, 1 1/8 inches.

In the flanges or legs of rolled sections the distance shall be:

For 1 1/8 inch fasteners, 1 3/8 inches.
For 1 inch fasteners, 1 1/4 inches.
For 7/8 inch fasteners, 1 1/8 inches.
For 3/4 inch fasteners, 1 inch.
For 5/8 inch fasteners, 7/8 inch.

The maximum distance from any edge shall be eight times the thickness of the thinnest outside plate, but shall not exceed 5 inches.

B. Special

In bearing type connections having no more than two fasteners in a line parallel to the direction of stress, the distance between the center of the nearest fastener and that end of the connected member towards which the pressure from the fastener is directed shall not be less than:

$$\frac{AC}{T} \quad \text{for single shear}$$

or:

$$\frac{2AC}{T} \quad \text{for double shear}$$

Where A is the nominal cross-sectional area of the fastener, "T" is the thickness of the connected part and "C" is the ratio of specified minimum tensile strength of the fastener to the specified minimum tensile strength of the connected part. This end distance may be proportionately less where the shear stress per fastener is less than that permitted in Article 1.7.5, but not less than 1½ times the fastener diameter. It need not exceed 1½ times the transverse spacing of the fasteners.

1.7.39 – LONG RIVETS (DELETED)

1.7.40 – LINKS AND HANGERS

In pin-connected tension members other than eyebars, the net section across the pin hole shall be not less than 140 percent, and the net section back of the pin hole not less than 100 percent of the required net section of the body of the member. The ratio of the net width (through the pin hole transverse to the axis of the member) to the thickness of the segment shall not be more than 8. Flanges not bearing on the pin shall not be considered in the net section across the pin.

1.7.41 – LOCATION OF PINS

Pins shall be so located, with respect to the gravity axis of the members, as to reduce to a minimum, stresses due to bending.

1.7.42 – SIZE OF PINS

Pins shall be proportioned for the maximum shears and bending moments produced by the stresses in the members connected. If there are eyebars among the parts connected, the diameter of the pin shall be as specified in Article 1.7.47.

1.7.43 – WEB SECTION AT PIN HOLES*****

When necessary for the required section or bearing area, the section at the pin holes shall be increased by splicing on a section of web plate of the required thickness to the normal web.

1.7.44 – PINS AND PIN NUTS

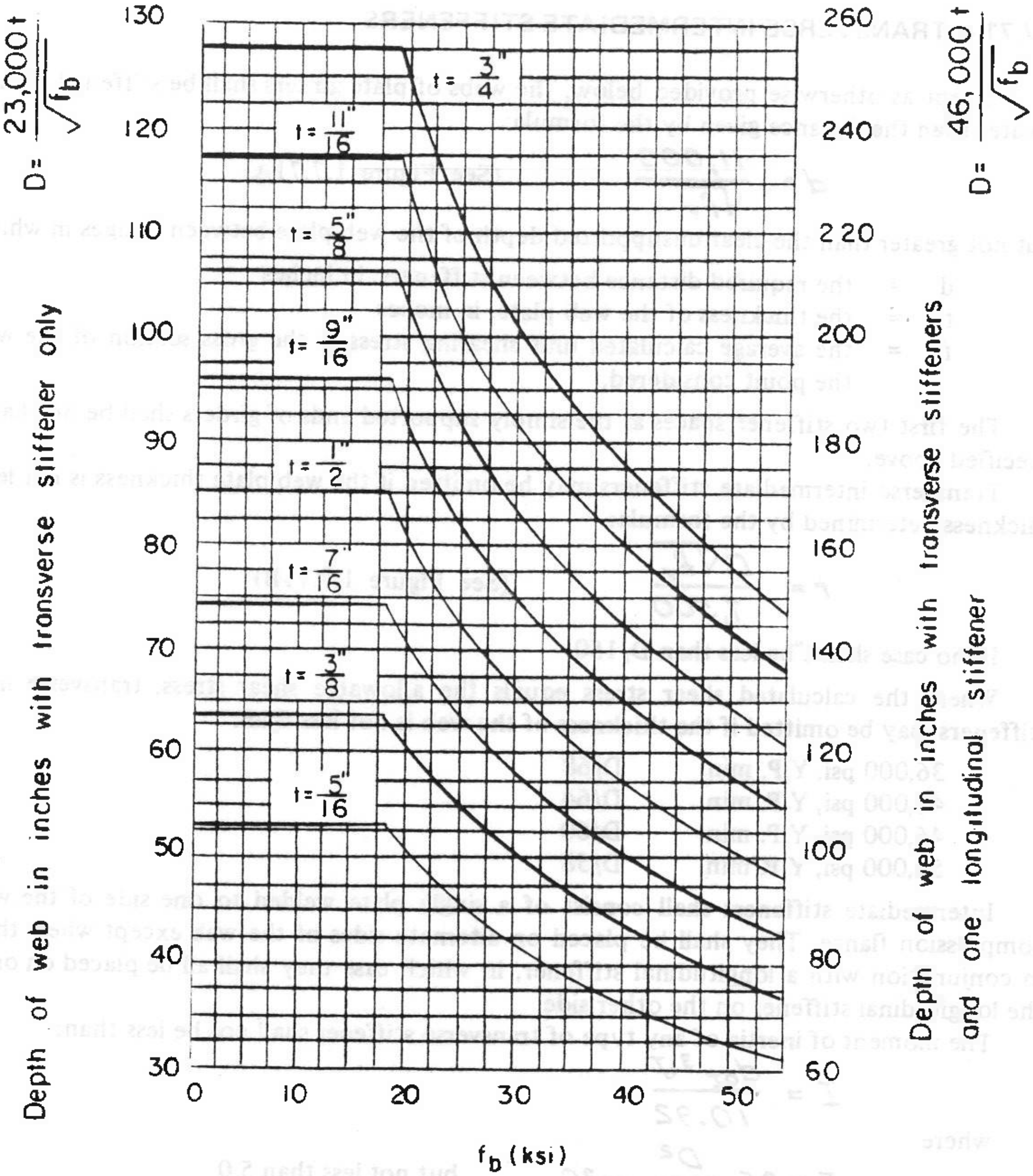
Pins shall be of sufficient length to secure a full bearing of all parts connected upon the turned body of the pin. They shall be secured in position by hexagonal recessed nuts or by hexagonal solid nuts with washers. If the pins are bored, through rods with cap washers may be used. Pin nuts shall be malleable castings or steel. They shall be secured by cotter pins in the screw ends or else the screw ends shall be long enough to permit burring the threads.

