



COMMONWEALTH of VIRGINIA

DEPARTMENT OF TRANSPORTATION
1401 EAST BROAD STREET
RICHMOND, 23219-2000

David S. Ekern, P.E.
COMMISSIONER

September 11, 2007

MEMORANDUM

TO: Holders of the AASHTO Standard Specifications for Highway Bridges, 1996, Sixteenth Edition, Including the 1997 and 1998 Interim Specifications.

The enclosed VDOT Modification is made to the AASHTO Standard Specifications for Highway Bridges, 1996, Sixteenth Edition, including the 1997 and 1998 Interim Specifications.

Revisions:

Point Page 226-1: Article 10.6 was inadvertently omitted on the November 9, 1999 issue.
Page 21: Article 3.8.2.1: Impact for piers was omitted in last revisions.

Errors/Questions:

If you have any questions, please advise. If errors are noted, please notify me at: (804) 786-7537 or e-mail: volgyi_jf@vdot.state.va.us

/original signed/
Julius F. J. Völgyi, Jr., P.E.
Assistant State Structure and Bridge Engineer

Kendal R. Walus
State Structure and Bridge Engineer



COMMONWEALTH of VIRGINIA

DEPARTMENT OF TRANSPORTATION

1401 EAST BROAD STREET
RICHMOND, 23219-2000

David S. Ekern, P.E.
COMMISSIONER

October 25, 2006

SUBJECT: VDOT Modifications to AASHTO *Standard Specifications for Highway Bridges*, 16th Edition

MEMORANDUM

TO: Holders of the AASHTO *Standard Specifications for Highway Bridges*, 16th Edition, including the 1997 and 1998 Interim Specifications

The enclosed VDOT Modification is made to the AASHTO *Standard Specifications for Highway Bridges*, 16th Edition, including the 1997 and 1998 Interim Specifications:

REVISION:

Page 188: Article 8.22.1 showing minimum covers for reinforced concrete box culverts and rigid frames with less than and more than 2 ft. fill over top of slab was revised as parts of the cover were deleted on the March 8, 2000 issue.

/original signed/

Julius F. J. Völgyi, Jr., P.E.

Assistant State Structure and Bridge Engineer

For: George M. Clendenin, P.E.

State Structure and Bridge Engineer



COMMONWEALTH of VIRGINIA

DEPARTMENT OF TRANSPORTATION

1401 EAST BROAD STREET
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CHARLES D. NOTTINGHAM
COMMISSIONER

MALCOLM T. KERLEY
STATE STRUCTURE AND BRIDGE ENGINEER

March 8, 2000

VDOT Modifications to AASHTO Standard Specifications for Highway Bridges

MEMORANDUM

TO: Holders of the AASHTO Standard Specifications for Highway Bridges, 1996, Sixteenth Edition, Including the 1997 and 1998 Interim Specifications.

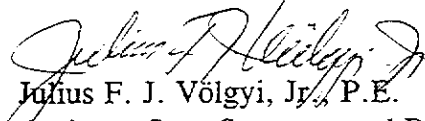
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Point Page 226-1: Article 10.6 was inadvertently omitted on the November 9, 1999 issue.

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Julius F. J. Volgyi, Jr., P.E.
Assistant State Structure and Bridge Engineer

Enclosures

For: Malcolm T. Kerley, P.E.
State Structure and Bridge Engineer

WE KEEP VIRGINIA MOVING



COMMONWEALTH of VIRGINIA

CHARLES D. NOTTINGHAM
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DEPARTMENT OF TRANSPORTATION
1401 EAST BROAD STREET
RICHMOND, 23219-1939

MALCOLM T. KERLEY
STATE STRUCTURE AND BRIDGE ENGINEER

November 9, 1999

VDOT Modifications to AASHTO Standard Specifications for Highway Bridges

MEMORANDUM

TO: Holders of the AASHTO Standard Specifications for Highway Bridges, 1996, Sixteenth Edition, Including the 1997 and 1998 Interim Specifications.

The enclosed VDOT Modifications are made to the AASHTO Standard Specifications for Highway Bridges, 1996, Sixteenth Edition, including the 1997 and 1998 Interim Specifications.

Effective Date:

The VDOT Modifications are in effect for all design started after December 1, 1999.

Instructions for Posting VDOT Modifications:

Post 1997 Interim Specifications to the 1996 AASHTO Specifications. Post 1998 Interim Specifications. Replace (or add) VDOT Modifications (summary of pages modified noted on enclosure).

Major changes to VDOT Modifications to the 14th Edition of the AASHTO Standard Specifications for Highway Bridges, including 1990 and 1991 Interims:

1. Cracking criteria for prestressed concrete beam design has been eliminated.
2. Restriction of plate girder flange thickness to 2½ inches has been eliminated provided that flanges > 2½ inches (in thickness) are welded in shop.
3. Overload vehicle for Load Factor Design has been deleted.

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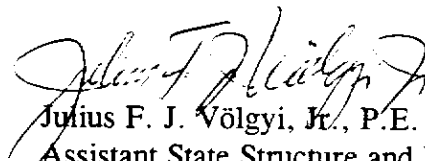
Page Sequencing and VDOT Modifications:

The date of the VDOT Modifications is located in the lower right hand corner for odd numbered pages or in the lower left hand corner for even numbered pages or adjacent to the AASHTO Interim dates (1997 or 1998). In the event there was not enough room to include the VDOT Modification on the original page, an extra page was added (labeled as a Point Page ---) to allow for shifting of the text. If the original text is modified, deleted, crossed through, etc. with a VDOT Modification, the change is noted with a vertical dashed line. In general, sheets are arranged as follows:

Sheet x	Example: 56
Sheet x inserted between sheets 56 and 57	Example: Point Page 56.1 and Point Page 56.2
Sheet x inserted between sheets Point Page 56.1 and Point Page 56.2	Example: Point Page 56.1-1 and Point Page 56.1-2

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Julius F. J. Volgyi, Jr., P.E.
Assistant State Structure and Bridge Engineer

Enclosures

For: Malcolm T. Kerley, P.E.
State Structure and Bridge Engineer

**VDOT Modifications to AASHTO Standard Specifications for Highway Bridges,
Sixteenth Edition (1996, Interim 1997 and 1998)**

Page 1 of 2

PAGE	DATE	PAGE	DATE
10	99Nov09	Point Page 184.1-1	99Nov09
Point Page 10.1	99Nov09	Point Page 184.1-2	99Nov09
Point Page 10.2	99Nov09	187	99Nov09
19	99Nov09	188	06Oct25
20	99Nov09	Point Page 188.1	99Nov09
21	07Sept11	Point page 188.2	99Nov09
23	99Nov09	195	99Nov09
26	99Nov09	198	99Nov09
Point Page 26.1	99Nov09	199	99Nov09
Point Page 26.2	99Nov09	Point Page 199.1	99Nov09
27	99Nov09	Point Page 199.2	99Nov09
30	99Nov09	200	99Nov09
Point page 30.1	99Nov09	202	99Nov09
Point Page 30.2	99Nov09	203	99Nov09
31	99Nov09	204	99Nov09
32	99Nov09	207	99Nov09
33	99Nov09	209	99Nov09
Point Page 33.1	99Nov09	217	99Nov09
Point Page 33.2	99Nov09	218	99Nov09
35	99Nov09	Point Page 218.1	99Nov09
36	99Nov09	Point Page 218.2	99Nov09
Point Page 36.1	99Nov09	223	99Nov09
Point Page 36.2	99Nov09	225	99Nov09
37	99Nov09	226	99Nov09
Point Page 37.1	99Nov09	Point Page 226.1	00Mar08
Point Page 37.2	99Nov09	Point Page 226.2	99Nov09
54	99Nov09	231	99Nov09
67	99Nov09	232	99Nov09
Point Page 67.1	99Nov09	233	99Nov09
Point Page 67.2	99Nov09	238	99Nov09
75	99Nov09	Point Page 238.1	99Nov09
Point Page 75.1	99Nov09	Point Page 238.2	99Nov09
Point Page 75.2	99Nov09	239	99Nov09
76	99Nov09	242	99Nov09
153	99Nov09	244	99Nov09
154	99Nov09	247	99Nov09
156	99Nov09	249	99Nov09
Point Page 156.1	99Nov09	256	99Nov09
Point Page 156.2	99Nov09	Point Page 256.1	99Nov09
165	99Nov09	Point Page 256.2	99Nov09
Point Page 165.1	99Nov09	265	99Nov09
Point Page 165.2	99Nov09	Point Page 265.1	99Nov09
169	99Nov09	Point Page 265.2	99Nov09
170	99Nov09	266	99Nov09
173	99Nov09	267	99Nov09
182	99Nov09	268	99Nov09
Point Page 184.1	99Nov09		

Note: Click the blue square at the top right of each page to return to this index.

07Sept11

**VDOT Modifications to AASHTO Standard Specifications for Highway Bridges,
Sixteenth Edition (1996, Interim 1997 and 1998)**

Page 2 of 2

PAGE	DATE
338	99Nov09
340	99Nov09
<u>Point Page 348.3</u>	99Nov09
<u>Point Page 348.4-1</u>	99Nov09
<u>Point Page 348.4-2</u>	99Nov09
<u>Point Page 348.5</u>	99Nov09
<u>Point Page 348.6</u>	99Nov09
<u>Point Page 348.7</u>	99Nov09
<u>Point Page 348.9</u>	99Nov09
<u>Point Page 348.10</u>	99Nov09
<u>371</u>	99Nov09
<u>Point Page 372.1</u>	99Nov09
<u>Point Page 372.2</u>	99Nov09
<u>397</u>	99Nov09
<u>Point Page 398.1</u>	99Nov09
<u>399</u>	99Nov09
<u>403</u>	99Nov09
<u>425</u>	07Sept11
<u>632</u>	99Nov09
<u>C-84</u>	99Nov09

07Sept11

The clearances and width of roadway for two-lane traffic shall be not less than those shown in Figure 2.5. The roadway width shall be increased at least 10 feet and preferably 12 feet for each additional traffic lane.

2.5.2 Clearance between Walls

The minimum width between walls of two-lane tunnels shall be 30 feet.

2.5.3 Vertical Clearance

The vertical clearance between curbs shall be not less than 14 feet.

2.5.4 Curbs

The width of curbs shall be not less than 18 inches. The height of curbs shall be as specified for bridges.

For heavy traffic roads, roadway widths greater than the above minima are recommended.

If traffic lane widths exceed 12 feet the roadway width may be reduced 2 feet 0 inches from that calculated from Figure 2.5.

2.6 HIGHWAY CLEARANCES FOR DEPRESSED ROADWAYS

2.6.1 Roadway Width

The clear width between curbs shall be not less than that specified for tunnels.

2.6.2 Clearance between Walls

The minimum width between walls for depressed roadways carrying two lanes of traffic shall be 30 feet.

2.6.3 Curbs

The width of curbs shall be not less than 18 inches. The height of curbs shall be as specified for bridges.

2.7 RAILINGS

Railings shall be provided along the edges of structures for protection of traffic and pedestrians. Other suitable applications may be warranted on bridge-length culverts as addressed in the 1989 AASHTO *Roadside Design Guide*.

Except on urban expressways, a pedestrian walkway may be separated from an adjacent roadway by a traffic

railing or barrier with a pedestrian railing along the edge of the structure. On urban expressways, the separation shall be made by a combination railing.

The designer shall consider the use of open rails/parapets for structures over bodies of water in accordance with the Commonwealth Transportation Board resolution dated July 20, 1995.

Special rail design for other uses on a case by case basis, e.g., historical considerations, may be used with the approval of the State Structure and Bridge Engineer.

For the timber initiative, FHWA crash tested timber rail shall be used.

See VDOT, Manual of the Structure and Bridge Division, Volume V – Part 3 (or part 3M for metric projects), Standard Details, for details on parapets and rails.

2.7.1 Vehicular Railing

2.7.1.1 General

2.7.1.1.1 Although the primary purpose of traffic railing is to contain the average vehicle using the structure, consideration should also be given to (a) protection of the occupants of a vehicle in collision with the railing, (b) protection of other vehicles near the collision, (c) protection of vehicles or pedestrians on roadways underneath the structure, and (d) appearance and freedom of view from passing vehicles.

2.7.1.1.2 Materials for traffic railings shall be concrete, metal, timber, or a combination thereof. Metal materials with less than 10-percent tested elongation shall not be used.

2.7.1.1.3 Traffic railings should provide a smooth, continuous face of rail on the traffic side with the posts set back from the face of rail. Structural continuity in the rail members, including anchorage of ends, is essential. The railing system shall be able to resist the applied loads at all locations.

2.7.1.1.4 Protrusions or depressions at rail joints shall be acceptable provided their thickness or depth is no greater than the wall thickness of the rail member or 3/8 inch, whichever is less.

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

2.7.1.2 Geometry

2.7.1.2.1 The heights of rails shall be measured relative to the reference surface which shall be the top of the roadway, the top of the future overlay if resurfacing is anticipated, or the top of the curb when the curb projection is greater than 9 inches from the traffic face of the railing.

2.7.1.2.2 Traffic railings and traffic portions of combination railings shall not be less than 2 feet 3 inches

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- Z = reduction for ductility and risk assessment
- β = (with appropriate script) coefficient applied to actual loads for service load and load factor designs (Article 3.22)
- Υ = load factor (Article 3.22)
- σ_{PL} = proportional limit stress perpendicular to grain (Article 3.25.1.4)
- β_B = load combination coefficient for buoyancy (Article 3.22.1)
- β_C = load combination coefficient for centrifugal force (Article 3.22.1)
- β_D = load combination coefficient for dead load (Article 3.22.1)
- β_E = load combination coefficient for earth pressure (Article 3.22.1)
- β_{EQ} = load combination coefficient for earthquake (Article 3.22.1)
- β_{ICE} = load combination coefficient for ice (Article 3.22.1)
- β_L = load combination coefficient for live load (Article 3.22)
- β_R = load combination coefficient for rib shortening, shrinkage, and temperature (Article 3.22.1)
- β_S = load combination coefficient for stream flow (Article 3.22.1)
- β_W = load combination coefficient for wind (Article 3.22.1)
- β_{WL} = load combination coefficient for wind on live load (Article 3.22.1)
- μ = Poisson's ratio (Article 3.23.4.3)

3.2 GENERAL

3.2.1 Structures shall be designed to carry the following loads and forces:

- Dead load.
- Live load.
- Impact of dynamic effect of the live load.
- Wind loads.
- Other forces when they exist, as follows:
Longitudinal forces, centrifugal force, thermal forces, earth pressure, buoyancy, shrinkage stresses, rib shortening, erection stresses, ice and current pressure, and earthquake stresses.

Provision shall be made for the transfer of forces between the superstructure and substructure to reflect the effect of friction at expansion bearings or shear resistance at elastomeric bearings.

3.2.2 Members shall be proportioned with reference to service loads and allowable stresses as provided in Service Load Design (Allowable Stress Design) Strength Design (Load Factor Design) with reference to load factors and factored strength may be used only when specifically authorized by the State Structure and Bridge Engineer.

3.2.3 When stress sheets are required, a diagram or notation of the assumed loads shall be shown and the stresses due to the various loads shall be shown separately.

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

3.2.5 The loading combinations shall be accordance with Article 3.22.

3.2.6 When a bridge is skewed, the loads and forces carried by the bridge through the deck system to pin connections and hangers should be resolved into vertical, lateral, and longitudinal force components to be considered in the design.

3.3 DEAD LOAD

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

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

3.3.2.1 If differential settlement is anticipated in a structure, consideration should be given to stresses resulting from this settlement.

3.3.3 If a separate wearing surface is to be placed when the bridge is constructed, or is expected to be placed in the future, adequate allowance shall be made for its weight in the design dead load. An allowance of 15 psf minimum shall be used for future wearing surface.

3.3.4 Special consideration shall be given to the necessity for a separate wearing surface for those regions where the use of chains on tires or studded snow tires can be anticipated.

3.3.5 Where traffic may bear directly on the concrete slab, ½ inch shall be added to the slab for a wearing surface.

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

	<u>#/cu.ft.</u>
Steel or cast steel	490
Cast Iron	150
Aluminum alloys	175
Timber (treated or untreated).....	50
Concrete, plain or reinforced	150
Compacted sand, earth and grave or ballast	120
Loose sand, earth and gravel	100
Macadam or gravel, rolled.....	140
Cinder filling	60
Pavement, other than wood block.....	150
Railways rails, guardrails and fastenings (per linear foot of track).....	200
Stone masonry	170
Asphalt plank, 1 in. thick.....	9 lb. sq.ft.

3.3.7 In addition to allowance for future wearing surface, provision shall be made for construction tolerances and construction methods by an allowance of 20 psf to be carried by the non-composite section of beam/girder spans having cast-in-place slabs.

This allowance is sufficient to cover additional loads incurred by the use of stay-in-place metal forms or prestressed concrete deck panels and extra slab thickness caused by construction tolerances.

3.4 LIVE LOAD

The live load shall consist of the weight of the applied moving load of vehicles, cars and pedestrians.

3.5 OVERLOAD PROVISIONS

3.5.1 For all loadings less than H20, provisions shall be made for an infrequent heavy load by applying Loading Combination IA (see Article 3.22), with the live load assumed to be H or HS truck and to occupy a single lane without concurrent loading in any other lane. The overload shall apply to all parts of the structure affected, except the roadway deck, or roadway deck plates and stiffening ribs in the case of orthotropic bridge superstructures.

3.5.2 Structures may be analyzed for an overload that is selected by the operating agency in accordance with Loading Combination IB in Article 3.22.

3.6 TRAFFIC LANES

3.6.1 The lane loading or standard truck shall be assumed to occupy a width of 10 feet.

3.6.2 These loads shall be placed in 12-foot wide design traffic lanes, spaced across the entire bridge roadway width measured between curbs.

3.6.3 Fractional parts of design lanes shall not be used, but roadway widths from 20 to 24 feet shall have two design lanes each equal to one-half the roadway width.

3.6.4 The traffic lanes shall be placed in such numbers and positions on the roadway, and the loads shall be placed in such positions within their individual traffic lanes, so as to produce the maximum stress in the member under consideration.

3.7 HIGHWAY LOADS

3.7.1 Standard Truck and Lane Loads*

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

3.7.1.2 Each lane shall consist of a uniform load per linear foot of traffic lane combined with a single concentrated load (or two concentrated loads in the case of continuous spans – see Article 3.11.3), so placed on the span as to produce maximum stress. The concentrated load and uniform load shall be considered as uniformly distributed over a 10-foot width on a line normal to the center line of the lane.

3.7.1.3 For the computation of moments and shears, different concentrated loads shall be used as indicated in Figure 3.7.6B. The lighter concentrated loads shall be used when the stresses are primarily bending stresses, and the heavier concentrated loads shall be used when the stresses are primarily shearing stresses.

*Note: The system of lane loads defined here (and illustrated in Figure 3.7.6B) was developed in order to give a simpler method of calculating moments and shears than that based on wheel loads of the truck.

Appendix B shows the truck train loadings of the 1935 Specifications of AASHTO and the corresponding lane loadings.

In 1944, the HS series of trucks was developed. These approximate the effect of the corresponding 1935 truck preceded and followed by a train of trucks weighing three-fourths as much as the basic truck.

3.7.2 Classes of Loading

There are four standard classes of highway loading: H 20, H 15, HS 20, and HS 15. Loading H 15 is 75 percent of loading H 20. Loading HS 15 is 75 percent of Loading HS 20. If loadings other than those designated are desired, they shall be obtained by proportionately changing the weights shown for both the standard truck and the corresponding lane loads.

3.7.3 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:

H15 Loading, 1944 Edition shall be designated	H 15-44
H 20 Loading, 1944 Edition shall be designated	H 20-44
H 15-S 12 Loading, 1944 Edition shall be designated	HS 15-44
H 20-S 16 Loading, 1944 Edition shall be designated	HS 20-44

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

3.7.4 Minimum Loading

Bridges supporting Interstate highways or other classes of roads shall be designed for HS20-44 Loading or an Alternate Military Loading of two axles four feet apart with each axle weighing 24,000 pounds, whichever produces the greatest stress. In special cases or as directed by the State Structure and Bridge Engineer, other loadings may be used including HS25. HS25 loading shall be 5/4 x HS20 loading and shall include Alternate Military Loading.