Members shall be held against lateral movement on the pins.

1.7.45 – UPSET ENDS (DELETED)

1.7.46 – EYEBARS

Eyebars shall be of a uniform thickness without reinforcement at the pin holes. The thickness of eyebars shall be not less than 1/8 of the width, nor less than 1/2 inch, and not greater than 2 inches. The section of the head through the center of the pin hole shall exceed that of the body of the bar by at least 35 percent. The net section back of the pin hole shall not be less than 75 percent



WEB THICKNESS AND GIRDER DEPTH

(a function of bending stress)

D = depth of web

t = thickness of web plate

f_b = calculated compressive bending stress in flange

FIGURE 1.7.70

1.7.71 – TRANSVERSE INTERMEDIATE STIFFENERS

Except as otherwise provided below, the webs of plate girders shall be stiffened at intervals not greater than the distance given by the formula:

$$d = \frac{11,000}{\sqrt{f_v}} \qquad \text{(See Figure 1.7.71A)}$$

but not greater than the clear unsupported depth of the web plate between flanges in which:

- d = the required distance between stiffeners, in inches
- t = the thickness of the web plate, in inches
- f_v = the average calculated unit shearing stress in the gross section of the web plate at the point considered.

The first two stiffener spaces at the simply supported ends of girders shall be one-half the value specified above.

Transverse intermediate stiffeners may be omitted if the web plate thickness is not less than the thickness determined by the formula:

$$t = \frac{D\sqrt{f_v}}{7,500} \qquad \text{(See Figure 1.7.71B)}$$

In no case shall "t" be less than D/150.

Where the calculated shear stress equals the allowable shear stress, transverse intermediate stiffeners may be omitted if the thickness of the web is not less than:

36,000 psi, Y.P. min.	D/68
42,000 psi, Y.P. min.	D/64
46,000 psi, Y.P. min.	D/60
50,000 psi, Y.P. min.	D/58

Intermediate stiffeners shall consist of a single plate welded to one side of the web and the compression flange. They shall be placed on alternate sides of the web except where they are used in conjunction with a longitudinal stiffener, in which case they shall all be placed on one side with the longitudinal stiffener on the other side.

The moment of inertia of any type of transverse stiffener shall not be less than:

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

where

$$J = 25\frac{D^2}{d^2} - 20 \qquad \text{but not less than 5.0.}$$

in these expressions

- I = The minimum permissible moment of inertia of any type of transverse intermediate stiffener.
- J = The required ratio of rigidity of one transverse stiffener to that of the web plate.
- d = The required distance between stiffeners, in inches.
- d_o = The actual distance between stiffeners, in inches.
- D = The unsupported depth of web plate between flange components, in inches.
- t = The thickness of the web plate, in inches.

For values of "w/t" exceeding $6650\sqrt{k_1} / \sqrt{F_y}$ but not exceeding 60, the stress in the flange, including stiffeners, shall not exceed the value given by the formula:

$$f_b = 14{,}400{,}000k(t/w)^2$$

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

If the longitudinal stiffeners are placed at their maximum "w/t" ratio in order for the flange to be designed for the basic allowable design stresses of "$0.55F_y$" and the number of longitudinal stiffeners exceeds 2, then transverse stiffeners should be considered.

D. Compression Flanges Stiffened Longitudinally and Transversely

The longitudinal stiffeners shall be at equal spacings across the flange width and shall be proportioned so that the moment of inertia of each stiffener about an axis parallel to the flange and at the base of the stiffener is at least equal to:

$$I_s = 8\, t^3\, w$$

The transverse stiffeners shall be proportioned so that the moment of inertia of each stiffener about an axis through the centroid of the section and parallel to its bottom edge is at least equal to:

$$I_t = 0.10\,(n+1)^3\, w^3\, \frac{f_s A_f}{E a}$$

where

A_f = area of bottom flange including longitudinal stiffeners
a = spacing of transverse stiffeners
f_s = maximum longitudinal bending stress in the flange of the panels on either side of the transverse stiffener
E = modulus of elasticity of steel

For the flange, including stiffeners, to be designed for the basic allowable stress of "$0.55F_y$", the ratio "w/t" for the longitudinal stiffeners shall not exceed the value given by the formula:

$$\frac{w}{t} = \frac{6650\sqrt{k_1}}{\sqrt{F_y}}$$

where:

$$k_1 = \frac{\left[1 + \left(\frac{a}{b}\right)^2\right]^2 + 87.3}{(n+1)^2 \left(\frac{a}{b}\right)^2 \left[1 + 0.1\,(n+1)\right]}$$

For greater values of w/t, but not exceeding $6650\sqrt{k_1}/\sqrt{f_y}$ or 60, whichever is less, the stress in the flange, including stiffeners shall not exceed the value given by the formula:

$$f_o = 0.55\,F_y - 0.224\,F_y\left[1 - \sin\left(2.92 - \frac{w\sqrt{F_y}}{2280\,t\sqrt{k_1}}\right)\right]$$

For values of "w/t" exceeding $6650\sqrt{k_1}/\sqrt{F_y}$ but not exceeding 60, the stress in the flange including stiffeners, shall not exceed the value given by the formula:

$$f_b = 14{,}400{,}000k_1(t/w)^2$$

The maximum value of the buckling coefficient "k_1" shall be 4. When "k_1" has its maximum value, the transverse stiffeners shall have a spacing, "a", equal to or less than "4w". If the ratio "a/b" exceeds 3, transverse stiffeners are not necessary.

The transverse stiffeners need not be connected to the flange plate but shall be connected to the webs of box and to each longitudinal stiffener. The connection to the web shall be designed to resist the vertical force determined by the formula:

$$R_w = \frac{F_y\,S_s}{2b}$$

where S_s = the section modulus of the transverse stiffener

The connection to each longitudinal stiffener shall be designed to resist the vertical force determined by the formula:

$$R_s = \frac{F_y\,S_s}{nb}$$

E. Compression Flange Stiffeners, General

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

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

where

b' = width of the outstanding stiffener element
t' = thickness of any outstanding stiffener element

Longitudinal stiffeners shall be extended to locations where the maximum stress in the flange does not exceed that allowed for base metal adjacent to or connected by fillet welds.

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

LOAD FACTOR DESIGN

1.7.117 – SCOPE

Load Factor design is an alternate method for design of simple and continuous beam and girder structures of moderate length. It is a method of proportioning structural members for multiples of the design loads. To insure serviceability and durability, consideration is given to the control of permanent deformations under overloads, to the fatigue characteristics under service loadings and to the control of live load deflections under service loadings. The unsupported compression flange of straight girders shall be checked for lateral buckling during erection in accordance with the procedure outlined in Article 1.7.69.