3.7.5 H Loading

The H loadings consist of a two-axle truck or corresponding lane loadings as illustrated in Figures 3.7.6A and 3.7.6B. The H loadings are designated H followed by a number indicating the gross weight in tons of the standard truck.

3.7.6 HS Loadings

The HS loadings consist of a tractor truck with semi-trailer or the corresponding lane load as illustrated in Figures 3.7.7A and 3.7.7B. 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 the 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 moments.

3.8 IMPACT

3.8.1 Application

Highway Live Loads shall be increased for those structural elements in Group A, below, to allow for dynamic, vibratory and impact effects. Impact allowances shall not be applied to items in Group B. It is intended that impact be included as part of the loads transferred from superstructure to substructure, but shall not be included in loads transferred to footings nor to those parts of piles or columns that are below ground.

3.8.1.1 Group A – Impact shall be included.

- (1) Superstructure, including legs of rigid frames.
- (2) Piers, (with or without bearings regardless of type) excluding footings and those portions below the ground line.
- (3) The portions above the ground line of concrete or steel piles that support the superstructure.

3.8.1.2 Group B – Impact shall not be included.

- (1) Abutments, retaining walls, piles except as specified in 3.8.1.1 (3).
- (2) Foundation pressures and footings.
- (3) Timberlake structures.
- (4) Sidewalk loads.
- (5) Culverts and structures having 3 feet or more cover.

3.8.2 Impact Formula

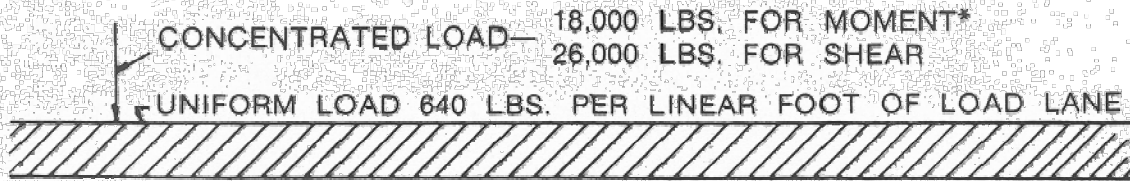
3.8.2.1 The amount of the impact allowance or increment is expressed as a fraction of the live load stress, and shall be determined by the formula:

$$I = \frac{50}{L + 125} \tag{3-1}$$

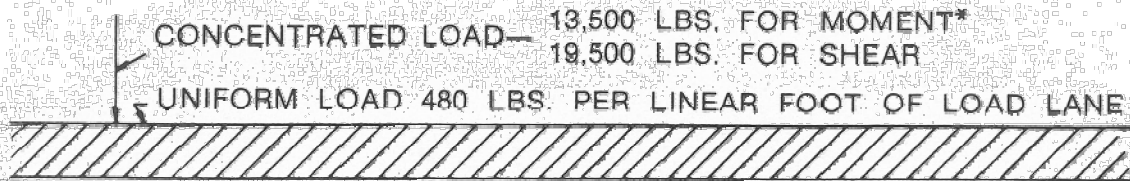
in which,

I = impact fraction (maximum 30 percent):

For piers, where impact is specified, use 25 percent.



H20-44 LOADING
HS20-44 LOADING



H15-44 LOADING
HS15-44 LOADING

Figure 3.7.6B. Lane Loading

*For the loading of continuous spans involving lane loading refer to Article 3.11.3 which provides for an additional concentrated load.

L = length in feet of the portion of the span that is loaded to produce the maximum stress in the member.

3.8.2.2 For uniformity of application, in this formula, the loaded length, L, shall be as follows:

- (a) For roadway floors: the design span length.
- (b) For transverse members, such as floor beams: the span length of member center to center of supports.
- (c) For computing truck load moments: the span length, or for cantilever arms the length from the moment center to the farthest axle.
- (d) For shear due to truck loads: the length of the loaded portion of span from the point under consideration to the far reaction; except, for cantilever arms, use a 30 percent impact factor.
- (e) For continuous spans: the length of span under consideration for positive moment, and the average of two adjacent loaded spans for negative moment.

3.8.2.3 For culverts with cover

- 0'-0" to 1'-0" inc. I = 30%
- 1'-1" to 2'-0" inc. I = 20%
- 2'-1" to 2'-11" inc. I = 10%

3.9 LONGITUDINAL FORCES

Provision shall be made for the effect of a longitudinal force of 5 percent of the live load in all lanes carrying traffic headed in the same direction. All lanes shall be loaded for bridges likely to become one directional in the future. The load used, without impact, shall be the lane load plus the concentrated load for moment specified in Article 3.7, with reduction for multiple-loaded lanes as specified in Article 3.12. The center of gravity of the longitudinal force shall be assumed to be located 6 feet above the floor slab and to be transmitted to the substructure through the superstructure.

In continuous spans, each expansion bearing resists its proportional share of the longitudinal force provided that its frictional resistance is not exceeded. The longitudinal force shall be distributed to the substructure in ratio of the reactions.

**3.14 SIDEWALK, CURB, AND RAILING
LOADING**

3.14.1 Sidewalk Loading

3.14.1 Sidewalk floors, stringers, and their immediate supports shall be designed for a live load of 85 pounds per square foot of sidewalk area. Girders, trusses, arches, and other members shall be designed for the following sidewalk live loads:

- Spans 0 to 25 feet in length85 lb./ft.²
- Spans 26 to 100 feet in length60 lb./ft.²
- Spans over 100 feet in length according to the formula

$$P = \left(30 + \frac{3,000}{L} \right) \left(\frac{55 - W}{50} \right) \quad (3-3)$$

in which

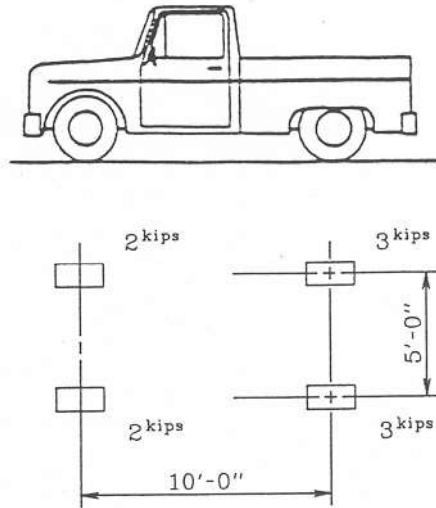
- P = live load per square foot, max. 60-lb. per sq. ft.
- L = loaded length of sidewalk in feet.
- W = width of sidewalk in feet.

3.14.1.2 In calculating stresses in structures that support cantilevered sidewalks, the sidewalk shall be fully loaded on only one side of the structure if this condition produces maximum stress.

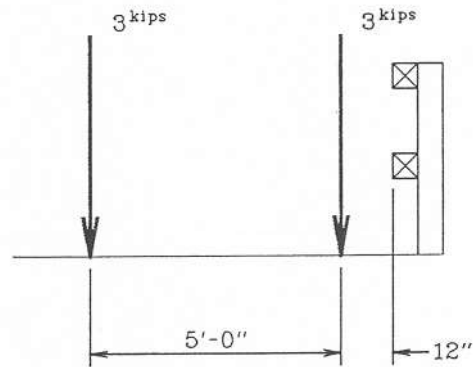
3.14.1.3 Bridges for pedestrian and/or bicycle traffic shall be designed for a live load of 85 PSF.

3.14.1.4 Where bicycle or pedestrian bridges are expected to be used by maintenance vehicles, special design consideration should be made for these loads.

Where the width of a structure between handrails is greater than 6 feet but less than 12 feet and accessible to small vehicle traffic for maintenance purposes, the longitudinal beams/girders shall be designed for a live load of 10,000 pounds as shown below:



The deck shall be designed for two concentrated loads of 3000 pounds each, at 5 foot centers located not less than 1'-0" from the face of rail.



Pedestrian bridges wider than 12 feet between handrails shall receive special consideration. A dead load wearing surface of 12 psf should be considered if it is anticipated that the bridge could be overlaid in the future.

3.15 WIND LOADS

The wind load shall consist of moving uniformly distributed loads applied to the exposed area of the structure. The exposed area shall be the sum of the areas of all members, including floor system and railing, as seen in elevation at 90 degrees to the longitudinal axis of the structure. The forces and loads given herein are for a base wind velocity of 100 miles per hour. For Group II and Group V loadings, but not for Group III and Group VI loadings, they may be reduced or increased in the ratio of the square of the design wind velocity to the square of the base wind velocity provided that the maximum probable wind velocity can be ascertained with reasonable accuracy, or provided that there are permanent features of the terrain which make such changes safe and advisable. If a change in the design wind velocity is made, the design wind velocity shall be shown on the plans.

3.15.1 Superstructure Design

3.15.1.1 Group II and Group V Loadings

3.15.1.1.1 A wind load of the following intensity shall be applied horizontally at right angles to the longitudinal axis of the structure:

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

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

3.15.1.2 Group III and Group VI Loadings

Group III and Group VI loadings shall comprise the loads used for Group II and Group V loadings reduced by 70 percent and a load of 100 pounds per linear foot applied at right angles to the longitudinal axis of the structure and 6 feet above the deck as a wind load on a moving live load. When a reinforced concrete floor slab or a steel grid deck is keyed to or attached to its supporting members, it may be assumed that the deck resists, within its plane, the shear resulting from the wind load on the moving live load.

3.15.2 Substructure Design

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

3.14.2 Curb Loading

3.14.2.1 Curbs shall be designed to resist a lateral force of not less than 500 pounds per linear foot of curb, applied at the top of the curb, or at an elevation 10 inches about the floor if the curb is higher than 10 inches.

3.14.2.2 Where sidewalk, curb and traffic rail form an integral system, the traffic railing loading shall be applied and stresses in curbs computed accordingly.

3.14.3 Railing Loading

For Railing Loads, see Article 2.7.

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3.15.2.1 Forces from Superstructure

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

Skew Angle Of Wind	Trusses		Girders	
	Lateral	Longitudinal	Lateral	Longitudinal
	Load	Load	Load	Load
Degrees	PSF	PSF	PSF	PSF
0	75	0	50	0
15	70	12	44	6
30	65	28	41	12
45	47	41	33	16
60	24	50	17	19

The loads listed above shall be used in Group II and Group V loadings as given in Article 3.22.

3.15.2.1.2 For Group III and Group VI loadings, these loads may be reduced by 70 percent and a load per linear foot added as a wind load on a moving live load, as given in the following table:

Skew Angle of Wind	Lateral Load	Longitudinal Load
Degrees	lb./ft.	lb./ft.
0	100	0
15	88	12
30	82	24
45	66	32
6	34	38

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

3.15.2.1.3 For the usual girder and slab bridges having maximum span lengths of 125 feet, the following wind loading may be used in lieu of the more precise loading specified above.

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

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

The loads are not specific with any wind direction; therefore, no direction of wind is implied in their usage.

3.15.2.2 Forces applied directly to the Substructure

The transverse and longitudinal forces to be applied directly to the substructure for a 100-mile per hour wind shall be calculated from an assumed wind force of 40 pounds per square foot. For wind directions assumed skewed to the substructure, this force shall be resolved into components perpendicular to the end and front elevations of the substructure. The component perpendicular to the end elevation shall act on the exposed substructure area as seen in end elevation and the component perpendicular to the front elevation shall act on the exposed areas and shall be applied simultaneously with the wind loads from the superstructure. The above loads are for Group II and Group V loadings and may be reduced by 70 percent for Group III and Group VI loadings, as indicated in Article 3.22.

3.15.3 Overturning Forces

The effect of forces tending to overturn structures shall be calculated under Groups II, III, V, and VI of Article 3.22 assuming that the wind direction is at right angles to the longitudinal axis of the structure. In addition, an upward force shall be applied at the windward quarter point of the transverse superstructure width. This force shall be 20 pounds per square foot of deck and sidewalk plan area for Group II and Group V combinations and 6 pounds per square foot for Group III and Group VI combinations.

3.16 THERMAL FORCES

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

TABLE 3.22.1A Table of Coefficients γ and β

Col. No.	1	2	3	3A	4	5	6	7	8	9	10	11	12	13	14	
GROUP	γ	β FACTORS													%	
		D	(L+I) _n	(L+I) _p	CF	E	B	SF	W	WL	LF	R+S+T	EQ	ICE		
SERVICE LOAD	I	1.0	1	1	0	1	β_E	1	1	0	0	0	0	0	0	100
	1A	1.0	1	2	0	0	0	0	0	0	0	0	0	0	0	150
	1B	1.0	1	0	1	1	β_E	1	1	0	0	0	0	0	0	**
	II	1.0	1	0	0	0	1	1	1	1	0	0	0	0	0	125
	III	1.0	1	1	0	1	β_E	1	1	0.3	1	1	0	0	0	125
	IV	1.0	1	1	0	1	β_E	1	1	0	0	0	1	0	0	125
	V	1.0	1	0	0	0	1	1	1	1	0	0	1	0	0	140
	VI	1.0	1	1	0	1	β_E	1	1	0.3	1	1	1	0	0	140
	VII	1.0	1	0	0	0	1	1	1	0	0	0	0	1	0	133
	VIII	1.0	1	1	0	1	1	1	1	0	0	0	0	0	1	140
IX	1.0	1	0	0	0	1	1	1	1	0	0	0	0	1	150	
X	1.0	1	1	0	0	β_E	0	0	0	0	0	0	0	0	100	
LOAD FACTOR DESIGN	I	1.3	β_D	1.67*	0	1.0	β_E	1	1	0	0	0	0	0	0	Not Applicable
	1A	1.3	β_D	2.20	0	0	0	0	0	0	0	0	0	0	0	
	1B	1.3	β_D	0	0	1.0	β_E	1	1	0	0	0	0	0	0	
	II	1.3	β_D	0	0	0	β_E	1	1	1	0	0	0	0	0	
	III	1.3	β_D	1	0	1	β_E	1	1	0.3	1	1	0	0	0	
	IV	1.3	β_D	1	0	1	β_E	1	1	0	0	0	1	0	0	
	V	1.25	β_D	0	0	0	β_E	1	1	1	0	0	1	0	0	
	VI	1.25	β_D	1	0	1	β_E	1	1	0.3	1	1	1	0	0	
	VII	1.3	β_D	0	0	0	β_E	1	1	0	0	0	0	0	0	
	VIII	1.3	β_D	1	0	1	β_E	1	1	0	0	0	0	0	1	
IX	1.20	β_D	0	0	0	β_E	1	1	1	0	0	0	0	1		
X	1.30	1	1.67	0	0	β_E	0	0	0	0	0	0	0	0	100	

(L + I)_n – Live load plus impact for AASHTO Highway H or HS loading.
 (L + I)_p – Live load plus impact consistent with the overload criteria of the operation agency.

For culvert loading specifications, see Article 6.2.

*1.25 may be used for design of outside roadway beam when combination of sidewalk live load as well as traffic live load plus impact governs the design, but the capacity of the section should not be less than required for highway traffic live load only using a beta factor of 1.67. 1.00 may be used for design of deck slab with combination of loads as described in Article 3.24.2.2.

**Percentage = $\frac{\text{Maximum Unit Stress (Operating Rating)}}{\text{Allowable Basic Unit Stress}} \times 100$

For Service Load Design

% (Column 14) Percentage of Basic Unit Stress

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

β_E : See Table 3.22.1B

For Load Factor Design

β_E : See Table 3.22.1B

$\beta_D = 0.75$ when checking member for minimum axial load and maximum eccentricityFor

$\beta_D = 1.0$ when checking number for maximum axial load and minimum momentDesign

$\beta_D = 1.0$ for flexural and tension members

For Group X loading (culverts) the β_E factor shall be applied to vertical and horizontal loads.

TABLE 3.22.1B				
β_E				
	SERVICE LOAD		LOAD FACTOR	
	Vertical Loads	Lateral Loads	Vertical Loads	Lateral Loads
Retaining Walls	1.00	1.00	1.00	1.30
Reinforced concrete boxes	1.00	††† 1.00	1.00	††† 1.00
Rigid frames	1.00	†† 1.00 & 0.50	1.00	†† 1.30 & 0.50
† Rigid culverts	1.00	1.00	1.00	1.00
Flexible culverts	1.00	1.00	1.50	1.50
All other structures	1.00	1.00	1.00	1.00

† Excluding reinforced concrete boxes.

†† Check both loading to see which one governs (See Article 3.20).

††† For culvert loading specifications, see Article 6.2.

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there is reason to believe that the ice sheets are subject to significant thermal movements relative to the piers.

3.19 BUOYANCY

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

3.20 EARTH PRESSURE

3.20.1 Structures which retain fills shall be proportioned to withstand pressure as given by Rankine's formula or by other expressions given in Section 5, "Retaining Walls"; provided, however, that no structure shall be designed for less than an equivalent fluid weight (mass) of 30 pounds per cubic foot.

3.20.2 Earth pressure shall be reduced to one-half of that specified above when this reduction produces maximum stresses.

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

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

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

3.21 EARTHQUAKES

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

Part B COMBINATIONS OF LOADS

3.22 COMBINATIONS OF LOADS

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

$$\begin{aligned} \text{Group (N)} = & \gamma[\beta_D \cdot D + \beta_L (L + I) + \beta_C CF + \beta_E E \\ & + \beta_B B + \beta_S SF + \beta_W W + \beta_{WL} WL \\ & + \beta_L \cdot LF + \beta_R (R + S + T) \\ & + \beta_{EQ} EQ + \beta_{ICE} ICE] \quad (3-10) \end{aligned}$$

where,

N	= group number;
γ	= load factor, see Table 3.22.1A;
β	= coefficient, see Table 3.22.1A;
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 pounds per linear foot;
LF	= longitudinal force from live load;
CF	= centrifugal force;
R	= rib shortening;
S	= shrinkage;
T	= temperature;
EQ	= earthquake;
SF	= stream flow pressure;
ICE	= ice pressure.

3.22.2 For service load design, the percentage of the basic unit stress for the various groups is given in Table 3.22.1A.

The loads and forces in each group shall be taken as appropriate from Articles 3.3 to 3.21. The maximum section required shall be used.

3.22.2 For load factor design, the gamma and beta factors given in Table 3.22.1A shall be used for designing structural members and foundations by the load factor concept.

TABLE 3.23.1 Distribution of Wheel Loads in Longitudinal Beams

Kind of Floor	Bridge Designed for One Traffic Lane	Bridge Designed for Two or more Traffic Lanes
Timber ^a Plank ^b	S/4.0	S/3.75
Nail laminated ^c 4" thick or multiple layer ^d floors over 5" thick	S/4.5	S/4.0
Nail laminated ^c 6" or more thick	S/5.0 If S exceeds 5' use footnote f.	S/4.25 If S exceeds 6.5' use footnote f.
Glued laminated ^e Panels on glued laminated stringers		
4" thick	S/4.5	S/4.0
6" or more thick	S/6.0 If S exceeds 6' use footnote f.	S/5.0 If S exceeds 7.5' use footnote f.
On steel stringers		
4" thick	S/4.5	S/4.0
6" or more thick	S/5.25 If S exceeds 5.5' use footnote f.	S/4.5 If S exceeds 7' use footnote f.
Concrete		
On steel I-Beam stringers ^g and prestressed concrete girders	S/7.0 If S exceeds 10' use footnote f.	S/5.5 If S exceeds 14' use footnote f.
On concrete T-Beams	S/6.5 If S exceeds 6' use footnote f.	S/6.0 If S exceeds 10' use footnote f.
On timber stringers	S/6.0 If S exceeds 6' use footnote f.	S/5.0 If S exceeds 10' use footnote f.
Concrete box girders ^h	S/8.0 If S exceeds 12' use footnote f.	S/7.0 If S exceeds 16' use footnote f.
On steel box girders	See Article 10.39.2	
On prestressed concrete spread box Beams	See Article 3.28	
Steel grid: (Less than 4" thick)	S/4.5	S/4.0
(4" or more)	S/6.0 If S exceeds 6' use footnote f.	S/5.0 If S exceeds 10.5' use footnote f.
Steel bridge Corrugated plank ⁱ (2" min. depth)	S/5.5	S/4.5

S = average stringer spacing in feet.