1.7.118 – NOTATION

A = area of cross section (in.^{2})
A_f = area of one flange of beam or girder (in.^{2})
A_s = total area of steel section including cover plates (in.^{2})
A_s = gross effective area of column cross section (in.^{2})
A_w = area of web of beam (in.^{2})
b' = width of projecting flange element (in.)
b' = width of outstanding stiffener element (in.)
D = dead load
D = distance center to center of box girder flange plates (in.)
d = depth of member (in.)
d_b = depth of beam
d_c = depth of column
d_o = distance between transverse stiffeners (in.)
d_w = depth of steel web of a composite section (in.)
E = modulus of elasticity (29,000,000 psi)
F = stress (psi)
F_{cr} = buckling stress (psi)
F_y = specified minimum yield point or yield strength of the type of steel being used (psi)
f'_c = specified 28-day compressive strength of concrete (psi)
I = impact
I = moment of inertia (in.^{4})
L_c = length of a compression member (in.)
L_b = distance between points of bracing of compression flange (in.)
L = live load
$M, M_1 M_2$ = moment on a cross section (in.-lb)
M_u = maximum moment capacity (in.-lb)
P = axial compression on the member (lb)
P_u = maximum axial compression capacity (lb)
r = radius of gyration (in.)
r_y = radius of gyration with respect to Y-Y axis (in.)

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

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

or

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

where

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

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

d. Maximum axial compression

$$P \leqslant 0.15 F_y A$$

where

A is the area of the cross section.

e. Maximum shear force

$$V \leqslant 0.55 F_y d t_w$$

2. Article 1.7.124(A) is applicable to steels with stress-strain diagrams which exhibit a yield plateau followed by a strain hardening range.

Steels such as ASTM A36, A242, A440, A441, A572 and A588 meet these requirements. The limitations set forth in Article 1.7.24(A) are given in Table 1.

TABLE 1

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

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

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

3. The shear capacity of beams and girders with webs fulfilling the requirements of Article 1.7.124 (E) (41) shall be computed as:

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

where:

$$V_P = 0.58 F_y D t_w$$

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

D = clear, unsupported distance between flange conponents.
d_o = distance between transverse stiffeners.

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

$$\frac{M}{M_u} = 1.375 - 0.625 \left(\frac{V}{V_u}\right)$$

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

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

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

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

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

where b′ is the projecting width of the stiffener.

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

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

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

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

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

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

$$I = d_o t_w^3 J$$

where:

$$J = 2.5\left(\frac{D}{d_o}\right)^2 - 2 \quad \text{but not less than 0.5.}$$

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

F. Longitudinally Stiffened Girders

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

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

$$\frac{D}{t_w} \leq \frac{73{,}000}{\sqrt{F_y}} \quad \text{but not less than 1/2''}$$

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

1.7.127 – POSITIVE MOMENT SECTIONS OF COMPOSITE BEAMS AND GIRDERS

A. Compact Sections

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

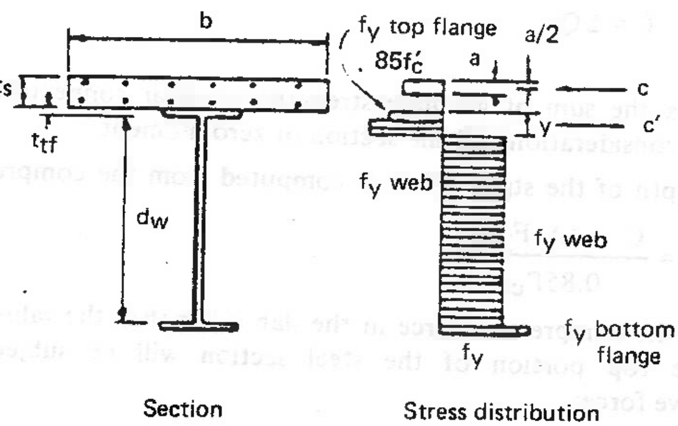


Fig. 1.7.127

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

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

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

where

"b" is the effective width of slab,

"t_s" is the slab thickness.

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

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

where

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

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

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

3. $C = \Sigma Q_u$

where

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

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

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

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

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

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

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

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

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

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

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

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

B. Non-Compact Sections

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

When the girders are not provided with temporary supports during the placing of dead loads, the sum of the stresses produced by $1.30D_S$ acting on the steel girder alone with $1.30(D_C + 5/3(L + I)$ acting on the composite girder shall not exceed yield stress at any point, where D_S and D_C are the moments 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 percent of its required 28-day strength, stresses produced by the loading, $1.30(D + 5/3(L + I))$, acting on the composite girder, shall not exceed yield stress at any point.

COMMENTARY

Article 1.7.127(C) General as now written indicates that its provisions would apply to both Compact and Non-Compact Sections. This is in error as a separation of non-composite stresses and composite stress has meaning only when the design criteria is based on first yielding. The proposed revision will correct an error made in the preparation of the original Specification.

1.7.128 – NEGATIVE MOMENT SECTIONS OF COMPOSITE BEAMS AND GIRDERS

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

1.7.129 – COMPOSITE BOX GIRDERS

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

A. Maximum Strength

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

$$M_u = F_{cr}S$$

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

B. Lateral Distribution

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

C. Web Plates

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

$$V_W = \frac{V}{\cos\theta}$$

where V = one half of the total vertical shear force on one box girder, θ = the angle of inclination of the web plate to the vertical.

The inclination of the web plates to the vertical shall not exceed 1 to 4.

D. Tension Flanges

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

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

E. Compression Flanges

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

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

where

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

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

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

in which c shall be taken as

$$c = \frac{13{,}000 - \frac{b}{t}\sqrt{F_y}}{7160}$$

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

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

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

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

where

ϕ = $0.07k^3 n^4$ when n equals 2, 3, 4 or 5.
ϕ = $0.125k^3$ when n = 1.
w = width of flange between longitudinal stiffeners or distance from a web to the nearest longitudinal stiffener.
n = number of longitudinal stiffeners.
k = buckling coefficient which shall not exceed 4.

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

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

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

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

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

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

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

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

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

where

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

F. Diaphragms

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

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

1.7.130 – SHEAR CONNECTORS

A. General

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

B. Design of Connectors

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

C. Maximum Spacing

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

2. Equivalent Moment Factor C

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

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

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

1.7.135 – SPLICES, CONNECTIONS & DETAILS

A. Connectors

1. General

Connectors shall be proportioned so that their maximum strength multiplied by the reduction factor, "ϕ" shall be at least equal to the effects of design loads multiplied by their respective load factors specified in Article 1.7.123. The maximum strengths multiplied by the reduction factors are listed in Table 3.

TABLE 3

Type of Fastener	Strength (ϕF)
Groove Weld[1]	1.00 F_y
Fillet Weld[2]	0.45 f_u
Low-Carbon Steel Bolts ASTM A307	
Tension	27 ksi
Shear[3]	25 ksi
Power-Driven Rivets ASTM A502	
Shear – Grade 1	25 ksi
Shear – Grade 2	30 ksi
High-Strength Bolts ASTM A325	
Tension[6]	76 ksi
Shear (Bearing-Type)[3 4 5]	54 ksi

(1)– F_y = yield point of connected material.
(2)– F_u = minimum strength of the welding rod metal but not greater than the tensile strength of the connected parts.
(3)–When a shear plane intersects the bolt threads, the root area shall be used.
(4)–Bearing stresses in bearing-type connections shall not exceed the tensile strength of the connected material.
(5)–For A235 bolts the tensile strength decreases for diameters greater than 7/8 in. The design value listed is for bolts up to 7/8 in. diameter. For diameters greater than 7/8 in. diameter the design value shall be computed as 0.56 F_u for tension and 0.45 F_u for shear where F_u is the ASTM minimum tensile strength of the bolt.

2. Welds

The ultimate strength of weld metal in groove welds shall be equal to or greater than that of the base metal. The ultimate strength of the weld metal in fillet welds need not match the strength of the base metal. However, the welding procedure and weld metal shall be selected to insure sound welds. The effective weld area shall be taken as defined in Article 1.7.28.

3. Bolts and Rivets

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

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

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

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

where

F_V = shear strength of the fastener, "ϕF", as given in Table 3.
f_t = tensile stress due to the applied load.

4. Friction Joints

Friction joints shall be designed to prevent slip at the overload in accordance with Article 1.7.136(C). Maximum strength of the bolts need not be considered in the design of such joints.

B. Connections

1. Splices

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

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

Section 8 – CORRUGATED METAL AND STRUCTURAL PLATE PIPES AND PIPE-ARCHES

1.8.1 GENERAL

The materials for the structure shall conform to the specifications set forth below, and the construction and installation shall conform to Section 23, Division II. The minimum gage or thickness shall be as determined by design in accordance with Article 1.8.2, except that such thickness shall be increased in accordance with Article 1.8.4 to provide for corrosion or abrasion unless there is evidence that corrosion or abrasion is not likely to occur.

Corrugated metal pipe composed of a smooth liner and corrugated shell attached integrally at seams spaced not more than 30 in. (762 mm) apart may be designed in accordance with Article 1.8.2 on the same basis as a standard corrugated metal pipe having the same corrugations as the shell and a weight per foot equal to the sum of the weights per foot of liner and corrugated shell.

This shall be limited to corrugations of 2 2/3 in. x 1/2 in and 3 in. x 1 in. The thickness of the corrugated shell shall be at least 60 percent of the total thickness of shell and liner, and the specified back-fill compaction shall be a minimum of 85 percent of standard density.

Where corrosion or abrasion are anticipated, thickness of shell and liner shall be increased in accordance with Article 1.8.4 or suitable coatings shall be specified.