^aTimber dimensions shown are for nominal thickness.

^bPlank floors consist of pieces of lumber laid edge to edge with the wide faces bearing on the supports (see Article 16.3.11 - Division II).

^cNail laminated floors consist of pieces of lumber laid face to face with the narrow edges bearing on the supports, each piece being nailed to the preceding piece (see Article 16.3.12 - Division II).

^dMultiple layer floors consist of two or more layers of planks, each layer being laid at an angle to the other (see Article 16.3.11 - Division II).

^eGlued laminated panel floors consist of vertically glued laminated

members with the narrow edges of the laminations bearing on the supports (see Article 16.3.13 - Division II).

^fIn this case the load on each stringer shall be the reaction of the wheel loads, assuming the flooring between the stringers to act as a simple beam.

^g"Design of I-Beam Bridges" by N. M. Newmark - Proceedings, ASCE, March 1948.

^hThe sidewalk live load (see Article 3.14) shall be omitted for interior and exterior box girders designed in accordance with the wheel load distribution indicated herein.

ⁱDistribution factors for Steel Bridge Corrugated Plank set forth above are based substantially on the following reference:

Journal of Washington Academy of Sciences, Vol. 67, No. 2, 1977 "Wheel Load Distribution of Steel Bridge Plank," by Conrad P. Heins, Professor of Civil Engineering, University of Maryland.

These distribution factors were developed based on studies using 6" x 2" steel corrugated plank. The factors should yield safe results for other corrugated configurations provided primary bending stiffness is the same as or greater than the 6" x 2" corrugated plank used in the studies.

3.22.4 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, expected loads, service conditions, or materials of construction are different from those anticipated by the specifications.

3.22.5 Structures may be analyzed for an overload that is selected by the operating agency. Size and configuration of the overload, loading combinations, and load distribution will be consistent with procedures defined in permit policy of that agency. The load shall be applied in Group IB as defined in Table 3.221A. For all loadings less than H 20, Group 1A loading combination shall be used (see Article 3.5).

Part C DISTRIBUTION OF LOADS

3.23 DISTRIBUTION OF LOADS TO STRINGERS, LONGITUDINAL BEAMS, AND FLOOR BEAMS*

3.23.1 Position of Loads for Shear and Reactions

In calculating shears and reactions in transverse floor beams and longitudinal beams and stringers, each wheel shall be considered as a concentrated load and the lateral distribution of the wheel loads shall be as follows, regardless of their position along the span:

3.23.1.1 For determination of shears, the lateral distribution of dead and live loads shall be the same as that for bending moments (Article 3.23.2).

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

3.23.12 For determination of reactions, lateral distribution of dead loads shall be the same as that for bending moments (Article 3.23.2).

Lateral distribution of wheel loads shall be that produced by assuming the flooring to act as a simple span between stringers or beams.

3.23.2 Bending Moments in Stringers and Longitudinal Beams**

3.23.2 General

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

3.23.2.2 Interior Stringers and Beams

The live load bending moment for each interior stringer shall be determined by applying to the stringer the fraction of a wheel load (both front and rear) determined in Table 3.23.1.

3.23.2.3 Outside Roadway Stringers and Beams

3.23.2.3.1 Steel-Timber-Concrete T-Beams

The dead load considered as supported by the exterior roadway stringer or beam shall be that portion of the deck slab, appurtenant bolsters, etc. carried by the stringer or beam. Each stringer or beam shall be assumed to carry these loads between vertical planes located midway between its centerline and the centerline of the adjacent stringer or beam. Wearing surface loads shall be equally distributed to all roadway stringers or beams. Curb, railing and sidewalk loads (including sidewalk live load) placed on the roadway slab shall be equally distributed as follows:

Outside beams or stringers: For design of outside beams or stringers, loads shall be distributed equally between the exterior and not more than two adjacent interior beams or stringers.

Pier cantilevers: For design of pier cantilevers, the exterior beam or stringer shall be assumed to carry the loads between the outside edge of the roadway slab and halfway between the exterior and adjacent interior beams or stringers.

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

3.23.2.3.1.3 When the outside roadway beam or stringer supports sidewalk live load as well as normal roadway traffic live load and impact, the allowable stress in the beam or stringer may be increased 25 percent for the combination of dead load, sidewalk live load, traffic live load and impact, providing the beam is of no less carrying capacity than would be required if there were no sidewalk. In the case of a mountable sidewalk (6 inches or less in height) or median curb, a wheel load may be placed on the sidewalk or median as close as one foot to the face of rail or median joint. In this case, the allowable stress may be increased 25 percent for the combination of dead load, traffic live load and impact. When one of these combinations governs the design and the structure is to be designed by the Load Factor Method, 1.25 may be used as the Beta factor in place of 1.67.

**In view of the complexity of the theoretical analysis involved in the distribution of wheel loads to stringers, the empirical method herein described is authorized for the design of normal highway bridges.

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3.23.2.3.1.4 In no cases shall an exterior stringer have less carrying capacity than an interior stringer.

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

$$\frac{S}{5.5}$$

where, S = 6 feet or less and is the distance in feet between outside and adjacent interior stringers, and

$$\frac{S}{4.0 + 0.25S}$$

where, S is more than 6 feet and less than 14 feet. When S is 14 feet or more, use footnote f, Table 3.23.1.

3.23.2.3.2 Concrete Box Girders

3.23.2.3.2.1 The dead load supported by the exterior girder shall be determined in the same manner as for steel, timber, or concrete T-beams, as given in Article 3.23.2.3.1.

3.23.2.3.2.2 The factor for the wheel load distribution to the exterior girder shall be $W_e/7$, where W_e is the width of exterior girder which shall be taken as the top slab width, measured from the midpoint between girders to the outside edge of the slab. The cantilever dimension of any slab extending beyond the exterior girder shall preferably not exceed half the girder spacing.

3.23.2.3.3 Total capacity of Stringers and Beams

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

3.23.3 Bending Moments in Floor Beams (Transverse)

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

3.23.3.2 If longitudinal stringers are omitted and the floor is supported directly on floor beams, the beams shall be designed for loads determined in accordance with Table 3.23.3.1.

The flanges and stems of stemmed or channel sections are considered as separate rectangular components whose values are summed together to calculate “J”. Note that for “Rectangular Beams with Circular Voids” the value of “J” can usually be approximated by using the equation above for rectangular sections and neglecting the voids.

For Box-Section Beams:

$$J = \frac{2t_f(b-t)^2(d-t_f)^2}{bt + dt_f - t^2 - t_f^2}$$

where,

- b = the overall width of the box,
- d = the overall depth of the box,
- t = the thickness of either web,
- t_f = the thickness of either flange.

The formula assumes that both flanges are the same thickness and uses the thickness of only one flange. The same is true of the webs.

For preliminary design, the following values of K may be used:

Bridge Type	Beam Type	K
Multi-beam	Non-voided rectangular beams	0.7
	Rectangular beams with circular voids	0.8
	Box Section beams	1.0
	Channel, single-and multi-stemmed tee beams	2.2

3.23.4.4 The dead load considered as supported by the exterior beam shall be determined in the same manner as given in Article 3.23.2.3.1.1. Parapet and sidewalk loads shall be distributed over a width of not more than 10 feet from the outside face of the exterior beam. Wearing surfaces shall be equally distributed over all beams.

Median load shall be distributed over a width of not more than 10 feet on each side of the median centerline or joint.

3.24 DISTRIBUTION OF LOADS AND DESIGN OF CONCRETE SLABS*

3.24.1 Span Lengths (See Article 8.8)

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

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

- (a) Slabs monolithic with beams or slabs monolithic with walls without haunches and rigid top flange prestressed beams with top flange width to minimum thickness ratio less than 4.0. “S” shall be the clear span.
- (b) Slabs supported on steel stringers, or slabs supported on thin top flange prestressed beams with top flange width thickness ratio equal to or greater than 4.0. “S” shall be the distance between the edges of the top flange plus one-half of stringer top flange width.
- (c) Slabs supported on timber stringers. “S” shall be the clear span plus one-half thickness of stringer.

3.24.2 Edge Distance of Wheel Loads

3.24.2.1 In designing slabs, the centerline of a wheel load may be placed as close as one foot to the face of curb, or to the face of rail if curbs are not used. Combined dead load, live load and impact stresses shall be not greater than the allowable stresses.

3.24.2.2 In designing slabs and supporting members except stringers and girders with mountable (6 inch or less height) curbs or sidewalks, the centerline of a wheel load may be placed as close as one foot to the face of rail. Combined dead load, truck live load and impact stresses for this loading shall not be greater than 150 percent of the allowable stresses. In Load Factor Design, 1.0 may be used as the Beta factor in place of 1.67 for the design of deck slabs. Wheel loads shall not be applied on sidewalks protected by a traffic barrier.

*The slab distribution set forth herein is based substantially on the “Westergaard” theory. The following references are furnished concerning the subject of slab design.

Public Roads, March 1930, “Computation of Stresses in Bridge Slabs Due to Wheel Loads,” by H. M. Westergaard.

University of Illinois, Bulletin No. 303, “Solutions for Certain Rectangular Slabs Continuous over Flexible supports,” by Vernon P. Jensen, Bulletin 304, “A Distribution Procedure for the Analysis of Slabs Continuous over Flexible Beams,” by Nathan M. Newmark; Bulletin 315, “Moments in Simple Span Bridge Slabs with Stiffened Edges,” by Vernon P. Jensen; and Bulletin 346, “Highway Slab Bridges with Curbs; Laboratory Tests and Proposed Design Method.”

3.24.3 Bending Moment

The bending moment per foot width of slab shall be calculated according to methods given under Cases A and B, unless more exact methods are used considering tire contact area. The tire contact area needed for exact methods is given in Article 3.30.

In Cases A and B:

- S = effective span length, in feet as defined under "Span Lengths", Articles 3.24 and 8.8;
- E = width of slab in feet over which a wheel is distributed;
- P = load on one rear wheel of truck
- P₁₅ = 12,000 pounds for H15 loading;
- P₂₀ = 16,000 pounds for H20 loading;
- P₂₅ = 20,000 pounds for H25 loading.

3.24.3.1 Case A – Main Reinforcement Perpendicular to Traffic (Spans 2 to 24 Feet Inclusive)

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

HS 20 Loading:

$$\left(\frac{S+2}{32}\right)P_{20} = \text{Moment in foot pounds per foot -- width of slab} \quad (3-15)$$

HS 15 Loading:

$$\left(\frac{S+2}{32}\right)P_{15} = \text{Moment in foot pounds per foot -- width of slab} \quad (3-16)$$

H 25 Loading:

$$\left(\frac{S+2}{32}\right)P_{25} = \text{Moment in foot pounds per foot -- width of slab} \quad (3-16A)$$

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

3.24.3.2 Case B – Main Reinforcement Parallel to Traffic

For wheel loads, the distribution width, E, shall be (4+ 0.06S) but shall not exceed 7.0 feet. Lane loads are distributed over a width of 2E. Longitudinally reinforced shall be designed for the appropriate HS loading.

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

~~HS 20 Loading:~~

- ~~Spans up to and including 50 feet: LLM = 900S~~
- ~~foot pounds~~
- ~~Spans 50 feet to 100 feet: LLM = 1,000~~
- ~~(1.30S - 20.0)~~
- ~~foot pounds~~

~~HS 15 Loading:~~

- ~~Use ¾ of the values obtained from the formulas for HS 20 loading~~

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

3.24.4 Shear and Bond

Slabs (except continuous slab spans) designed for bending moment in accordance with the foregoing shall be considered satisfactory in bond and shear. Continuous slab spans shall be designed for shear and bond in accordance with the provisions of Section 8.

3.24.5 Cantilever Slabs

Overhangs

Overhangs (cantilever decks) for stringers (steel, concrete, prestressed) shall normally be limited to 0.3 of the beam/girder spacing.

3.24.5.1 Truck Loads

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

3.24.5.1.1 Case A – Reinforcement Perpendicular to Traffic

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

$$E = 0.8X + 3.75 \quad (3-17)$$

The moment per foot of slab shall be (P/E) X foot-pounds, in which X is the distance in feet from load to point of support.

3.24.5.1.2 Case B – Reinforcement Parallel to Traffic

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

$$E = 0.35X + 3.2, \text{ but shall not exceed } 7.0 \text{ feet} \quad (3-18)$$

The moment per foot of slab shall be (P/E) X foot-pounds.

3.24.5.2 Railing Loads

Railing loads shall be applied in accordance with Article 2.7. The effective length of slab resisting post loadings shall be equal to $E = 0.8X + 3.75$ feet where no parapet is used and equal to $E = 0.8X + 5.0$ feet where a parapet is used, where X is the distance in feet from the center of the post to the point under investigation. Railing and wheel loads shall not be applied simultaneously.

3.24.6 Slabs Supported on Four Sides

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

$$\text{For uniformly distributed load, } p = \frac{b^4}{a^4 + b^4} \quad (3-19)$$

$$\text{For concentrated load at center, } p = \frac{b^3}{a^3 + b^3} \quad (3-20)$$

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where,

- p = proportion of load carried by short span;
- a = length of short span of slab;
- b = length of long span of slab.

3.24.6.2 Where the length of the slab exceeds 1½ times its width, the entire load shall be carried by the transverse reinforcement.

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

3.24.7 Median Slabs

Roadway slabs supporting medians shall be designed for full live load with truck loadings so placed as to produce maximum stress. Where a roadway slab supports a raised median, it shall be designed for either live load or the median dead load, whichever is greater.

Median slabs which are not supported directly by the main roadway slab shall, if flush, be designed for full live load and their own dead load; if raised, the combined dead load, live load and impact stresses may be increased to 150 percent of the allowable stress.

3.24.8 Longitudinal Edge Beams

3.24.8.1 Edge beams shall be provided for all cast-in-place slabs having main reinforcement parallel to traffic. An edge beam shall be integral with the roadway slab and shall be added outside the vertical plane thru the face of a curb, or rail if no curb is used. The edge beam shall be designed to support fully its own dead weight and any superimposed loads and 0.2 of a lane of live load and its impact. The edge beam may consist of an extension of the roadway slab, an extension additionally reinforced, a reinforced beam deeper than the slab or a reinforced section of slab and curb.

3.24.8.2 The combined roadway slab and edge beam shall be designed to carry at least the total live and dead loads.

3.24.8.3 For continuous spans, the moment may be reduced by 20 percent unless a greater reduction results from a more exact analysis.

3.24.9 Unsupported Transverse Edges

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

3.24.10 Distribution Reinforcement

3.24.10.1 To provide for the lateral distribution of the concentrated live loads, reinforcement shall be placed transverse to the main steel reinforcement in the bottoms of all slabs except culvert or bridge slabs where the depth of fill over the slab exceeds two feet.

3.24.10.2 The amount of reinforcement shall be the percentage of the main reinforcement actually used as given by the following formulas:

For main reinforcement parallel to traffic,

$$\text{Percentage} = \frac{100}{\sqrt{S}} \quad \text{Maximum 50\%} \quad (3-21)$$

For main reinforcement perpendicular to traffic,

$$\text{Percentage} = \frac{220}{\sqrt{S}} \quad \text{Maximum 67\%} \quad (3-22)$$

Where S = span length in feet (c-c bearings for slab spans or as given in Article 3.24.1 for deck slabs). "Span" as used in this article refers to the span of a slab and not to that of any supporting beams/girders.

3.24.10.3 In simple spans, the computed amount of distribution reinforcement shall be placed in the middle half of the span in the outer quarters of the span, not less than 50 percent of the computed amount shall be used.

3.24.10.4 In continuous spans, the computed amount of distribution reinforcement shall be placed in the positive moment area, between points 0.2S from a free end and 0.2S from an intermediate support. In the remainder of the span, not less than 50 percent of the computed amount shall be used. In deck slabs, the distribution reinforcement near an exterior beam shall be placed in the same manner as that near an interior beam.

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3.24.10.5 The size of distribution reinforcing bars shall be selected so that the spacing will not be less than 4 ½", nor greater than 24" or twice the thickness of the slab, whichever is less.

3.24.10.6 Longitudinal bars shall be provided at hooks and bends (top and bottom) in main reinforcing bars of deck slabs.

3.24.10.7 In the tops of deck slabs, reinforcement shall be placed transverse to and under the main reinforcement, at a density of not less than 50 percent of that computed for distribution reinforcement.

3.24.10.8 In slab spans, reinforcement consisting of #3 bars at a maximum spacing of 12 inches c-c in both directions shall be placed in the tops of slabs. Where main reinforcement is used in the tops of slabs, this additional reinforcement shall be placed only in the direction transverse to the main reinforcement.

3.25 DISTRIBUTION OF WHEEL LOADS ON TIMBER FLOORING

For the calculation of bending moments in timber flooring each wheel load shall be distributed as follows.

3.25.1 Transverse Flooring

3.25.1.1 In the direction of flooring span, the wheel load shall be distributed over the width of tire as given in Article 3.30.

Normal to the direction of flooring span, the wheel load shall be distributed as follows:

Plank floor: the width of plank.

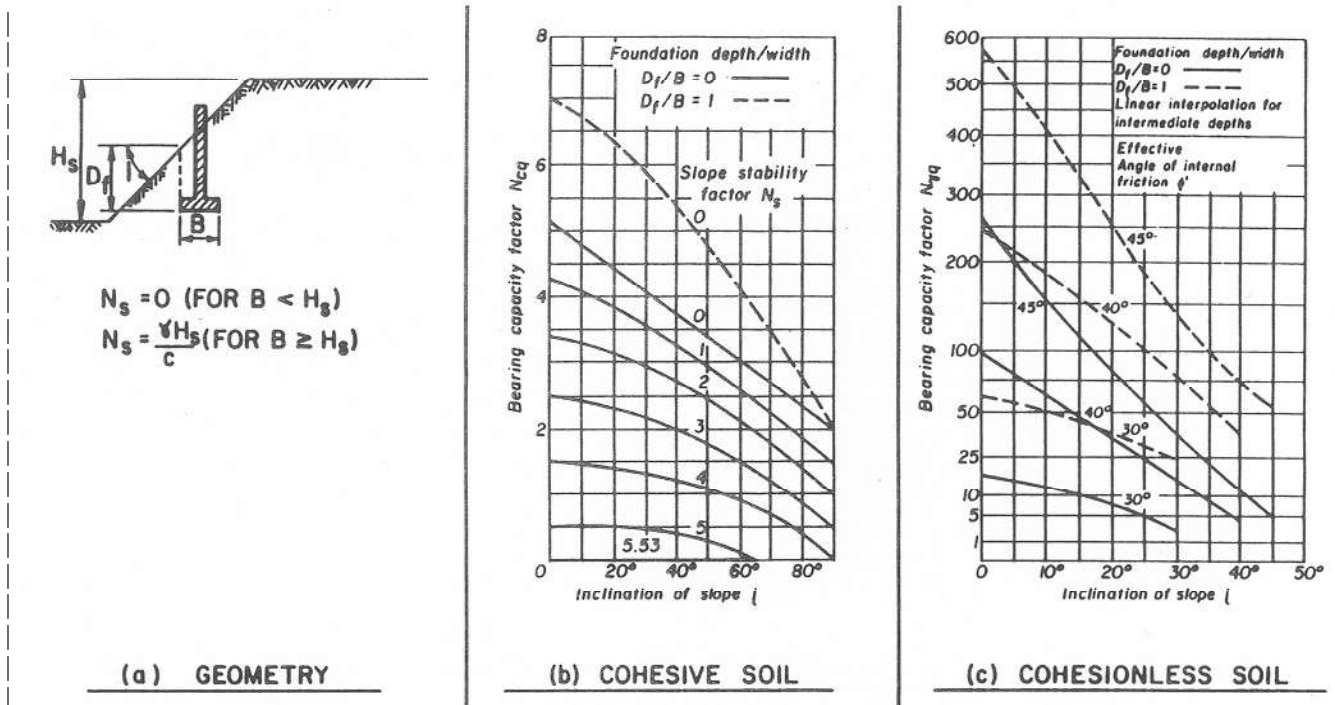


Figure 4.4.7.1.1.4A Modified Bearing Capacity Factors for Footing on Sloping Ground
Modified after Meyerhof (1957)

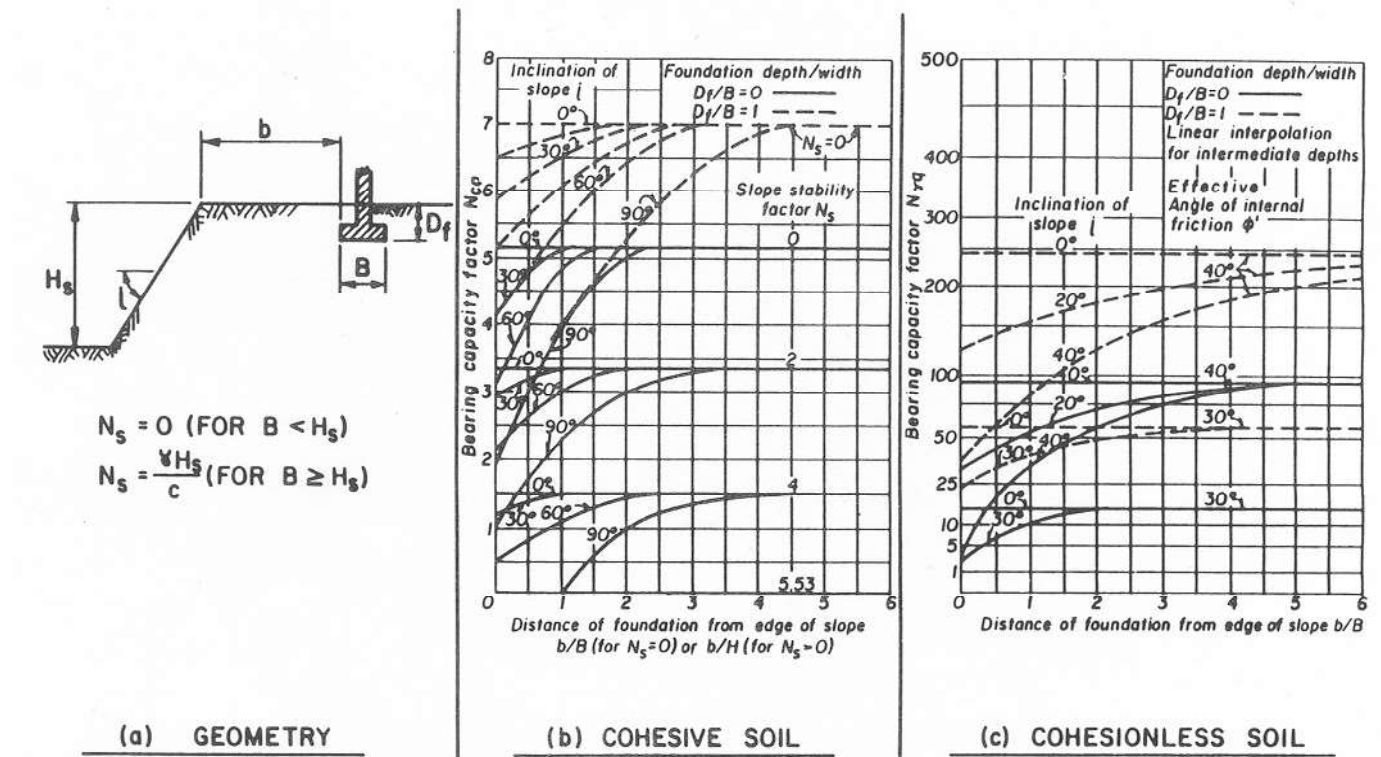


Figure 4.4.7.1.1.4B Modified Bearing Capacity Factors for Footing Adjacent Sloping Ground
Modified after Meyerhof (1957)

4.4.11.1.2 Isolated and Multiple Footing Reactions

When a single isolated footing supports a column, pier or wall the footing shall be assumed to act as a cantilever. When footings support more than one column, pier, or wall, the footing slab shall be designed for the actual conditions of continuity and restraint.

4.4.11.2 Moments

4.4.11.2.1 Critical Section

External moment on any section of a footing shall be determined by passing a vertical plane through the footing, and computing the moment of the forces acting over the entire area of the footing on one side of that vertical plane. The critical section for bending shall be taken at the face of the column, pier, or wall. In the case of columns that are not square or rectangular, the section shall be taken at the side of the concentric square of equivalent area. For footings under masonry walls, the critical section shall be taken halfway between the middle and edge of the wall. For footings under metallic column bases, the critical section shall be taken halfway between the column face and the edge of the metallic base.

4.4.11.2.2 Distribution of Reinforcement

Reinforcement of one-way and two-way square footings shall be distributed uniformly across the entire width of footing.

Reinforcement of two-way rectangular footings shall be distributed uniformly across the entire width of footing in the long direction. Reinforcement in the short direction shall be uniformly distributed across the length of footing. The total reinforcement in the short direction shall be computed as follows:

Let A_S = amount of reinforcement in short direction required by analysis.

β = ratio of long side to short side of footing.

$$A_{total} = A_S \times (\beta + 1) / 2$$

Compare A_{total} with minimum reinforcement required by Section 8.17.1 as follows:

Let A_{SC} = amount of reinforcement as calculated using CALTRANS equation.

If $A_{total} > A_{SC}$, use A_{total} as the amount of reinforcement required.

If $A_{total} < A_{sc}$, compare $4/3 A_S$ and A_{SC} and use smaller value as the amount of reinforcement required.

4.4.11.3 Shear

4.4.11.3.1 Critical Section

Computation of shear in footings, and location of critical section, shall be in accordance with Articles 8.15.5.6 or 8.16.6.6. Location of critical section shall be measured from the face of column, pier or wall, for footings supporting a column, pier, or wall. For footings supporting a column or pier with metallic base plates, the critical section shall be measured from the location defined in Article 4.4.11.2.

4.4.11.3.2 Footings on Piles or Drilled Shafts

For footings supported on piles, shear on the critical section shall be in accordance with the following:

(a) Entire reaction from any pile whose center is located on or outside the critical section shall be considered as producing shear on that section.

(b) Reaction from any pile whose center is located inside the critical section shall be considered as producing no shear on that section.

(c) Driving tolerance(s) for piles shall be considered when determining shear at the critical section.

Driving Tolerances for Steel and Concrete Piles*		
Pile Location/ Condition	Tolerance for Single Pile	Direction
Columns for bent caps	± 3"	About long axis of footing
Footings for column piers	± 6"	About both major axes
Footing for abutments, retaining walls & pier other than column piers	± 6"	About long axis of footing

*For driving tolerances of timber piles and other information, see VDOT *Road and Bridge Specifications*, Section 403 and Table IV-1.

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4.4.11.4 Development of Reinforcement

4.4.11.4.1 Development Length

Computation of development length of reinforcement in footings shall be in accordance with Articles 8.24 through 8.32.

4.4.11.4.2 Critical Section

Critical sections for the development of reinforcement shall be assumed at the same locations as defined in Articles 4.4.11.2 and at all other vertical planes where changes in section or reinforcement occur. See also Article 8.24.1.5

4.4.11.5 Transfer of Force at Base of Column

4.4.11.5.1 Transfer of Force

All forces and moments applied at the base of column or pier shall be transferred to the top of footing by bearing on concrete and by reinforcement.

4.4.11.5.2 Lateral Forces

Lateral forces shall be transferred to supporting footing in accordance with shear-transfer provisions of Articles 8.15.5.4 and 8.16.6.4.

4.5.14.2 Concrete Piles

A concrete pile foundation shall consider that deterioration of concrete piles can occur due to sulfates in soil, ground water, or sea water; chlorides in soils and chemical wastes; acidic ground water and organic acids. Laboratory testing of soil and ground water samples for sulfates and ph is usually sufficient to assess pile deterioration potential. A full chemical analysis of soil and ground water samples is recommended when chemical wastes are suspected. Methods of protecting concrete piling can include dense impermeable concrete, sulfate resisting portland cement, minimum cover requirements for reinforcing steel, and use of epoxies, resins, or other protective coatings.

4.5.14.3 Timber Piles

A timber pile foundation shall consider that deterioration of timber piles can occur due to decay from wetting and drying cycles or from insects or marine borers. Methods of protecting timber piling include pressure treating with creosote or other wood preservers.

4.5.15 Spacing, Clearances, and Embedment

4.5.15.1 Pile Footings

4.5.15.1.1 Pile Spacing

The spacing of concrete piles shall be not be less than three times the diameter or side dimension of the piles. The spacing of steel H friction piles shall be not less than 3.5 times the normal size of the piles. The spacing of timber piles and steel H point-bearing piles shall not be less than 2'-6".

When the tops of foundation piles are incorporated into a concrete footing, the distance from the side of any pile to the nearest edge of the footing shall be not less than 9 inches.

4.5.15.1.2 Minimum Projection into Cap

The tops of timber or steel H-piles shall project not less than 12 inches into footings. When piles are subjected to intermittent uplift under any loading condition, are subjected to scour at street crossings, or are in any areas where Seismic Performance Category B is applied, they shall project not less than 18 inches into the footing or have a means of anchoring the pile to the footing or both. Cast-in-place or precast piles having reinforcement projecting into footings shall be embedded a minimum of 6 inches into the concrete.

4.5.15.2 Bent Caps

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

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4.5.16 Precast Concrete Piles

4.5.16.1 Size and Shape

Precast concrete piles shall be of approved size and shape but may be either of uniform section or tapered. In general, tapered piling shall not be used for trestle construction except for the 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.

4.5.16.2 Minimum Area

In general, concrete piles shall have a cross-sectional area, measured above the taper, of not less than 98 square inches. In saltwater a minimum cross-sectional area of 140 square inches shall be used. If a square section is employed, the corners shall be chamfered at least 1 inch.

4.5.16.3 Minimum Diameter of Tapered Piles

The diameter of tapered piles measured at the point shall be not less than 8 inches. In all cases the diameter shall be considered as the least dimension through the center.

4.5.16.4 Driving Points

Piles preferably shall be cast with a driving point and, for hard driving, preferably shall be shod with a metal shoe of approved pattern.

4.5.16.5 Vertical Reinforcement

Vertical reinforcement shall consist of not less than four bars spaced uniformly around the perimeter of the pile, 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 amount of reinforcement shall be at least 1 ½ percent of the total section measured above the taper.

4.5.16.6 Spiral Reinforcement

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 the pile shall have five turns of spiral winding at 1-inch pitch. For the remainder of the pile, the lateral reinforcement shall be a No. 5 gage spiral with not more than 6-inch pitch, or ¼-inch round hoops spaced on not more than 6-inch centers.

4.5.16.7 Reinforcement Cover

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

4.5.16.8 Splices

Piles may be spliced provided that the splice develops the full strength of the pile. Splices should be detailed on the contract plans. Any alternative method of splicing that provides equal results may be considered for approval.

4.5.16.9 Handling Stresses

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

4.5.17 Cast-in-Place Concrete Piles

4.5.17.1 Materials

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

4.5.17.2 Shape

Cast-in place concrete piles may have a uniform cross-section or may be tapered over any portion.

4.5.17.3 Minimum Area

The minimum area at the butt of a pile shall be 100 inches and the minimum diameter at the tip of the pile shall be 8 inches. Above the butt or taper, the minimum size shall be specified for precast piles.

4.5.17.4 General Reinforcement Requirements

Cast-in-place piles, carrying axial loads only where the possibility of lateral forces being applied to the piles is insignificant, need not be reinforced where the soil provides adequate lateral support. These portions of cast-in-place concrete piles that are not supported laterally shall be designed as reinforced concrete columns in accordance with Articles 8.15.4 and 8.16.4, and the reinforcing steel shall extend 10 feet below the plane where the soil provides adequate lateral restraint. Where the shell is smooth pipe and more than 0.12 inch in thickness, it may be considered as load carrying in the absence of corrosion. Where the shell

is corrugated and is at least 0.075 inch in thickness, it may be considered as providing confinement in the absence of corrosion.

4.5.17.5 Reinforcement into Superstructure

Sufficient reinforcement shall be provided at the junction of the pile with the superstructure to make a suitable connection. The embedment of the reinforcement into the cap shall be as specified for precast piles.

4.5.17.6 Shell Requirements

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

4.5.17.7 Splices

Piles may be spliced provided that the splice develops the full strength of the pile. Splices should be detailed on the contract plans. Any alternative method of splicing providing equal results may be considered for approval.

4.5.17.8 Reinforcement Cover

The reinforcement shall be placed a clear distance of not less than 2 inches from the cased or uncased sides. When piles are in corrosive or marine environments, or when concrete is placed by the water or slurry displacement method, the clear distance shall be not less than 3 inches for uncased piles and piles with shells not sufficiently corrosion resistant.

4.5.17.9

Cast-in-place piles shall not be used in pile bents or in soils which may induce shell collapse during driving.

4.5.18 Steel H-Piles

4.5.18.1 Metal Thickness

Steel piles shall have a minimum thickness of web of 0.400 inch. Splice plates shall be not less than $\frac{3}{8}$ in. thick.

4.5.18.2 Splices

Piles shall be spliced to develop the net section of the pile. The flanges and web shall be either spliced by butt welding or with plates that are welded, riveted, or bolted. Splices shall be detailed on the contract plans. Prefabri-

Section 6 CULVERTS

6.1 CULVERT LOCATION, LENGTH, AND WATERWAY OPENINGS

Recommendations on culvert location, length, and waterway openings are given in the AASHTO *Guide on Hydraulic Design of Culverts*.

6.2 DEAD LOADS

Vertical and horizontal earth pressures on culverts may be computed by recognized or appropriately documented analytical techniques based on the principles of soil mechanics and soil structure interaction, or design pressures shall be calculated as being the result of an equivalent fluid weight as follows.

6.2.1 Culvert in Trench, or culvert untrenched on yielding foundation

- A. Rigid culverts except reinforced concrete boxes:
 - (1) For vertical earth pressure----- 120 pcf
For lateral earth pressure-----30 pcf
 - (2) For vertical earth pressure----- 120 pcf
For lateral earth pressure----- 120 pcf
- B. Reinforced concrete boxes:
 - (1) For maximum positive moments
vertical earth pressure ---- 120 pcf
lateral earth pressure-----15 pcf
 - (2) For maximum negative moments
vertical earth pressure ---- 120 pcf
lateral earth pressure-----30 pcf
- C. Flexible culverts:
 - For vertical earth pressure ---- 120 pcf
 - For lateral earth pressure----- 120 pcf

When concrete pipe culverts are designed by the Indirect Design Method of Article 17.4.5, the design lateral earth pressure shall be determined using the procedures given in Article 17.4.5.2.1 for embankment installations and in Article 17.4.5.2.2 for trench installations.

6.2.2 Culvert untrenched on unyielding foundation

A special analysis is required.

6.3 FOOTINGS

Footings for culverts shall be carried to an elevation sufficient to secure a firm foundation, or a heavy rein-

forced floor shall be used to distribute the pressure over the entire horizontal area of the structure. In any location subject to erosion, aprons or cutoff 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.

6.4 DISTRIBUTION OF WHEEL LOADS THROUGH EARTH FILLS

6.4.1 When the depth of fill is 2 feet or more, concentrated loads shall be considered as uniformly distributed over a square with sides equal to $1\frac{3}{4}$ times the depth of fill.

6.4.2 When such areas from several concentrations overlap, the total load shall be uniformly distributed over the area defined by the outside limits of the individual areas, but the total width of distribution shall not exceed the total width of the supporting slab. For single spans, the effect of live load may be neglected when the depth of fill is more than 8 feet and exceeds the span length; for multiple spans it may be neglected when the depth of fill exceeds the distance between faces of end supports or abutments. When the depth of fill is less than 2 feet the wheel load shall be distributed as in slabs with concentrated loads. When the calculated live load and impact moment in concrete slabs, based on the distribution of the wheel load through earth fills, exceeds the live load and impact moment calculated according to Article 3.24, the latter moment shall be used.

6.5 DISTRIBUTION REINFORCEMENT

Where the depth of fill exceeds 2 feet, reinforcement to provide for the lateral distribution of concentrated loads is not required.

6.6 DESIGN

For culvert design guidelines, see Section 17.

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ous soffit of concrete box superstructure sections. These walls are integral with the superstructure and must also be designed for the superstructure moments which develop from live loads and erection conditions.

7.3.1.3 Bent Piers

Bent type piers consist of two or more transversely spaced columns of various solid cross sections, and these types are designed for frame action relative to forces acting about the strong axis of the pier. They are usually fixed at the base of the pier and are either integral with the superstructure or with a pier cap at the top. The columns may be supported on a spread-or pile supported footing, or a solid wall shaft, or they may be extensions of the piles or shaft above the ground line.

7.3.1.4 Single-Column Piers

Single-column piers, often referred to as "T" or "Hammerhead" piers, are usually supported at the base by a spread-or pile supported footing, and may be either integral with, or provide independent support for, the superstructure. Their cross section can be of various shapes and the column can be prismatic or flared to form the pier cap or to blend with the sectional configuration of the superstructure cross section. This type pier can avoid the complexities of skewed supports if integrally framed into the superstructure and their appearance reduces the massiveness often associated with superstructures.

7.3.2 Pier Protection

7.3.2.1 Collision

Where the possibility of collision exists from highway or river traffic, an appropriate risk analysis should be made to determine the degree of impact resistance to be provided and/or the appropriate protection system.

7.3.2.2 Collision Walls

Piers having less than 25' horizontal clearance from the centerline of a main or passing railroad track shall be of heavy construction or shall be protected by a reinforced concrete collision wall extending to not less than 6' above top of rail for piers 12'-25' from the centerline and 12' above top of rail for piers less than 12' from the centerline. Collision walls shall extend to at least 4' below the lowest surrounding grade. When two or more light columns compose a pier, a wall at least 12' long and 2'-6" thick shall connect the columns and extend 1' past the end columns. When a pier consists of a single

column with a stem less than 3' in width and 12' in length, it shall be protected by a crash wall parallel to the track. The wall shall be at least 2'-6" thick and extend parallel to the track for a distance of at least 6' from both sides of the column centerline. In general, the face of a collision wall shall extend a distance of at least 6' beyond the face of the column(s) on the side adjacent to the track and shall be anchored to the column(s) and footings with adequate steel reinforcement. When economically feasible, piers should be located at least 25' from the centerline of track to avoid the need for collision walls.

7.3.2.3 Scour

The scour potential must be determined and the design must be developed to minimize failure from this condition.

7.3.2.4 Facing

Where appropriate, the pier nose should be designed to effectively break up or deflect floating ice or drift. In these situations, pier life can be extended by facing the nosing with steel plates or angles, and by facing the pier with granite.

7.4 TUBULAR PIERS

7.4.1 Materials

Tubular piers of hollow core section may be of steel, reinforced concrete or prestressed concrete, of such cross section to support the forces and moments acting on the elements.

7.4.2 Configuration

The configuration can be as described in Article 7.3.1 and, because of their vulnerability to lateral loadings, shall be of sufficient wall thickness to sustain the forces and moments for all loading situations as are appropriate. Prismatic configurations may be sectionally precast or prestressed as erected.

7.5 ABUTMENTS

7.5.1 Abutment Types

7.5.1.1 Stub Abutment

Stub abutments are located at or near the top of approach fills, with a backwall depth sufficient to accommodate the structure depth and bearings which sit on the bearing seat.