Corrugated metal pipe and pipe-arches may be riveted, welded, or helical fabrication. The specifications are:

	Aluminum	Steel
Riveted	AASHTO M196	AASHTO M36*
Continuous Welded and Spot Welded	AASHTO M196	AASHTO M36
Helical Underdrain Culvert	AASHTO M196	AASHTO M36

Structural plate pipe and pipe-arches shall be bolted. The specifications are:

	Aluminum	Steel
Bolted	AASHTO M219	AASHTO M167 (6 x 2 corrugations)

Nothing included in this section shall be interpreted as prohibiting the use of new developments where usefulness can be substantiated.

*For 3 x 1 corrugations an equal number of 1/2 in. round ASTM A 325 bolts may be substituted for rivets.

1.8.2 – DESIGN

Four criteria must be considered in the structural design of a flexible buried conduit. Each considers the mutual function of the metal ring and the soil envelope surrounding it; interaction of these two materials produces a composite structure.

The criteria are:

A. Seam Strength
B. Handling and Installation Strength
C. Failure of the Conduit Wall
D. Deflection or Flattening

A. Seam Strength

Seam strength must be sufficient to withstand the thrust developing from the total load supported by the conduit.

This thrust, in lbs. per lineal ft. of structure is:

$$T = (LL + DL) \times \frac{\text{Span}}{2}$$

where

LL = design live load, psf. See Article 1.3.3
DL = dead load, psf. See Article 1.2.2(A) and 1.8.8
Span (or diameter), in ft.

Thrust, T, multiplied by safety factor, (see Article 1.8.8) should not exceed the seam strength. The strengths shown in Table 1.8.1 are recommended in the determination of fill heights. Longitudinal seams for corrugated metal pipe and pipe-arch shall develop the minimums shown in Table 1.8.1.

The ring compression stress, at which buckling becomes critical, in the elastic buckling zone for diameters greater than

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

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

where

f_u = minimum tensile strength, psi.
f_c = critical stress, psi, not to exceed the yield strength
k = soil stiffness factor
D = pipe diameter or span, in.
r = radius of gyration (corrugation)
E = modulus of elasticity, psi.

Design for buckling is accomplished by limiting the ring compression thrust, T, to the buckling stress multiplied by the condui. wall area per linear foot of structure divided by the safety factor.

D. Deflection or Flattening

Deflection or flattening. The Iowa Deflection Formula provides one approach to prediction of ring deflection. It relates ring deflection to the passive side pressure resisting horizontal movement of the pipe wall and to the inherent strength of the pipe. Pipe arches need not be checked for deflection.

The Iowa Deflection Formula is:

$$X = D_l \frac{K W_c R^3}{EI + 0.061 E' R^3}$$

where

X = horizontal deflection of the pipe, in.
D_l = deflection lag factor
K = a bedding constant (depends on bedding angle).
W_c = vertical load per unit length of pipe, lb/lin. in.
E = modulus of elasticity of pipe, psi (see Article 1.8.3)
R = mean radius of pipe, in.
I = moment of inertia per unit length of cross section of pipe wall, inches to the fourth power per in.
El = horizontal soil modulus, psi/in.

Other methods are available for predicting ring deflection.

1.8.3 – CHEMICAL AND MECHANICAL REQUIREMENTS

A. Aluminum – Corrugated Metal Pipe and Pipe-Arch

Chemical (ASTM B209)

Mechanical

Thickness, In.	Minimum Tensile Strength PSI	Minimum Yield Strength PSI	Minimum Elongation 2 inches	Modulus of Elasticity PSI
0.051 to 0.113	31,000	24,000	4%	10×10^6
0.114 to 0.249	31,000	24,000	5%	10×10^6

B. Aluminum – Structural Plate Pipe and Pipe-Arch

Chemical – AASHO M 219 ALLOY 5052

Mechanical

Thickness Inches	Minimum Tensile Strength psi	Minimum Yield Strength psi	Minimum Elon-gation 2 inches	Modulus of Elasticity psi
0.090 to 0.175	35,500	24,000	6%	10×10^6
0.175 to 0.250	34,000	24,000	6%	10×10^6

C. Steel – Corrugated Metal Pipe and Pipe-Arch

Chemical (ASTM A444)

Mechanical

Minimum Tensile Strength psi	Minimum Yield Strength psi	Minimum Elon-gation 2 inches	Modulus of Elasticity psi
45,000	33,000	20%	29×10^6

D. Steel – Structural Plate Pipe and Pipe-Arch

Chemical – AASHO M 167

Mechanical

Minimum Tensile Strength psi	Minimum Yield Strength psi	Minimum Elon-gation 2 inches	Modulus of Elasticity psi
42,000	28,000	30%	29×10^6

The mechanical properties shown are for the flat material prior to corrugating. These properties in the flat plate assure that the fabricated end product will have an effective minimum yield point of 33,000 psi or (227.527 MPa). A certificate on compliance shall be required from the manufacturer.