7.5.1.2 Partial-Depth Abutment

Partial-depth abutments are located approximately at mid-depth of the front slope of the approach embankment. The higher backwall and wingwalls may retain fill material, or the embankment slope may continue behind the backwall. In the latter case, a structural approach slab or end span design must bridge the space over the fill slope, and curtain walls are provided to close off the open area. Inspection access should be provided for this situation.

7.5.1.3 full-Depth Abutment

Full-depth abutments are located at the approximate front toe of the approach embankment, restricting the opening under the structure.

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the plans. The requirements for f'_c shall be based on tests of cylinders made and tested in accordance with Section 4 – Division II.

Classes of concrete and corresponding compressive strengths for use in design shall be selected according to the table shown below.

Concrete Class	General Use	Min.Conc. Compressive Strength (f'_c) psi
A5	Prestressed members	5000-10,000
A4	Superstructures, (incl. sidewalks, rails, parapets, terminal walls, medians and median barriers); box culverts; rigid frames; reinforced substructures when needed for strength.	4000
A3	Reinforced substructures	3000
B2	Massive or lightly reinforced substructures	2200
C1	Massive unreinforced substructures; bag rip-rap.	1500
T3	Tremie seal	3000

8.3 REINFORCEMENT

8.3.1 The yield strength or grade of reinforcement shall be shown on the plans.

8.3.2 Reinforcement to be welded shall be indicated on the plans and the welding procedure to be used shall be specified.

8.3.3 Designs shall not use a yield strength, f_y , in excess of 60,000 psi.

8.3.4 Deformed reinforcement shall be used except that plain bars of smooth wire may be used for spirals and ties.

8.3.5 Deformed reinforcement shall conform to the specifications listed in Division II, Section 5, except that, for reinforcing bars, the yield strength and tensile strength shall correspond to that determined by tests on full-sized bars.

8.3.6 Reinforcing Steel Bar Lengths

8.3.6.1 The maximum length of any type bar, coated or uncoated, shall be 60'. Bar lengths greater than 40' will require oversized vehicles for hauling. Therefore, the designer should take into account the total number of bars in excess of 40' to justify using longer bars.

8.3.6.2 Bars in deck slabs may be detailed as one-piece bars (up to 60' fabrication length) with a note in the reinforcing steel schedule as follows: "If fabrication of deck slab bar(s) is not possible for length detailed and multiple bars are required, bars shall have the least number of Class B splices possible. Splices shall be located approximately at points of contraflexure and splices in alternate bars shall be located in different bays."

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Part B ANALYSIS

8.4 GENERAL

All members of continuous and rigid frame structures shall be designed for the maximum effects of the loads specified in Articles 3.2 through 3.22 as determined by the theory of elastic analysis.

8.5 EXPANSION AND CONTRACTION

8.5.1 In general, provisions for temperature changes shall be made in simple spans when the span length exceeds 40 feet.

8.5.2 In continuous bridges, the design shall provide for thermal stresses or for the accommodation of thermal movement with rockers, sliding plates, elastomeric pads, or other means.

8.5.3 The coefficient of thermal expansion and contraction for normal weight concrete may be taken as 0.000006 per deg F.

8.5.4 The coefficient of shrinkage for normal weight concrete may be taken as 0.0002.

8.5.5 Thermal and shrinkage coefficients for light-weight concrete shall be determined for the type of light-weight aggregate used.

8.6 STIFFNESS

8.6.1 Any reasonable assumptions may be adopted for computing the relative flexural torsional stiffnesses of continuous and rigid frame members. The assumptions made shall be consistent throughout the analysis.

8.6.2 The effect of haunches shall be considered both in determining moments and in design of members.

8.7 MODULUS OF ELASTICITY AND POISSON'S RATIO

8.7.1 The modulus of elasticity, E_c , for concrete may be taken as $w_c^{1.5} 33 \sqrt{f'_c}$ in psi for values of w_c between 90 and 155 pounds per cubic foot. For normal weight concrete ($w_c=145$ pcf), E_c may be considered as $57,000 \sqrt{f'_c}$.

8.7.2 The modulus of elasticity, E_s , for non-prestressed steel reinforcement may be taken as 29,000,000 psi.

8.7.3 Poisson's ratio may be assumed as 0.2.

8.8 SPAN LENGTH

8.8.1 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.

8.14.3.6 Walls exceeding 8 feet in height on filled spandrel arches shall be laterally supported by transverse diaphragms or counterforts with a slope greater than 45 degrees with the vertical to reduce transverse stresses in the arch barrel. The top of the arch barrel and interior faces of the spandrel walls shall be waterproofed and a drainage system provided for the fill.

**8.15 SERVICE LOAD DESIGN METHOD
(ALLOWABLE STRESS DESIGN)**

8.15.1 General Requirements

8.15.1.1 Service load stresses shall not exceed the values given in Article 8.15.1.

8.15.1.2 Development and splices of reinforcement shall be as required in Articles 8.24 through 8.32.

8.15.2 Allowable Stresses

8.15.2.1 Concrete

Stresses in concrete shall not exceed the following:

8.15.2.1.1 Flexure

Extreme fiber stress in compression, f_c	0.40 f'_c
Extreme fiber stress in tension for plain concrete, f_t	0.21 f_c

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

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

8.15.2.1.2 Shear

For detailed summary of allowable shear stress, v_c , see Article 8.15.5.2.

8.15.2.1.3 Bearing Stress

The bearing stress, f_b , on load area shall not exceed 0.30 f'_c .

When the supporting surface is wider on all sides than the loaded area, the allowable bearing stress on the loaded area may be multiplied by $\sqrt{A_2/A_1}$, but not by more than 2.

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 the 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.

When the loaded area is subjected to high-edge stresses due to deflection or eccentric loading, the allowable bearing stress on the loaded area, including any increase due to the supporting surface being larger than the loaded area, shall be multiplied by a factor of 0.75.

8.15.2.2 Reinforcement

The tensile stress in the reinforcement, f_s , shall not exceed the following:

Grade 40 reinforcement	20,000 psi
Grade 60 reinforcement	24,000 psi

In straight reinforcement, the range between the maximum tensile stress and the minimum stress caused by live load plus impact shall not exceed the value given in Article 8.16.8.3. Bends in primary reinforcement shall be avoided in regions of high-stress range.

All structures shall be designed using Grade 60 reinforcement and this shall be noted on the plans.

8.15.3 Flexure

8.15.3.1 For the investigation of stresses at service loads, the straight-line theory of stress and strain in flexure shall be used with the following assumptions.

8.15.3.2 The strain in reinforcement and concrete is directly proportional to the distance from the neutral axis, except that for deep flexural members with overall depth to span ratios greater than $2/5$ for continuous spans and $4/5$ for simple spans, a nonlinear distribution of strain shall be considered.

8.15.3.3 In reinforced concrete members, concrete resists no tension.

8.15.3.4 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.

8.15.3.5 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.

8.15.4 Compression Members

The combined flexural and axial load capacity of compression members shall be taken as 35 percent of that

computed in accordance with the provisions of Article 8.16.4. Slenderness effects shall be included according to the requirements of Article 8.16.5. The term P_u in Equation (8-41) shall be replaced by 2.5 times the design axial load. In using the provisions of Article 8.16.4 and 8.16.5, ϕ shall be taken as 1.0.

Round columns are to be designed as tied columns with continuous ties (#3 reinforcing bars minimum) or welded wire fabric unless special circumstances warrant the use of spirally reinforced columns.

8.15.5 Shear

8.15.5.1 Shear Stress

8.15.5.1.1 Design shear stress, v , shall be computed by:

$$v = \frac{V}{b_w d} \quad (8-3)$$

where V is design shear force at section considered, b_w is the width of web, and d is the distance from the extreme compression fiber to the centroid of the longitudinal tension reinforcement. Whenever applicable, effects of torsion* shall be included.

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

8.15.5.1.3 For tapered webs, b_w shall be the average width or 1.2 times the minimum width, whichever is smaller.

8.15.5.1.4 When the reaction, in the direction of the applied shear, introduces compression into the end regions of a member, sections located less than a distance d from the face of support may be designed for the same shear, V , as that computed at a distance d . An exception occurs when major concentrated loads are imposed between that point and the face of support. In that case sections closer than d to the support shall be designed for V at distance d plus the major concentrated loads.

8.15.5.2 Shear Stress Carried by Concrete

8.15.5.2.1 Shear in Beams and One-Way Slabs and Footings

For members subject to shear and flexure only, the allowable shear stress carried by the concrete v_c , may be

taken as $0.95\sqrt{f'_c}$. A more detailed calculation of the allowable shear stress can be made using:

$$v_c = 0.9\sqrt{f'_c} + 1,100\rho_w\left(\frac{Vd}{M}\right) \leq 1.6\sqrt{f'_c} \quad (8-4)$$

Note:

- (a) M is the design moment occurring simultaneously with V at the section being considered.
- (b) The quantity Vd/M shall not be taken greater than 1.0.

8.15.5.2.2 Shear in Compression Members

For members subject to axial compression, the allowable shear stress carried by the concrete, v_c , may be taken as $0.95\sqrt{f'_c}$. A more detailed calculation can be made using:

$$v_c = 0.9\left(1 + 0.0006\frac{N}{A_g}\right)\sqrt{f'_c} \quad (8-5)$$

The quantity N/A_g shall be expressed in pounds per square inch

8.15.5.2.3 Shear in Tension Members

For members subject to axial tension, shear reinforcement shall be designed to carry total shear, unless a more detailed calculation is made using

$$v_c = 0.9\left(1 + 0.004\frac{N}{A_g}\right)\sqrt{f'_c} \quad (8-6)$$

Note:

- (a) N is negative for tension.
- (b) The quantity N/A_g shall be expressed in pounds per square inch.

8.15.5.2.4 Shear in Lightweight Concrete

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

- (a) When f_{ct} is specified, the shear stress v_c , shall be modified by substituting $f_{ct}/6.7$ for $\sqrt{f'_c}$ but the value of $f_{ct}/6.7$ used shall not exceed $\sqrt{f'_c}$.
- (b) When f_{ct} is not specified, the shear stress, v_c , shall be multiplied by 0.75 for "all-lightweight" concrete, and

*The design criteria for combined torsion and shear given in "Building Code Requirements for Reinforced Concrete"—American Concrete Institute 318 Bulletin may be used.

(a) Beam action for the slab or footing, with a critical section extending in a plane across the entire width and located at a distance d from the face of the concentrated load or reaction area for spread footings and at a distance $d/2$ for pile foundations. For this condition, the slab or footing shall be designed in accordance with Articles 8.15.5.1 through 8.15.5.3, except at footings supported on piles, the shear on the critical section (including driving tolerances) shall be determined in accordance with Article 4.4.11.3.

(b) Two-way action for the slab or footing, with a critical section perpendicular to the plane of the member and located so that its perimeter b_o is a minimum, but not closer than $d/2$ to the perimeter of the concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with Articles 8.15.5.6.2 and 8.15.5.6.3.

8.15.5.6.2 Design shear stress, v , shall be computed by:

$$v = \frac{V}{b_o d} \quad (8-12)$$

where V and b_o shall be taken at the critical section defined in 8.15.5.6.1(b).

8.15.5.6.3 Design shear stress, v , shall not exceed v_c given by Equation (8-13) unless shear reinforcement is provided in accordance with Article 8.15.5.6.4.

$$v_c = \left(0.8 + \frac{2}{\beta_c} \right) \sqrt{f'_c} \leq 1.8 \sqrt{f'_c} \quad (8-13)$$

β_c is the ratio of long side to short side of concentrated load or reaction area.

8.15.5.6.4 Shear reinforcement consisting of bars or wires may be used in slabs and footings in accordance with the following provisions:

- (a) Shear stresses computed by Equation (8-12) shall be investigated at the critical section defined in 8.15.5.6.1(b) and at successive sections more distant from support.
- (b) Shear stress V_c at any section shall not exceed $0.9 \sqrt{f'_c}$ and v shall not exceed $3 \sqrt{f'_c}$.
- (c) Where v exceeds $0.9 \sqrt{f'_c}$, shear reinforcement shall be provided in accordance with Article 8.15.5.3.

8.15.5.7 Special Provisions for Slabs of Box Culverts

For slabs of box culverts under 2 feet or more fill, shear stress v_c may be computed by:

$$v_c = \sqrt{f'_c} + 2,200 \rho \left(\frac{Vd}{M} \right) \quad (8-14)$$

but v_c shall not exceed $1.8 \sqrt{f'_c}$. For single cell box culverts only, v_c for slabs monolithic with walls need not be taken less than $1.4 \sqrt{f'_c}$ and v_c for slabs simply supported need not be taken less than $1.2 \sqrt{f'_c}$. The quantity Vd/M shall not be taken greater than 1.0 where M is the moment occurring simultaneously with V at the section considered. For slabs of box culverts under less than 2 feet of fill, applicable provisions of Articles 3.24 and 6.4 should be used.

8.15.5.8 Special Provisions for Brackets and Corbels*

8.15.5.8.1 Provisions of paragraph 8.15.5.8 shall apply to brackets and corbels with a shear span-to-depth ratio a_v/d not greater than unity, and subject to a horizontal tensile force N_c not larger than V . Distance d shall be measured at the face of support.

8.15.5.8.2 Depth at outside edge of bearing area shall not be less than $0.5d$.

8.15.5.8.3 The section at the face of support shall be designed to resist simultaneously a shear V , a moment $[Va_v + N_c(h - d)]$, and a horizontal tensile force N_c . Distance h shall be measured at the face of support.

- (a) Design of shear-friction reinforcement, A_{vf} , to resist shear, V , shall be in accordance with Article 8.15.5.4. For normal weight concrete, shear stress v shall not exceed $0.09 f'_c$ nor 360 psi. For all "light-weight" or sand-lightweight" concrete, shear stress v shall not exceed $(0.09 - 0.03 a_v/d) f'_c$ nor $(360 - 126 a_v/d)$ psi.
- (b) Reinforcement A_f to resist moment $[Va_v + N_c(h - d)]$ shall be computed in accordance with Articles 8.15.2 and 8.15.3.
- (c) Reinforcement A_n to resist tensile force N_c shall be computed by $A_n = N_c/f_s$. Tensile force N_c shall not be taken less than $0.2V$ unless special provisions are made to avoid tensile forces.
- (d) Area of primary tension reinforcement, A_s , shall be made equal to the greater of $(A_f + A_n)$, or $2A_{vf}/3 + A_n$.

8.15.5.8.4 Closed stirrups or ties parallel to A_s , with a total area A_h not less than $0.5(A_s - A_n)$, shall be uni-

*These provisions do not apply to beam ledges. The PCA publication, "Notes on ACI 318-83," contains an example design of beam ledges - Part 16, example 16-3.

load or reaction area for spread footings and at a distance $d/2$ for pile foundations. For this condition, the slab or footing shall be designed in accordance with Articles 8.16.6.1 through 8.16.6.3, except at footings supported on piles, the shear on the critical section (including driving tolerances) shall be determined in accordance with Article 4.4.11.3.

(b) Two-way action for the slab or footing, with a critical section perpendicular to the plane of the member and located so that its perimeter b_o is a minimum, but need not approach closer than $d/2$ to the perimeter of the concentrated load or reaction area. For this condition, the slab or footings shall be designed in accordance with Articles 8.16.6.2 and 8.16.6.3.

8.16.6.6.2 Design of slab or footing for two-way action shall be based on Equation (8-46), where shear strength V_n shall not be taken greater than shear strength V_c given by Equation (8-58), unless shear reinforcement is provided in accordance with Article 8.16.6.6.3.

$$V_c = \left(2 + \frac{4}{\beta_c} \right) \sqrt{f'_c} b_o d \leq 4 \sqrt{f'_c} b_o d \quad (8-58)$$

β_c is the ratio of long side to short side of concentrated load or reaction area, and b_o is the perimeter of the critical section defined in Article 8.16.6.6.1(b).

8.16.6.6.3 Shear reinforcement consisting of bars or wires may be used in slabs and footings in accordance with the following provisions:

- (a) Shear strength V_n shall be computed by Equation (8-47), where shear strength V_c shall be in accordance with paragraph (d) and shear strength V_s shall be in accordance with paragraph (e).
- (b) Shear strength shall be investigated at the critical section defined in 8.16.6.6.1(b), and at successive sections more distant from the support.
- (c) Shear strength V_n shall not be taken greater than $6 \sqrt{f'_c} b_o d$, where b_o is the perimeter of the critical section defined in paragraph (b).
- (d) Shear strength V_c at any section shall not be taken greater than $2 \sqrt{f'_c} b_o d$, where b_o is the perimeter of the critical section defined in paragraph (b).
- (e) Where the factored shear force V_u exceeds the shear strength ϕV_c as given in paragraph (d), the required area A_v and shear strength V_s of shear reinforcement shall be calculated in accordance with Article 8.16.6.3.

*These provisions do not apply to beam edges. The PCA publication *Notes on ACI 318.83 contains an example design of beam ledges—Part 16, example 16-3.

8.16.6.7 Special Provisions for Slabs of Box Culverts

8.16.6.7.1 For slabs of box culverts under 2 feet or more fill, shear strength V_c may be computed by:

$$V_c = \left(2.14 \sqrt{f'_c} + 4,600 \rho \frac{V_u d}{M_u} \right) b d \quad (8-59)$$

but V_c shall not exceed $4 \sqrt{f'_c} b d$. For single cell box culverts only, V_c for slabs monolithic with walls need not be taken less than $3 \sqrt{f'_c} b d$, and V_c for slabs simply supported need not be taken less than $2.5 \sqrt{f'_c} b d$. The quantity $V_u d / M_u$ shall not be taken greater than 1.0 where M_u is the factored moment occurring simultaneously with V_u at the section considered. For slabs of box culverts under less than 2 feet of fill, applicable provisions of Articles 3.24 and 6.4 should be used.

8.16.6.8 Special Provisions for Brackets and Corbels*

8.16.6.8.1 Provisions of Article 8.16.6.8 shall apply to brackets and corbels with a shear span-to-depth ratio a_v/d not greater than unity, and subject to a horizontal tensile force N_{uc} not larger than V_u . Distance d shall be measured at the face of support.

8.16.6.8.2 Depth at the outside edge of bearing area shall not be less than $0.5d$.

8.16.6.8.3 The section at the face of the support shall be designed to resist simultaneously a shear V_u , a moment ($V_u a_v + N_{uc} (h - d)$), and a horizontal tensile force N_{uc} . Distance h shall be measured at the face of support.

- (a) In all design calculations in accordance with Article 8.16.6.8, the strength reduction factor ϕ shall be taken equal to 0.85.
- (b) Design of shear-friction reinforcement A_{vf} to resist shear V_u shall be in accordance with Article 8.16.6.4. For normal weight concrete, shear strength V_n shall not be taken greater than $0.2 f'_c b_w d$ nor $800 b_w d$ in pounds. For “all lightweight” or “sand-lightweight” concrete, shear strength V_n shall not be taken greater than $(0.2 - 0.07 a_v/d) f'_c b_w d$ nor $(800 - 280 a_v/d) b_w d$ in pounds.
- (c) Reinforcement A_f to resist moment ($V_u a_v + N_{uc} (h - d)$) shall be computed in accordance with Articles 8.16.2 and 8.16.3.
- (d) Reinforcement A_n to resist tensile force N_{uc} shall be determined from $N_{uc} \leq \phi A_n f_y$. Tensile force N_{uc} shall not be taken less than $0.2 V_u$ unless special provisions are made to avoid tensile forces. Tensile force N_{uc}

**Part D
REINFORCEMENT**

8.17 REINFORCEMENT OF FLEXURAL MEMBERS

8.17.1 Minimum Reinforcement

8.17.1.1 At any section of a flexural member where tension reinforcement is required by analysis, the reinforcement provided shall be adequate to develop a moment at least 1.2 times the cracking moment calculated on the basis of the modulus of rupture for normal weight concrete specified in Article 8.15.2.1.1.

$$\phi M_n \geq 1.2 M_{cr} \quad (8-62)$$

As an aid to the designer, the minimum requirements for this section may be obtained from the following appropriate equations:

For rectangular, I and T-sections:

$$\rho_{min} = \left[10 + \frac{(I/y_t)}{bd^2} \right] \left[\frac{I/y_t}{bd^2} \right] \frac{\sqrt{f'_c}}{f_y}$$

For rectangular or I-sections:

$$\rho_{min} = 10.2 \left[\frac{(I/y_t)}{bd^2} \right] \frac{\sqrt{f'_c}}{f_y}$$

For rectangular sections only:

$$\rho_{min} = 1.7 \left[\frac{h}{d} \right]^2 \frac{\sqrt{f'_c}}{f_y}$$

8.17.1.2 The requirements of Article 8.17.1.1 may be waived if the area of reinforcement provided at a section is at least one-third greater than that required by analysis based on the loading combinations specified in Article 3.22.

SUMMARY

Let A_s = amount of reinforcement required by analysis
 A_{sc} = amount of reinforcement as calculated using the approximate equation(s) in Article 8.17.1.1

If $A_s > A_{sc}$,
 use A_s as minimum reinforcement.

If $A_s < A_{sc}$,
 compare $4/3 \times A_s$ and A_{sc} , and use the smaller value as minimum reinforcement.

8.17.2 Distribution of Reinforcement

8.17.2.1 Flexural Tension Reinforcement in Zones of Maximum Tension

8.17.2.1.1 Where flanges of T-girders and box-girders are in tension, tension reinforcement shall be distributed over an effective tension flange width equal to $1/10$ the girder span length or a width as defined in Article 8.10.1, whichever is smaller. If the actual width, center-to-center of girder webs, exceeds the effective tension flange width, and for excess portions of the deck slab overhang, additional longitudinal reinforcement with area not less than 0.4 percent of the excess slab area shall be provided in the excess portions of the slab.

8.17.2.1.2 For integral bent caps of T-girder and box-girder construction, tension reinforcement shall be placed within a width not to exceed the web width plus an overhanging slab width on each side of the bent cap web equal to one-fourth the average spacing of the intersecting girder webs or a width as defined in Article 8.10.1.4 for integral bent caps, whichever is smaller.

8.17.2.1.3 If the depth of the side face of a member exceeds 3 feet, longitudinal skin reinforcement shall be uniformly distributed along both side faces of the member for a distance $d/2$ nearest the flexural tension reinforcement.

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ment. The area of skin reinforcement A_{sk} per foot of height on each side face shall be $\geq 0.012 (d - 30)$. The maximum spacing of skin reinforcement shall not exceed the lesser of $d/6$ and 12 inches. Such reinforcement may be included in strength computations if a strain compatibility analysis is made to determine stresses in the individual bars or wires. The total area of longitudinal skin reinforcement in both faces need not exceed one-half of the required flexural tensile reinforcement.

8.17.2.2 Transverse Deck Slab Reinforcement in T-Girders and Box Girders

At least one-third of the bottom layer of the transverse reinforcement in the deck slab shall extend to the exterior face of the outside girder web in each group and be anchored by a standard 90-degree hook. If the slab extends beyond the last girder web, such reinforcement shall extend into the slab overhand and shall have an anchorage beyond the exterior face of the girder web not less than that provided by a standard hook.

8.17.2.3 Bottom Slab Reinforcement for Box Girders

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

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

8.17.3 Lateral Reinforcement of Flexural Members

8.17.3.1 Compression reinforcement used to increase the strength of flexural members shall be enclosed by ties or stirrups which shall be at least No. 3 in size for longitudinal bars that are No. 10 or smaller, and at least No. 4 in size for No. 11, No. 14, No. 18, and bundled longitudinal bars. Welded wire fabric of equivalent area may be used instead of bars. The spacing of ties shall not ex-

- (a) For design by Strength Design, factored shear force V_u exceeds one-half the shear strength provided by concrete ϕV_c .
- (b) For design by Service Load Design, design shear stress v exceeds one-half the permissible shear stress carried by concrete v_c .

8.19.1.2 Where shear reinforcement is required by Article 8.19.1.1, or by analysis, the area provided shall not be less than:

$$A_v = \frac{50b_w s}{f_y} \quad (8-64)$$

where b_w and s are in inches.

8.19.1.3 Minimum shear reinforcement requirements may be waived if it is shown by test that the required ultimate flexural and shear capacity can be developed when shear reinforcement is omitted.

8.19.2 Types of Shear Reinforcement

8.19.2.1 Shear reinforcement may consist of:

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

8.19.2.2 Shear reinforcement shall be developed at both ends in accordance with the requirements of Article 8.27.

8.19.3 Spacing of Shear Reinforcement

Spacing of shear reinforcement placed perpendicular to the axis of the member shall not exceed $d/2$ or 24 inches. Inclined stirrups and bent longitudinal reinforcement shall be so spaced that every 45-degree line extending toward the reaction from the mid-depth of the member, $d/2$, to the longitudinal tension reinforcement shall be crossed by at least one line of shear reinforcement.

8.19.4 The spacing of stirrups computed for shear at a support shall be continued from the center of the support to a distance "d" away from the face of the support, in

the case of continuous end framing or a simply supported end bearing, unless a concentrated load within this distance reduces the shear.

For simply supported ends, spacing of stirrups shall be at 4" from end of beam completely across the bearing area.

8.20 SHRINKAGE AND TEMPERATURE REINFORCEMENT

8.20.1 Reinforcement for shrinkage and temperature stresses shall be provided near exposed surfaces of walls and slabs not otherwise reinforced. The total area of reinforcement provided shall be at least $\frac{1}{8}$ square inch per foot in each direction.

8.20.2 The spacing of shrinkage and temperature reinforcement shall not exceed three times the wall or slab thickness, or 18 inches.

8.21 SPACING LIMITS FOR REINFORCEMENT

8.21.1 For cast-in-place concrete the clear distance between parallel bars in a layer shall not be less than 1.5 bar diameters, 1.5 times the maximum size of the coarse aggregate, or 1 $\frac{1}{2}$ inches.

8.21.2 For precast concrete (manufactured under plant control conditions) the clear distance between parallel bars in a layer shall be not less than 1 bar diameter, 1 $\frac{1}{3}$ times the maximum size of the coarse aggregate, or 1 inch.

8.21.3 Where positive or negative reinforcement is placed in two or more layers, bars in the upper layers shall be placed directly above those in the bottom layer with the clear distance between layers not less than 1 inch.

8.21.4 The clear distance limitation between bars shall also apply to the clear distance between a contact lap splice and adjacent splices or bars.

8.21.5 Groups of parallel reinforcing bars bundled in contact to act as a unit shall be limited to 4 in any one bundle. Bars larger than No. 11 shall be limited to two in any one bundle in beams. Bundled bars shall be located within stirrups or ties. Individual bars in a bundle cut off within the span of a member shall terminate at points at least 40-bar diameters apart. Where spacing limitations are based on bar diameter, a unit of bundled bars shall be treated as a single bar of a diameter derived from the equivalent total area.

8.21.6 In walls and slabs the primary flexural reinforcement shall be spaced not farther apart than 1.5 times the wall or slab thickness, or 18 inches.

8.22 PROTECTION AGAINST CORROSION

8.22.1 The following minimum concrete cover shall be provided for reinforcement:

	Min. cover (inches)
Pier caps, bridge seats and backwalls:	
Principal reinforcement	2 3/4
Stirrups and ties	2 1/4
Footings and pier columns:	
Principal reinforcement	3
Stirrups and ties	2 1/2
Cast-in-place deck slabs:	
Top reinforcement	2 1/2†
Bottom reinforcement.....	1 1/4
Cast-in-place slab spans:	
Top reinforcement	2 1/2†
Bottom reinforcement.....	2
Slabs and box beams: top steel	1 3/4
Reinforced concrete box culverts and rigid frames with more than 2 ft. fill over top of slab:	
All reinforcement.....	1 1/2
Reinforcement concrete box culverts and rigid frames with less than 2 ft. fill over top of slab:	
Top slab – top reinforcement.....	2 1/2
Top slab – bottom reinforcement.....	2
All other reinforcement	1 1/2
Rails, railposts, curbs and parapets:	
Principal reinforcement	1 1/2
Stirrups, ties and spirals.....	1
Concrete piles cast against and/or permanently exposed to earth.....	2
All other components not indicated above:	
Principal reinforcement	2 1/2
Stirrups and ties	2

† Includes 1/2 inch monolithic wearing surface.

8.22.2 For bundled bars, the minimum concrete cover shall be equal to the equivalent diameter of the bundle, but need not be greater than 2 inches, except for concrete cast against permanently exposed to earth in which case the minimum cover shall be 3 inches. However, in no case shall the cover be less than that required by Article 8.22.1.

8.22.3 In corrosive* or marine environments or other severe exposure conditions, the minimum cover required by Article 8.22.1 shall be increased 1 inch except where epoxy-coated reinforcement is used.

*For additional information on corrosion protection methods, refer to National Cooperative Highway Research Report 297, "Evaluation of Bridge Deck Protective Strategies."

8.22.4 Exposed reinforcement, inserts, and plates intended for bonding with future extensions shall be protected from corrosion.

8.22.5 Galvanized Reinforcement

The following reinforcing steel bars shall be galvanized:

Reinforcing bars connecting parapets/rails and terminal walls to the deck slab or abutment wingwall.

Any reinforcing bars protruding from prestressed concrete beams after the beams are cast, such as stirrups or positive moment reinforcement for live load continuity.

8.22.6 Epoxy-coated Reinforcement

Epoxy-coated bars shall be used in the following areas:

Abutments: bridge seats, backwalls and wingwalls except as noted under galvanized bars (Article 8.22.5);

Box culverts with 0 to 2 foot fills;

Deck slabs: all reinforcement;

End diaphragms in concrete spans and end bolster in steel spans: all reinforcement (under joints only);

Medians: all reinforcement;

Parapets and terminal walls: all reinforcement except as noted under galvanized bars (Article 8.22.5);

Piers (single stem and column, including bents): all reinforcement in cap (under joints only);

Prestressed I-beams, box beams and slabs: all reinforcement except strands and reinforcement noted under galvanized bars (Article 8.22.5);

Prestressed piles: all reinforcement except strands when piles are exposed above ground (pile bents, piles under footings raised above streams or in tidal areas, etc.);

Rigid frames with less than 2 foot fill: all reinforcement in and extending into the top slab;

Slab spans: all reinforcement.

8.23 HOOKS and BENDS

8.23.1 Standard Hooks

The term “standard hook” as used herein shall mean one of the following:

- (1) 180-deg bend plus $4d_b$ extension, but not less than $2\frac{1}{2}$ in. at free end of bar.
- (2) 90-deg bend plus $12d_b$ extension at free end of bar.
- (3) For stirrup and tie hooks:
 - (a) No. 5 bar and smaller, 90-deg bend plus $6d_b$ extension at free end of bar, or
 - (b) No. 6, No. 7, and No. 8 bar, 90-deg bend plus $12d_b$ extension at free end of bar, or
 - (c) No. 8 bar and smaller, 135-deg bend plus $6d_b$ extension at free end of bar.

8.23.2 Minimum Bend Diameters

8.23.2.1 Diameter of bend measured on the inside of the bar, other than for stirrups and ties, shall not be less than the values given in Table 8.23.2.1.

8.23.2.2 The inside diameter of bend for stirrups and ties shall not be less than 4 bar diameters for sizes No. 5 and smaller. For bars larger than size No. 5 diameter of bend shall be in accordance with Table 8.23.2.1.

8.23.2.3 The inside diameter of bend in smooth or deformed welded wire fabric for stirrups and ties shall not be less than 4-wire diameters for deformed wire larger than D6 and 2-wire diameters for all other wires. Bends with inside diameters of less than 8-wire diameters shall not be less than 4-wire diameters from the nearest welded intersection.

8.24 DEVELOPMENT OF FLEXURAL REINFORCEMENT

8.24.1 General

8.24.1.1 The calculated tension or compression in the reinforcement at each section shall be developed on each side of that section by embedment length, hook or mechanical device, or a combination thereof. Hooks may be used in developing bars in tension only.

8.24.1.2 Critical sections for development of reinforcement in flexural members are at points of maximum stress and at points within the span where adjacent reinforcement terminates or is bent. The provisions of Article 8.24.2.3 must also be satisfied.

TABLE 8.23.2.1 Minimum Diameters of Bend

Bar Size	Minimum Diameter
Nos. 3 through 8	6-bar diameters
Nos. 9, 10, and 11	8-bar diameters
Nos. 14 and 18	10-bar diameters

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Section 9 PRESTRESSED CONCRETE

Part A GENERAL REQUIREMENTS AND MATERIALS

9.1 APPLICATION

9.1.1 General

The specifications of this section are intended for design of prestressed concrete bridge members. Members designed as reinforced concrete, except for a percentage of tensile steel stressed to improve service behavior, shall conform to the applicable specifications of Section 8.

Exceptionally long span or unusual structures require detailed consideration of effects which under this Section may have been assigned arbitrary values.

The weight of any prestressed member shall be limited to 72,000 lbs and the length limited to 75 feet in order to allow hauling by truck. When investigating the possibilities of greater lengths or heavier members, the Materials Division should be consulted.

9.1.2 Notations

<p>A_s = area of non-prestressed tension reinforcement (Articles 9.7 and 9.19)</p> <p>A'_s = area of compression reinforcement (Article 9.19)</p> <p>A^*_s = area of prestressing steel (Article 9.17)</p> <p>A_{sf} = steel area required to develop the compressive strength of the overhanging portions of the flange (Article 9.17)</p> <p>A_{sr} = steel area required to develop the compressive strength of the web of a flanged section (Article 9.17-9-19)</p> <p>A_v = area of web reinforcement (Article 9.20)</p> <p>B = width of flange of flanged member or width of rectangular member</p> <p>b_v = width of cross section at the contact surface being investigated for horizontal shear (Article 9.20)</p> <p>b' = width of web of a flanged member</p> <p>CR_c = loss of prestress due creep of concrete (Article 9.16)</p> <p>CR_s = loss of prestress due to relaxation of prestressing steel (Article 9.16)</p> <p>D = nominal diameter of prestressing steel (Articles 9.17 and 9.27)</p>	<p>d = distance from extreme compressive fiber to centroid of the prestressing force, or to centroid of negative moment reinforcing for precast girder bridges made continuous</p> <p>d_t = distance from the extreme compressive fiber to the centroid of the non-prestressed tension reinforcement (Articles 9.7 and 9.17-9.19)</p> <p>ES = loss of prestress due to elastic shortening (Article 9.16)</p> <p>e = base of Napierian logarithms (Article 9.16)</p> <p>f_{cds} = average concrete compressive stress at the c.g. of the prestressing steel under full dead load (Article 9.16)</p> <p>f_{cir} = average concrete stress at the c.g. of the prestressing steel at time of release (Article 9.16)</p> <p>f'_c = compressive strength of concrete at 28 days</p> <p>f'_{ci} = compressive strength of concrete at time of initial prestress (Article 9.15)</p> <p>f_{ct} = average splitting tensile strength of lightweight aggregate concrete, psi</p> <p>f_d = stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads (Article 9.20)</p> <p>f_{pc} = compressive stress in concrete (after allowance for all prestress losses) at centroid of cross section resisting externally applied loads or at junction of web and flange when the centroid lies within the flange (In a composite member f_{pc} is resultant compressive stress at centroid of composite section, or at junction of web and flange when the centroid lies within the flange, due to both prestress and moments resisted by precast member acting alone.) (Article 9.20)</p> <p>f_{pc} = compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads (Article 9.20)</p>
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Tendon—Wire, strand, or bar, or bundle of such elements, used to impart prestress to concrete.

Transfer—Act of transferring stress in prestressing tendons from jacks or pretensioning bed to concrete member.

Transfer Length—Length over which prestressing force is transferred to concrete by bond in pretensioned members.

Wobble Friction—Friction caused by unintended deviation of prestressing sheath or duct from its specified profile or alignment.

Wrapping or Sheathing—Enclosure around a prestressing tendon to avoid temporary or permanent bond between prestressing tendon and surrounding concrete.

9.2 CONCRETE

The specified compressive strength f'_c , of the concrete for each part of the structure shall be shown on the plans. The requirements for f'_c shall be based on tests of cylinders made and tested in accordance with Division II, Section 8, “Concrete Structures.”

9.4 GENERAL

Members shall be proportioned for adequate strength using these specifications as minimum guidelines. 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.

9.5 EXPANSION AND CONTRACTION

9.5.1 In all bridges, provisions shall be made in the design to resist thermal stresses induced, or means shall be provided for movement caused by temperature changes.

9.5.2 Movements not otherwise provided for, including shortening during stressing, shall be provided for by means of hinged columns, rockers, sliding plates, elastomeric pads, or other devices.

9.3 REINFORCEMENT

9.3.1 Prestressing Steel

Wire, strands, or bars shall conform to one of the following specifications.

“Uncoated Stress-Relieved Wire for Prestressed Concrete,” AASHTO M 204.

Uncoated Seven-Wire Stress-Relieved Strand for Prestressed Concrete,” AASHTO M 203.

“Uncoated High-Strength Steel Bar for Prestressing Concrete,” ASTM A 722.

Wire, strands, and bars not specifically listed in AASHTO M204, AASHTO M 203, or ASTM A 722 may be used provided they conform to the minimum requirements of these specifications.

9.3.2 Non-Prestressed Reinforcement

Non-prestressed reinforcement shall conform to the requirements in Article 8.3.

Part B ANALYSIS

9.6 SPAN LENGTH

The effective span lengths of simply supported beams shall not exceed the clear span plus the depth of the beam. The span length of continuous or restrained floor slabs and beams shall be the clear distance between faces of support. Where fillets making an angle of 45 degrees or more with the axis of a continuous or restrained slab are built monolithic with the slab and support, the span shall be measured from the section where the combined depth of the slab and the fillet is at least one and one-half times the thickness of the slab. Maximum negative moments are to be considered as existing at the ends of the span, as above defined. No portion of the fillet shall be considered as adding to the effective depth.

In lieu of the above, span length may be taken as the distance center-to-center of bearings.

9.7 FRAMES AND CONTINUOUS CONSTRUCTION

9.7.1 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 pre-

stressing (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.

9.7.2 Bridges Composed of Simple-Span Precast Prestressed Girders Made continuous

9.7.2.1 General

When structural continuity is assumed in calculating live loads plus impact and composite dead load moments, the effects of creep and shrinkage shall be considered in the design of bridges incorporation simple span precast, prestressed girders and deck slabs continuous over two or more spans.

9.7.2.2 Positive Moment Connection at Piers

9.7.2.2.1 Provisions shall be made in the design for the positive moments that may develop in the negative moment region due to the combined effects of creep and shrinkage in the girders and deck slab, and due to the effects of live load plus impact in remote spans. Shrinkage and elastic shortening of the pier shall be considered when significant.

9.7.2.2.2 Non-prestressed positive moment connection reinforcement at piers may be designed at a working stress of 0.6 times the yield strength but not to exceed 36 ksi. ½" diameter strands may be designed for connection reinforcement at a working stress $\leq 0.15 f'_s$ when considering a service life up to two million cycles of maximum loading. The pin diameter for bending the strands shall be ¾" and the minimum strand embedment from the beam end shall be 2'-6".

9.7.2.3 Negative Moments

9.7.2.3.1 Negative moment reinforcement in deck slab shall be proportioned by strength design with load factors in accordance with Article 9.14.

9.7.2.3.2 The ultimate negative resisting moment shall be calculated using the compressive strength of the girder concrete regardless of the strength of the diaphragm concrete.

9.7.2.3.3 ½" diameter strands in beams used for resisting negative moment due to deck slab dead load at piers may be designed at working stress

$$= 0.75 (6.14L_e - 50.61) \text{ (ksi)}$$

where L_e (inch) = Total strand embedment length

In such cases, concrete in the closure diaphragms shall attain a compressive strength ≥ 3750 psi before casting the deck slab concrete.