1.8.4 – ABRASIVE OR CORROSIVE CONDITIONS

For corrugated metal and structural plate pipes and pipe-arches having a thickness less than 0.25 inches, the entire conduit, or bottom plate only in the case of structural plate pipe, shall be of greater thickness, or protected by some other means, when required for resistance to abrasion or corrosion.

1.8.5 – RIVETS AND BOLTS

Rivets for corrugated sections and bolts for structural plate sections shall conform with the following:

Aluminum Corrugated Section:
Rivets-Aluminum, ASTM B 316, Alloy 6053-T4

Aluminum Structural Plates:
Bolts-Aluminum, ASTM B 211, Alloy 6061-T6
Bolts-Steel, AASHO M 164 (ASTM A 325)

Steel Corrugated Section:
Rivets-Steel, AASHO M 36

Steel Structural Plates:
Bolts-Steel, AASHO M 164 (ASTM A 325)

Where end treatment requires a rigid headwall, the plates or pipe shall be anchored to the headwall with not less than 3/4 inch anchor bolts at not more than 19 inch centers. Steel bolts for structural plate sections shall be torqued during installation to a minimum of 100 ft-lbs. and a maximum of 300 ft-lbs. Aluminum bolts for structural plate sections shall be torqued during installation to a minimum of 100 ft-lbs., and a maximum of 150 ft-lbs. For power-driven tools, the hold-on period may vary from 2 to 5 seconds.

Bolts shall be of sufficient length to provide for a full nut.

1.8.6 – MULTIPLE STRUCTURES

Where multiple lines of pipes or pipe-arches greater than 48 inches in diameter or span are used, they shall be spaced so that adjacent sides of the pipe shall be at least one-half diameter or 3 feet apart, whichever is less, to permit adequate compaction of backfill material. For diameters up to 48 inches, the minimum spacing shall be not less than 24 inches.

1.8.7 – SLOPED ENDS – SKEWED

When the skew angle exceed 20 degrees and the structure has the ends cut to fit the slope, the ends shall be reinforced.

1.8.8 – MAXIMUM DEPTHS OF COVER

Maximum depths of cover may be obtained by use of Section 1.8.2 and the following basic data: (Nothing included herein shall prohibit the use of other appropriate basic values).

Weight of embankment–120 lbs./cu.ft. or (1 930 kg/m^3)

k = 0.22, soil stiffness coefficient for good side fill material compacted to 90 percent of standard density based on AASHTO Specifications T99 (ASTM D698).

E′ = Modulus of passive soil (side fill) resistance: 1400 psi or (9.653 MPa)

Elongation:

5 percent of nominal diameter

Maximum Delfection:

5 percent of nominal diameter below circular shape

Safety factors used:

Longitudinal test seam strength = 3.0

Pipe wall buckline = 2.0

For pipe-arch structures placed on a stable foundation, the confining backfill must be capable of supporting a corner pressure of 2 tons per square foot. Marginally stable or compressible foundations require special investigation. Fill heights exceeding 100 feet shall be used only after a thorough investigation of the foundation material.

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.

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Section 9 – STRUCTURAL PLATE ARCHES

1.9.1 – GENERAL

Structural Plate Arches shall conform to Section 8, Division I, and to the specifications set forth below.

1.9.2 – RATIO, RISE TO SPAN

The design of single radius structural plate arches should be based on ratios of rise to span varying from 0.3 to 0.5.

1.9.3 – MINIMUM HEIGHT OF COVER

The minimum cover for design loads shall be $\frac{\text{Span}}{6}$ but not less than 12″.

1.9.4 – SCOUR CONDITIONS

Invert slabs shall be provided when scour is anticipated.

1.9.5 – MULTIPLE ARCHES

Where multiple arch spans are used, the distance between plates shall not be less than 1/10 of the longer adjoining span.

1.9.6 – SUBSTRUCTURE DESIGN

Special design considerations may be applicable. A buried Flexible structure may raise two important considerations. First is that it is undesirable to make the metal arch relatively unyielding or fixed compared to the adjacent sidefill. The use of massive footings or piles to prevent any settlement of the arch is generally not recommended. Where poor materials are encountered consideration should be given to removing some of this poor material and replacing it with acceptable material. The footing should be designed to provide uniform longitudinal settlement, of acceptable magnitude from a functional aspect. Providing for the arch to settle will protect it from possible overloading drag down forces settling adjacent side fill.

The second consideration is bearing power of soils under footings. Recognition must be given to the effect of depth of the base of footing and the direction of the footing reaction from the arch.