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9.7.3 Segmental Box Girders

9.7.3.1 General

9.7.3.1.1 Elastic analysis and beam theory may be used in the design of segmental box girder structures.

9.7.3.1.2 In the analysis of precast segmental box girder bridges, no tension shall be permitted across any joint between segments during any state of erection or service loading.

9.7.3.1.3 In addition to the usual substructure design considerations, unbalanced cantilever moments due to segment weights and erection loads shall be accommodated in pier design or with auxiliary struts. Erection equipment which can eliminate these unbalanced moments may be used.

9.7.3.2 Flexure

The transverse design of segmental box girders for flexure shall consider the segments as rigid box frames. Top slabs shall be analyzed as variable depth sections considering the fillets between top slab and webs. Wheel loads shall be positioned to provide maximum moments, and elastic analysis shall be used to determine the effective longitudinal distribution of wheel loads for each load location. (See Article 3.11.) Transverse prestressing of top slabs is generally recommended.

9.7.3.3 Torsion

In the design of the cross section, consideration shall be given to the increase in web shear resulting from eccentric loading or geometry of structure.

9.8 EFFECTIVE FLANGE WIDTH

9.8.1 T-Beams

9.8.1.1 For composite prestressed construction where slabs or flanges are assumed to act integrally with the beam, the effective flange width shall conform to the provisions for t-girder flanges in Article 8.10.1.

9.8.1.2 For monolithic prestressed construction, with normal slab span and girder spacing, the effective flange width shall be the distance center-to-center of beams. For very short spans, or where girder spacing is excessive, analytical investigations shall be made to determine the anticipated width of flange acting with the beam.

9.8.1.3 For monolithic prestressed design of isolated beams, the flange width shall not exceed 15 times the web width and shall be adequate for all design loads.

9.8.2 Box Girders

9.8.2.1 For cast-in-place box girders with normal slab span and girder spacing, where the slabs are considered an integral part of the girder, the entire slab width shall be assumed to be effective in compression.

9.8.2.2 For box girders of unusual proportions, including segmental box girders, methods of analysis which consider shear lag shall be used to determine stresses in the cross section due to longitudinal bending.

9.8.2.3 Adequate fillets shall be provided at the intersections of all surfaces within the cell of a box girder, except at the junction of web and bottom flange where none are required.

9.8.3 Precast/Prestressed Concrete Beams with Wide Top Flanges

9.8.3.1 For composite prestressed concrete where slabs or flanges are assumed to act integrally with the precast beam, the effective web width of the precast beam shall be the lesser of (1) six times the maximum thickness of the flange (excluding fillets) on either side of the web plus the web and fillets, and (2) the total width of the top flange.

9.8.3.2 The effective flange width of the composite section shall be the lesser of (1) one-fourth of the span length of the girder, (2) six (6) times the thickness of the slab on each side of the effective web width as determined by Article 9.8.3.1 plus the effective web width and (3) one-half the clear distance on each side of the effective web width plus the effective web width.

9.9 FLANGE AND WEB THICKNESS—BOX GIRDERS

9.9.1 Top Flange

The minimum top flange thickness shall be $\frac{1}{30}$ th of the clear distance between fillets or webs but not less than 6 inches, except the minimum thickness may be reduced for factory product precast, pretensioned elements to $5\frac{1}{2}$ inches

9.9.2 Bottom Flange

The minimum bottom flange thickness shall be $\frac{1}{30}$ th of the clear distance between fillets or webs but not less than $5\frac{1}{2}$ inches, except the minimum thickness may be reduced for factory produced precast, pretensioned elements to 5 inches.

9.9.3 Web

Changes in girder stem thickness shall be tapered for a minimum distance of 12 times the difference in web thickness.

9.10 DIAPHRAGMS

9.10.1 General

Diaphragms shall be provided in accordance with Articles 9.10.2 and 9.10.3 except that diaphragms may be omitted where tests or structural analysis show adequate strength.

9.10.2 T-Beams and Composite I-Beams

Diaphragms or other means shall be used at span ends to strengthen the free edge of the slab and to transmit lateral forces to the substructure. Intermediate diaphragms shall be placed between the beams only for spans over 40 feet as follows:

For spans $\leq 80'$: at point of maximum moment;
For spans $> 80'$: equal spacing with a maximum Spacing of 40 feet.

Intermediate diaphragms shall be normal to the main members when the skew is greater than 20 degrees.

9.10.3 Box Girders

9.10.3.1 For spread box beams, diaphragms shall be placed within the box and between boxes at span ends and at the points of maximum moment for spans over 80 feet.

9.10.3.2 For precast box multi-beam bridges, diaphragms are required only if necessary for slab-end support or to contain or resist transverse tension ties.

9.10.3.3 For cast-in-place box girders, diaphragms or other means shall be used at span ends to resist lateral forces and maintain section geometry. Intermediate diaphragms are not required for bridges with inside radius of curvature of 800 feet or greater

9.10.3.4 For segmental box girders, diaphragms shall be placed within the box at span ends. Intermediate diaphragms are not required for bridges with inside radius of curvature of 800 feet or greater.

9.10.3.5 For all types of prestressed boxes in bridges with inside radius of curvature less than 800 feet, intermediate diaphragms may be required and the spacing and strength of diaphragms shall be given special consideration in the design of the structure.

9.11 DEFLECTIONS

9.11.1 General

Deflection calculations shall consider dead load, live load, prestressing, erection loads, concrete creep and shrinkage, and steel relaxation.

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

9.14 LOAD FACTORS

The computed strength capacity shall not be less than the largest value from load factor design in Article 3.22. For the design of post-tensioned anchorage zones a load factor of 1.2 shall be applied to the maximum tendon jacking force.

The following strength capacity reduction factors shall be used.

- For factory produced precast prestressed concrete members $\phi = 1.0$
- For post-tensioned cast-in-place concrete members $\phi = 0.95$
- For shear $\phi = 0.90$
- For anchorage zones $\phi = 0.85$ for normal weight concrete and $\phi = 0.70$ for lightweight concrete.

9.15 ALLOWABLE STRESSES

The design of precast prestressed members ordinarily shall be based on $f'_c = 5000$ psi. High performance concrete with f'_c up to 8000 psi may be used in prestressed concrete whenever it is economical. Strengths between 8000 to 10,000 psi may be used with the approval of the State Structure and Bridge Engineer. The Engineer shall satisfy himself completely that the controls over materials and fabrication procedures will provide the required strengths. The provisions of this section are equally applicable to prestressed concrete structures and components designed with lower concrete strengths.

9.15.1 Prestressing Steel

- Pretensioned members:
 - Stress immediately prior to transfer --
 - Low-relaxation strands $0.75 f'_s$
 - Stress-relieved strands $0.75 f'_s$
- Post-tensioned members
 - Stress immediately after seating --
 - At anchorage..... $0.70 f'_s$

- At the end of the seating loss zone $0.83 f_y^*$
- Tensioning to $0.90 f_y^*$ for short periods of time prior to seating may be permitted to offset seating and friction losses provided the stress at the anchorage does not exceed the above value
- Stress at service load † after losses $0.80 f_y^*$

9.15.2 Concrete

9.15.2.1 Temporary Stresses Before Losses Due to Creep and Shrinkage

- Compression:
 - Pretensioned members $0.60 f'_{ci}$
 - Post-tensioned members $0.55 f'_{ci}$
- Tension:
 - Precompressed tensile zone No temporary allowable stresses are specified. See Article 9.15.2.2 for allowable stresses after losses.
 - Other Areas
 - In tension areas with
 - no bonded reinforcement 200 psi or $3 \sqrt{f'_{ci}}$
 - Where the calculated tensile stress exceeds this value, bonded reinforcement shall be provided to resist the total tension force in the concrete computed on the assumption of an uncracked section. The maximum tensile stress shall not exceed $7.5 \sqrt{f'_{ci}}$

9.15.2.2 Stress at Service Load After Losses Have Occurred

- Compression
 - (a) The compressive stresses under all load combinations, except as stated in (b) and (c), shall not exceed $0.60 f'_c$
 - (b) The compressive stresses due to effective prestress plus permanent (dead) loads shall not exceed $0.40 f'_c$
 - (c) The compressive stress due to live loads plus one-half of the sum of compressive stresses due to prestress and permanent (dead) loads shall not exceed $0.40 f'_c$
- Tension in the precompressed tensile zone:
 - (a) For members with bonded reinforcement* $6 \sqrt{f'_{ci}}$
 - For severe corrosive exposure conditions, such as coastal areas $3 \sqrt{f'_{ci}}$

*Includes bonded prestressed strands.
 †Service load consists of all loads contained in Article 3.2 but does not include overload provisions.

(b) For members without bonded reinforcement0
Tension in other areas is limited by allowable temporary stresses specified in Article 9.1555.2.1.

9.15.2.3 Cracking Stress

Modulus of rupture from tests or if not available.

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

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

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

9.15.2.5 Anchorage Bearing Stress

Post-tensioned anchorage at service load.... 3,000 psi
(but not to exceed $0.9 f'_{ci}$)

9.16 LOSS OF PRESTRESS

9.16.1 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 elongations shall be shown on the plans. These friction losses shall be calculated as follows:

$$T_o = T_x e^{(KL + \mu\alpha)} \quad (9-1)$$

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

$$T_o = T_x (1 + KL + \mu\alpha) \quad (9-2)$$

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/ft	μ
Wire or strand	Rigid and semi-rigid galvanized metal sheathing	0.0002	0.15-0.25**
	Polyethylene	0.0002	0.23
	Rigid steel pipe	0.0002	0.25***
High Strength bars	Galvanized metal sheathing	0.0002	0.15

**A friction coefficient of 0.25 is appropriate for 12 strand tendons. A lower coefficient may be used for larger tendon and duct sizes.
***Lubrication will probably be required.

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

*Refer to Article 9.18.

the welded seam will not be required.

9.16.2 Prestress Losses

9.16.2.1 General

Loss of prestress due to all causes, excluding friction, may be determined by the following method.* The method is based on normal weight concrete and one of the following types of prestressing steel: 250 or 270 ksi, seven-wire, stress-relieved or low-relaxation strand; 240 ksi stress-relieved wires; or 145 to 160 ksi smooth or deformed bars. Refer to documented tests for data regarding the properties and the effects of lightweight aggregate concrete on prestress losses.

TOTAL LOSS

$$\Delta f_s = SH + ES + CR_c + CR_s \quad (9-3)$$

where:

- Δf_s = total loss excluding friction in pounds per square;
- SH = loss due to concrete shrinkage in pounds per square inch;
- ES = loss due to elastic shortening in pounds per square inch;
- CR_c = loss due to creep of concrete in pounds per square inch;
- CR_s = loss due to relaxation of prestressing steel in pounds per square inch.

However, in no case shall be total prestress losses (excluding friction) be less than the following:

Stress-relieved strands:
35,000 psi for pre-tensioned members
30,000 psi for post-tensioned members

Low-relaxation strands:
25,000 psi for both pre-tensioned and post-tensioned members.

*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. See also FHWA Report FHWA/RD 85/045, *Criteria for Designing Lightweight Concrete Bridges*.

9.16.2.1.1 Shrinkage

Pretensioned Members:

$$SH = 17,000 - 150 R \quad (9-4)$$

Post-tensioned Members:

$$SH = -.80(17,000 - 150 RH) \quad (9-5)$$

where RH = mean annual ambient relative humidity in percent (See Figure 9.16.2.1.1)

9.16.2.1.2 Elastic Shortening

Pretensioned Members*

$$ES = \frac{E_s}{E_{ci}} f_{cir} \quad (9-6)$$

Post-tensioned Members*

$$ES = 0.5 \frac{E_s}{E_{ci}} f_{cir} \quad (9-7)$$

where,

E_s = modulus of elasticity of prestressing steel strand, which can be assumed to be 28×10^6 psi;

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

$$E_{ci} = 33w_{3/2} \sqrt{f'_{ci}} \quad (9.8)$$

in which w is the concrete unit weight in pounds per cubic foot and f'_{ci} is in pounds per square inch;

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 state, 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 stress relieved strand or $0.69 f'_s$ for low relaxation strand in typical pre-tensioned members.)

9.16.2.1.3 Creep of Concrete

Pretensioned and post-tensioned members

$$CR_c = 12 f_{cir} - 7 f_{cds} \quad (9-9)$$

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.

9.16.2.1.4 Relaxation of Prestressing Steel*

Pretensioned Members

250 to 270 ksi Strand

$$CR_x = 20,000 - 0.4 ES - 0.2 (SH + CR_c) \quad (9-10)$$

for stress relieved strand

$$CR_s = 5,000 - 0.10 ES - 0.05 (SH + CR_c) \quad (9-10A)$$

for low relaxation strand

Post-tensioned Members

250 to 270 ksi Strand

$$CR_s = 20,000 - 0.3 FR - 0.4 ES - 0.2 (SH + CR_c) \quad (9-11)$$

for stress relieved strand

$$CR_s = 5,000 - 0.07 FR - 0.1 ES - 0.05 (SH + CR_c) \quad (9-11A)$$

for low relaxation strand

240 ksi Wire

$$CR_s = 18,000 - 0.3 FR - 0.4 ES - 0.2 (SH + CR_c) \quad (9-12)$$

145- to 160-ksi Bars

$$CR_s = 3,000$$

where

FR = friction loss stress reduction in psi below the level of $0.70 f'_s$ at the point under consideration, computed according to Article 9.16.1,

ES, SH, = appropriate values as determined for and CR_c either pre-tensioned or post-tensioned members.

*Certain tensioning procedures may alter the elastic shortening losses.

*The relaxation losses are based on an initial stress equal to the stress at anchorages allowed by Article 9.15.1.

9.17.4 Steel Stress

9.17.4.1 Unless the value of f_{su}' can be more accurately known from detailed analysis, the following values may be used:

Bonded Members . . .

With prestressing only (as defined);

$$f_{su}^* = f_s' \left[1 - (\gamma^* / \beta_1) (p^* f_s' / f_c') \right] \quad (9-17)$$

with non-prestressed tension reinforcement included;

$$f_{su}^* = f_s' \left\{ 1 - \frac{\gamma^*}{\beta_1} \left[\frac{p^* f_s'}{f_c'} + \frac{d_t}{d} \left(\frac{p f_{sy}}{f_c'} \right) \right] \right\} \quad (9-17a)$$

Unbonded members . . . $f_{su}^* = f_{sc} + 900((d - y_u) / l_c)$ (9-18)

but shall not exceed f_y^* .

Where

- y_u = distance from extreme compression fiber to the neutral axis assuming the tendon prestressing steel has yielded.
- l_c = $l_t / (1 + 5N_s)$; effective tendon length.
- l_t = tendon length between anchorages (in.).
- N_s = number of support hinges crossed by the tendon between anchorages or discretely bonded points.

provided that

- (1) The stress-strain properties of the prestressing steel approximate those specified in Division II, Article 10.3.1.1.
- (2) The effective prestress after losses is not less than $0.5 f_s'$.

9.17.4.2 At ultimate load, the stress in the prestressing steel of precast deck panels shall be limited to

$$f_{su}^* = \frac{\ell_x}{1.6D} + \frac{2}{3} f_{se} \quad (9-19)$$

but shall not be greater than f_{su}^* as given by the equations in Article 9.17.4.1. In the above equation:

- D = nominal diameter of strand in inches;
- f_{se} = effective stress in prestressing strand after losses in kips per square inch;
- ℓ_x = distance from end of prestressing strand to center of panel in inches.

9.18 DUCTILITY LIMITS

9.18.1 Maximum Prestressing Steel

Prestressed concrete members shall be designed so that the steel is yielding as ultimate capacity is approached. In general, the reinforcement index shall be such that

$$(p^* f_{su}^*) / f_c' \text{ or rectangular sections} \quad (9-20)$$

and

$$A_{sr} f_{su}^* / (b' d) f_c' \text{ for flanged sections} \quad (9-21)$$

does not exceed $0.36 \beta_1$. (See Article 9.19 for reinforcement indices of sections with non-prestressed reinforcement.)

For members with reinforcement indices greater than $0.36 \beta_1$, the design flexural strength shall be assumed not greater than:

For rectangular sections

$$\phi M_n = \phi [(0.36 \beta_1 - 0.08 \beta_1^2) f_c' b d^2] \quad (9-22)$$

For flanged sections

$$\phi M_n = \phi [(0.36 \beta_1 - 0.08 \beta_1^2) f_c' b' d^2 + 0.85 f_c' (b - b') t (d - 0.5t)] \quad (9-23)$$

9.18.2 Minimum Steel

9.18.2.1 The total amount of prestressed and non-prestressed reinforcement shall be adequate to develop an ultimate moment at the critical section at least 1.2 times the cracking moment M_{cr}^* .

$$\phi M_n \geq 1.2 M_{cr}^*$$

where

$$M_{cr}^* = S_c (f_r + f_{pe}) - M_{d/nrc} (S_c / S_b - 1)$$

Appropriate values for $M_{d/nrc}$ and S_b shall be used for any intermediate composite sections. Where beams are designed to be noncomposite, substitute S_b for S_c in the above equation for the calculation of M_{cr}^* .

9.18.2.2 The requirements of Article 9.18.2.1 may be waived if the area of prestressed and non-prestressed reinforcement provided at a section is at least one-third

9.20.2.5 The provisions for computing the shear strength provided by concrete V_{ci} and V_{cw} , apply to normal weight concrete. When lightweight aggregate concretes are used (see definition, concrete, structural lightweight, Article 8.1.3), one of the following modifications shall apply:

- (a) When f_{ct} is specified, the shear strength, V_{ci} and V_{cw} , shall be modified by substituting $f_{ct}/6.7$ for $\sqrt{f'_{ci}}$, but the value of $f_{ct}/6.7$ used shall not exceed $\sqrt{f'_c}$.
- (b) When f_{ct} is not specified, V_{ci} and V_{cw} shall be modified by multiplying each term containing $\sqrt{f'_c}$ 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.

9.20.3 Shear Strength Provided by Web Reinforcement

9.20.3.1 The shear strength provided by web reinforcement shall be taken as:

$$V_s = \frac{A_v f_{sy} d}{s} \quad (9-30)$$

where A_v is the area of web reinforcement within a distance s . V_s shall not be taken greater than $8\sqrt{f'_c} b'd$ and d need not be taken less than $0.8h$.

9.20.3.2 The spacing of web reinforcing shall not exceed $0.75h$ or 24 inches. When V_s exceeds $4 f'_c b'd$, this maximum spacing shall be reduced by one-half.

9.20.3.3 The minimum area of web reinforcement shall be—

$$A_v = \frac{50 b's}{f_{sy}} \quad (9-31)$$

where b' and s are in inches and f_{sy} is in psi.

9.20.3.4 The design yield strength of web reinforcement, f_{sy} , shall not exceed 60,000 psi.

9.20.4 Horizontal Shear Design—Composite Flexural Members

9.20.4.1 In a composite member, full transfer of horizontal shear forces shall be assured at contact surfaces of interconnected elements.

9.20.4.2 Design of cross sections subject to horizontal shear may be in accordance with provisions of Article 9.20.4.3 or 9.20.4.4, or any other shear transfer design method that results in prediction of strength in substantial agreement with results of comprehensive tests.

9.20.4.3 Design of cross sections subject to horizontal shear may be based on :

$$V_u \leq \phi V_{nh} \quad (9-31a)$$

where V_u is factored shear force at section considered. V_{nh} is nominal horizontal shear strength in accordance with the following, and where d is for the entire composite section.

- (a) When contact surface is clean, free of laitance, and intentionally roughened, shear strength V_{nh} shall not be taken greater than $80b_v d$, in pounds.
- (b) When minimum ties are provided in accordance with Article 9.20.4.5, and contact surface is clean and free of laitance, but not intentionally roughened, shear strength V_{nh} shall not be taken greater than $80b_v d$, in pounds.
- (c) When minimum ties are provided in accordance with Article 9.20.4.5, and contact surface is clean, free of laitance, and intentionally roughened to a full amplitude of approximately $\frac{1}{4}$ in., shear strength V_{nh} shall not be taken greater than $350b_v d$, in pounds.
- (d) For each percent of tie reinforcement crossing the contact surface in excess of the minimum required by Article 9.20.4.5, shear strength V_{nh} may be increased by $(160f_y/40,000)b_v d$, in pounds.