Footing reactions for the metal arch are considered to act tangential to the metal plate at its point of connection to the footing. The value of the reaction is the thrust in the metal arch plate at the footing.

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The multiplying factors for indicated lengths of bearing on such small areas as plates and washers become:

In using the preceding formula and table for round washers or bearing areas, use a length equal to the diameter.

D. Simple Solid Column Design

These formulas for simple solid columns are based on pin-end conditions but shall be applied also to square-end conditions.

Allowable unit stresses in pounds per square inch of cross-sectional area of simple solid columns shall be determined by the following formula, but such unit stresses shall not exceed the tabular values for compression parallel to grain, F_c as provided in Article 1.10.1 and adjusted in accordance with the applicable provisions of Article 1.10.1:

$$F'_c = \frac{\pi^2 E}{2.727\left(\frac{\ell}{r}\right)^2} = \frac{3.619E}{\left(\frac{\ell}{r}\right)^2}$$

where

F'_c = allowable unit stress in compression parallel to grain, in psi, adjusted for $\frac{\ell}{r}$ ratio
E = modulus of elasticity, psi
ℓ = unsupported overall length, in inches, between points of lateral support of simple columns
r = least radius of gyration of section

For columns of square or rectangular cross-section, this formula becomes:

$$F'_c = \frac{0.30E}{\left(\frac{\ell}{d}\right)^2}$$

where d = dimensions of least side of simple solid column in inches

For simple solid columns, the ℓ/d ratio may not exceed 50.

The values of F'_c as determined from the formulae listed are subject to adjustment for duration of load as given in Articles 1.10.1(D) and 1.10.1(E).

E. Spaced Column Design

Spaced columns are formed of two or more individual members with their longitudinal axes parallel, separated at the ends and middle points of their length by blocking and joined at the ends by timber connectors capable of developing the required shear resistance. To obtain spaced column action, end blocks with connectors and spacer blocks are required when the individual members of a spaced column assembly have an l/d ratio greater than

$$\sqrt{\frac{0.30E}{F_c}}$$

For an assembly of members having a lesser l/d ratio, the individual members are designed as simple solid columns. Spaced columns are classified as to fixity, i.e., condition "a" or condition "b," which introduces a multiplying factor applicable in the design of its individual members.

For individual members of a spaced column, ℓ/d shall not exceed 80, nor shall ℓ/d exceed 40.

The individual members in a spaced column are considered to act together to carry the total column load. Each member is designed separately on the basis of its l/d ratio.

A greater l/d ratio than allowed for simple solid columns is permitted because of the end fixity developed by the connectors and end blocks. This fixity is effective only in the thickness direction. The l/d ratio in the direction of width is subject to the provisions for simple solid columns.

When a single spacer block is located within the middle tenth of the column length (l), connectors are not required for this block. If there are two or more spacer blocks, connectors are required and the distance between two adjacent blocks shall not exceed one-half the distance between centers of connectors in the end blocks.

For spaced columns used as compression members of a truss, a panel point which is stayed laterally shall be considered as the end of the spaced column, and the portion of the web members, between the individual pieces making up a spaced column, may be considered as the end blocks.

If there are two or more connectors in a contact face, the center of gravity of the group shall be used in measuring the distance from connectors in the end block to the end of the column for determining fixity condition "a" or "b."

Thickness of spacer and end blocks shall not be less than that of individual members of the spaced column, nor shall thickness, width, and length of spacer and end blocks be less than required for connectors of a size and number capable of carrying the load computed in accordance with Article 1.10.2(I). Blocks thicker than a side member do not appreciably increase load capacity.

The total allowable load for a spaced column is the sum of the allowable loads for each of its individual members. Allowable unit stresses shall be determined as follows, but the maximum unit stress shall not exceed the values for compression parallel to grain "F_c" in Table 1.10.1, or as tabulated in the reference listed in Article 1.10.1(B)(1), and as adjusted in accordance with provisions of Article 1.10.1, nor shall the load exceed that permitted by the following provisions.

The net section shall be determined by subtracting, from the full cross-sectional area of the timber, the projected area of that portion of the connector groove within the members and that portion of the bolt hole not within the connector groove located at the critical plane. (See Table 2.20.1 for typical dimensions for Timber Connectors). Where connectors are staggered, adjacent connectors, with parallel-to-grain spacing equal to or less than one connector diameter, shall be considered as occurring at the same critical section.

In tension and compression members the required net area, in square inches, shall be determined by dividing the total load transferred through the critical section by the allowable tension stress for tension members, or by the allowable compression parallel to grain stress for compression members, for the species and grade of lumber used.

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