9.20.4.4 Horizontal shear may be investigated by computing, in any segment not exceeding one-tenth of the span, the change in compressive or tensile force to be transferred, and provisions made to transfer that force as horizontal shear between interconnected elements. The factored horizontal shear force shall not exceed horizontal shear strength of ϕV_{nh} in accordance with Article 9.20.4.3, except that length of segment considered shall be substituted for d .

9.20.4.5 Ties for Horizontal Shear

- (a) When required, a minimum area of tie reinforcement shall be provided between interconnected elements. Tie area shall not be less than $50 b_v s/f_y$, and tie spacing "s" shall not exceed four times the least web width of support element, nor 24 in.
- (b) Ties for horizontal shear may consist of single bars or wire, multiple leg stirrups, or vertical legs of welded wire fabric. All ties shall be adequately anchored into interconnected elements by embedment or hooks.

9.20.5 Horizontal Shear – Box Girders

The requirements of Article 9.20.4 shall apply to the junctions between the flanges and webs of composite or non-composite box girders.

wedge plate, the size of the wedge plate shall be taken as the distance between the extreme wedge holes in the corresponding direction

t = the average thickness of the bearing plate.

E_b = the modulus of elasticity of the bearing-plate material.

9.21.7.2.4 For bearing plates that do not meet the stiffness requirements of Article 9.21.7.2.3, the effective gross-bearing area, A_g , shall be taken as the area geometrically similar to the wedge plate (or to the outer perimeter of the wedge-hole pattern for plates without separate wedge plate) with dimensions increased by assuming load spreading at a 45-degree angle. A larger effective-bearing area may be calculated by assuming an effective area and checking the new f_b and n/t values for conformance with Articles 9.21.7.2.2 and 9.21.7.2.3.

9.21.7.3 Special Anchorage Devices

Special anchorage devices that do not meet the requirements of Article 9.21.7.2 as well as other devices that do meet the requirements of Article 9.21.7.2 but which the engineer-of-record requires to have tested may be used provided that they have been tested by an independent testing agency acceptable to the engineer of record according to the procedures described in Division II, Article 10.3.2 (or equivalent) and meet the acceptance criteria specified in Division II, Article 10.3.2.3.10. For a series of similar special anchorage devices, tests are only required for representative samples unless tests for each capacity of the anchorages in the series are required by the engineer of record.

9.22 PRETENSIONED ANCHORAGE ZONES

9.22.1 In pretensioned beams, vertical stirrups acting at a unit stress of 24,000 psi to resist at least 4 percent of

the total pressing force shall be placed within the distance of $d/4$ of the end of the beam.

9.22.2 For at least the distance d from the end of the beam, nominal reinforcement shall be placed to enclose the prestressing steel in the bottom flange.

9.22.3 for box girders, transverse reinforcement shall be provided and anchored by extending the leg into the web of the girder.

9.22.3 Unless otherwise specified, stress shall not be transferred to concrete until the compressive strength of the concrete as indicated by test cylinders, cured by methods identical with the curing of the member, is at least 4,000 psi.

9.23 CONCRETE STRENGTH AT STRESS TRANSFER

Unless otherwise specified, stress shall not be transferred to concrete until the compressive strength of the concrete as indicated by test cylinders, cured by methods identical with the curing of the members, is at least 4,000 psi for pretensioned members (other than piles) and 3,500 psi for post-tensioned members and pretensioned piles.

9.24 DECK PANELS

9.24.1 Deck panels shall be prestressed with pretensioned strands. The strands shall be in a direction transverse to the stringers when the panels are placed on the supporting stringers. The top surface of the panels shall be roughened in such a manner as to ensure composite action between the precast and cast-in-place concrete.

The size of prestressing strands shall not exceed 3/8".

9.24.2 Reinforcing bars, or equivalent mesh, shall be placed in the panel transverse to the strands to provide at least 0.11 square inches per foot of panel.

The minimum amount of longitudinal steel provided in the cast-in-place portion of slab above deck panels shall be 1/2 the amount required in Article 9.18.2.2 and Article 3.24.10 for distribution reinforcement in a concrete slab with depth equal to the total thickness of the deck panel and the cast-in-place concrete above it.

**Part D
DETAILING**

9.25 FLANGE REINFORCEMENT

Bar reinforcement for cast-in-place T-beam and box girder flanges shall conform to the provisions in Articles 8.17.2.2 and 8.17.2.3 except that the minimum reinforcement in bottom flanges shall be 0.3 percent of the flange section.

9.26 COVER AND SPACING OF STEEL

9.26.1 Minimum Cover

The following minimum concrete cover shall be provided for prestressing and conventional steel:

9.26.1.1 Prestressing Steel and Main
Reinforcement 1½ inch

9.26.1.2. Slab Reinforcement

9.26.1.2.1 Top of Slab 1½ inch
When deicers are used 2 inch

9.26.1.2.2 Bottom of Slab 1 inch

9.26.1.3 Stirrups and Ties 1 inch

9.26.1.4 When deicer chemicals are used, drainage details shall dispose of deicer solutions without constant contact with the prestressed girders. Where such contact cannot be avoided, or in locations where members are exposed to salt water, salt spray, or chemical vapor, additional cover should be provided.

9.26.2 Minimum spacing

9.26.2.1 The minimum clear spacing of prestressing steel at the ends of beams shall be as follows:

Pretensioning steel: three times the diameter of the steel or 1⅓ times the maximum size of the concrete aggregate, whichever is greater.

Post-tensioning steel: 1½ inches or 1½ times the maximum size of the concrete aggregate, whichever is greater.

Normally, the spacing of strands shall be 2 inches c-c min. to strands up to 0.6 inches.

9.26.2.2 Prestressing strands in deck panels shall be spaced symmetrically and uniformly across the width of the panel. They shall not be spaced farther apart than 1½

times the total composite slab thickness or more than 18 inches.

9.26.3 Bundling

9.26.3.1 When post-tensioning steel is draped or deflected, post-tensioning ducts may be bundled in groups of three maximum, provided that the spacing specified in Article 9.26.2 is maintained in the end 3 feet of the member.

9.26.3.2 Where pretensioning steel is bundled, all bundling shall be done in the middle third of the beam length and the deflection points shall be investigated for secondary stresses.

9.26.4 Size of Ducts

9.26.4.1 For tendons made up of a number of wires, bars, or strands, duct area shall be at least twice the net area of the prestressing steel.

9.26.4.2 For tendons made up of a single wire, bar, or strand, the duct diameter shall be at least ¼ inch larger than the nominal diameter of the wire, bar, or strand.

9.27 POST-TENSIONING ANCHORAGES AND COUPLERS

9.27.1 Anchorages, couplers, and splices for bonded post-tensioned reinforcement shall develop at least 95 percent of the minimum specified ultimate strength of the prestressing steel, tested in an unbonded state without exceeding anticipated set. Bond transfer lengths between anchorages and the zone where full prestressing force is required under service and ultimate loads shall normally be sufficient to develop the minimum specified ultimate strength of the prestressing steel. Couplers and splices shall be placed in areas approved by the Engineer and enclosed in a housing long enough to permit the necessary movements. When anchorages or couplers are located at critical sections under ultimate load, the ultimate strength required of the bonded tendons shall not exceed the ultimate capacity of the tendon assembly, including the anchorage or coupler, tested in an unbonded state.

9.27.2 The anchorages of unbonded tendons shall develop at least 95 percent of the minimum specified ultimate strength of the prestressing steel without exceeding anticipated set. The total elongation under ultimate load of the tendon shall not be less than 2 percent measured in a minimum gauge length of 10 feet.

9.27.3 For unbonded tendons, a dynamic test shall be performed on a representative specimen and the tendon shall withstand, without failure, 500,000 cycles from 60 percent to 66 percent of its minimum specified ultimate strength, and also 50 cycles from 40 percent to 80 percent of its minimum specified ultimate strength. The period of each cycle involves the change from the lower stress level to the upper stress level and back to the lower. The specimen used for the second dynamic test need not be the same used for the first dynamic test. Systems utilizing multiple strands, wires, or bars may be tested utilizing a test tendon of smaller capacity than the full size tendon. The test tendon shall duplicate the behavior of the full size tendon and generally shall not have less than 10 percent of the capacity of the full size tendon. Dynamic tests are not required on bonded tendons, unless the anchorage is located or used in such manner that repeated load applications can be expected on the anchorage.

9.27.4 Couplings of unbonded tendons shall be used only at locations specifically indicated and/or approved by the Engineer. Couplings shall not be used at points of sharp tendon curvature. All couplings shall develop at least 95 percent of the minimum specified ultimate strength of the prestressing steel without exceeding anticipated set. The coupling of tendons shall not reduce the elongation at rupture below the requirements of the tendon itself. Couplings and/or coupling components shall be enclosed in housings long enough to permit the necessary movements. All the coupling components shall be completely protected with a coating material prior to final encasement in concrete.

9.27.5 Anchorages, end fittings, couplers, and exposed tendons shall be permanently protected against corrosion.

9.28 EMBEDMENT OF PRESTRESSED STRAND

9.28.1 Three-or seven-wire pretensioning strand shall be bonded beyond the critical section for a development length in inches not less than

$$1.6D \left(f_{su}^* - \frac{2}{3} f_{se} \right) \quad (9-32A)$$

where D is the nominal diameter in inches f_{su}^* and f_{se} are in kips per square inch, and the parenthetical expression is considered to be without units.

9.28.2 Investigations may be limited to those cross sections nearest each end of the member which are required to develop their full ultimate capacity.

9.28.3 Where strand is debonded at the end of a member and tension at service load is allowed in the precompressed tensile zone, the development length in inches shall not be less than

$$2D \left(f_{su}^* - \frac{2}{3} f_{se} \right) \quad (9-32B)$$

9.29 BEARINGS

Bearing devices for prestressed concrete structures shall be designed in accordance with Article 10.29 and Section 14.

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- V_r = range of shear due to live loads and impact, kips (Article 10.38.5.1.1)
- V_u = maximum shear force (Articles 10.34.4, 10.48.5.3, 10.48.8 and 10.53.1.4)
- V_v = vertical shear (Article 10.39.3.1)
- V_w = design shear for a web (Articles 10.39.3.1 and 10.51.3)
- W = length of a channel shear connector, in (Article 10.38.5.1.2)
- W_c = roadway width between curbs in feet or barriers if curbs are not used (Article 10.39.2.1)
- W_L = fraction of a wheel load (Article 10.39.2)
- w = length of a channel shear connector in inches measured in a transverse direction of the flange of a girder (Article 10.38.5.1.1)
- w = unit weigh of concrete, lb per cu ft (Article 10.38.5.1.2)
- w = width of flange between longitudinal stiffeners (Articles 10.39.4.3, 10.39.4.4, and 10.51.5.4)
- Y = ratio of web plate yield strength to stiffener plate yield strength (Articles 10.34.4 and 10.48.5.3)
- Y_o = distance from the neutral axis to the extreme outer fiber, in (Article 10.15.3)
- \bar{y} = location of steel sections from neutral axis (Article 10.50.1.1.1)
- Z = plastic section modulus (Articles 10.48.1, 10.53.1.1, and 10.54.2.1)
- Z_r = allowable range of horizontal shear, in pounds on an individual connector (Article 10.38.5.1)
- α = constant based on the number of stress cycles (Article 10.38.5.1.1)
- α = minimum specified yield strength of the web divided by the minimum specified yield strength of the tension flange (Articles 10.40.2 and 10.40.4)
- β = area of the web divided by the area of the tension flange (Articles 10.40.2 and 10.53.1.2)
- ρ = F_{yw}/F_{yf} (Article 10.53.1.2)
- θ = angle of inclination of the web plate to the vertical (Articles 10.39.3.1 and 10.51.3)
- ψ = ratio of total cross-sectional area to the cross-sectional area of both flanges (Article 10.15.2)
- ψ = distance from the outer edge of the tension flange to the neutral axis divided by the depth of the steel section (Articles 10.40.2 and 10.53.1.2)
- Δ = amount of camber, in (Article 10.15.3)
- Δ_{DL} = dead load camber in inches at any point (Article 10.15.3)
- Δ_m = maximum value of Δ_{DL} , in (Article 10.15.3)
- ϕ = reduction factor (Articles 10.38.5.1.2, 10.56.1.1, and 10.56.1.3)

- ϕ = longitudinal stiffener coefficient (Articles 10.39.4.3 and 10.51.5.4)
- μ = slip coefficient in a slip-critical joint (Article 10.57.3)

10.2 MATERIALS

10.2.1 General

Structures may be designed using ASTM A709 Grades 36, 50 or 50W. ASTM A709 Grade 50 should be used where it is more economical than ASTM A709 Grade 36 steel. In this case, ASTM A709 Grade 36 steel may be used for secondary members if stresses permit. ASTM A709 Grade 50W steel may be used in accordance with the guidelines issued in the FHWA Technical Advisory T5140.22 dated October 3, 1989. Joints in bridges should be eliminated whenever possible by the use of continuous spans and integral abutments.

ASTM A709 Grades 70W and 100/100W steels shall not be used without the permission of the State Structure and Bridge Engineer.

10.2.2 Structural Steels

Structural steels shall conform to the material designated in Table 10.2A. (The stresses in this table are in pounds per square inch.) The modulus of elasticity of all grades of structural steel shall be assumed to be 29,000,000 psi and the coefficient of linear expansion 0.0000065 per degree Fahrenheit.

10.2.3 Steels for Pins, Rollers, and Expansion Rockers

Steels for pins, rollers, and expansion rockers shall conform to one of the designations listed in Table 10.2A and 10.2B, or shall be stainless steel conforming to ASTM A240 or ASTM A276 HNS 21800.

10.2.4 Fasteners—Rivets and Bolts

Fasteners may be carbon steel bolts (ASTM A 307); power-driven rivets, AASHTO M 228 Grades 1 or 2 (ASTM A 502 Grades 1 or 2); or high-strength bolts, AASHTO M 164 (ASTM A 325) or AASHTO M 253 (ASTM A 490).

10.2.5 Weld Metal

Weld metal shall conform to the current requirements of the *ANSI/AASHTO/AWS D1.5 Bridge Welding Code*.

10.2.6 Cast Steel, ductile Iron Castings, Malleable Castings, and Cast Iron

743). Ductile iron castings shall conform to ASTM A 536.

10.2.6.1 Cast Steel and Ductile Iron

Cast steel shall conform to specifications for Steel Castings for Highway Bridges, AASHTO M 192 (ASTM A 486); Mild-to-Medium-Strength Carbon Steel Castings for General Application, AASHTO M 103 (ASTM A 27); and Corrosion-Resistant Iron-Chromium, Iron-Chromium-Nickel and Nickel-Based Alloy Castings for General Application, AASHTO M 163 (ASTM A

10.2.6.2 Malleable Castings

Malleable castings shall conform to specifications for Malleable Iron Castings, ASTM A 47, Grade 35018 (minimum yield point 35,000 psi).

10.2.6.3 Cast Iron

Cast iron castings shall conform to specifications for Gray Iron Castings, AASHTO M 105, Class 30.

Part B DESIGN DETAILS

10.3 REPETITIVE LOADING AND TOUGHNESS CONSIDERATIONS**10.3.1 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 Article 10.32 and that the actual range of stress does not exceed the allowable fatigue stress range given in Table 10.3.1A for the appropriate type and location of material given in Table 10.3.1B and shown in Figure 10.3.1C.

For unpainted weathering steel, A709, all grades, the values of allowable fatigue stress range, Table 10.3.1A, as modified by footnote d, are valid only when the design and details are in accordance with the FHWA *Technical Advisory on Uncoated Weathering Steel in Structures*, dated October 3, 1989.

Main load carrying components subjected to tensile stresses that may be considered non-redundant load path members—that is, where the failure of a single element could cause collapse—shall be designed for the allowable stress ranges indicated in Table 10.3.1A – for Non-redundant Load Path Structures. These tension members or components shall be designated on the plans as fracture critical members or member components (*FCMs) and are subject to the AASHTO *Fracture Control Plan for Non-Redundant Steel Structures*. Examples on non-redundant load path members are flange and web plates in one or two girder bridges, main one-element truss members, hanger plates, and caps at single or two-column bents. Tension members or components of pedestrian bridges shall be excluded from the requirements of the Fracture Control Plan.

10.3.2 Load Cycles

10.3.2.1 All structures shall be designed for the number of stress cycles under Case II in Table 10.3.2A with the following exceptions: structures on the interstate routes and in cases where the 20-year projected ADTT in one direction 2500 or more shall be designed for Case I. Where a traffic count as shown in the plans is used, the directional split shall be assumed to be 50-50 unless otherwise noted. When designing a structure for Group IB loading, the number of stress cycles under Case III shall be used.

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

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

10.3.3 Charpy V-Notch Impact Requirements

10.3.3.1 Main load carrying component members subjected to tensile stress requires supplemental impact properties as described in the Materials Specifications.**

**AASHTO Standard Specifications for Transportation Materials add Methods of Sampling and Testings.

10.3.3.2 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.*** Table 10.3.3A contains the temperature zone designations.

***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.

**TABLE 10.3.1A Allowable Fatigue Stress Range
Redundant Load Path Structures***

Allowable Range of Stress, F_{sr} (ksi) ^a				
Category (See Table 10.3.1B)	For 100,000 Cycles	For 500,000 Cycles	For 2,000,000 Cycles	For over 2,000,000 Cycles
A	63(49) ^d	37(29) ^d	24(18) ^d	24(16) ^d
B	49	29	18	16
B'	39	23	14.5	12
C	35.5	21	13	10 12 ^b
D	28	16	10	7
E	22	13	8	4.5
E'	16	9.2	5.8	2.6
F	15	12	9	8

Nonredundant Load Path Structures*

Allowable Range of Stress, F_{sr} (ksi) ^a				
Category (See Table 10.3.1B)	For 100,000 Cycles	For 500,000 Cycles	For 2,000,000 Cycles	For over 2,000,000 Cycles
A	50(39) ^d	29(23) ^d	24(16) ^d	24(16) ^d
B	39	23	16	16
B'	31	18	11	11
C	28	16	10 12 ^b	9 11 ^b
D	22	13	8	5
E ^c	17	10	6	2.3
E'	12	7	4	1.3
F	12	9	7	6

*Structure types with multi-load paths where a single fracture in a member cannot lead to collapse. For example, a simply supported single span multi-beam bridge or a multi-element eye bar truss member has redundant load paths.

^aThe 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.

^bFor transverse stiffener welds on girder webs or flanges.

^cPartial length welded cover plates shall not be used on flanges more than 0.8 inch thick for nonredundant load path structures.

^dFor unpainted weathering steel, A709, all grades, when used in conformance with FHWA *Technical Advisory on Uncoated Weathering Steel in Structures*, dated October 3, 1989.

The following fillet weld strengths are based on the allowable ranges of stress shown for Category F:

Size of weld inches	Range of Stress: Strength of weld – pounds per inch F_{sr} =					
	15 ksi	12 ksi	9 ksi	8 ksi	7 ksi	6 ksi
1/4	2650	2120	1590	1420	1240	1060
5/16	3310	2650	1990	1770	1550	1325
3/8	3980	3180	2390	2120	1860	1590
1/2	5300	4240	3180	2830	2470	2120
5/8	6630	5300	3980	3540	3090	2650

10.3.3.3 Components requiring mandatory impact properties shall be designated on the drawings and the appropriate zone shall be designated in the contract documents. Limits of tensile stress for top and bottom flanges shall be detailed. All web plates shall be designated as areas of tensile stress.

10.3.3.4 M270 Grades 100/100W steel shall be supplied to Zone 2 requirements as a minimum.

10.3.4 Shear

10.3.4.1 When longitudinal beam or girder members in bridges designed for Case I roadways are investigated for "over 2 million" stress cycles produced by placing a single truck on the bridge (see footnote c of Table 10.3.2A), the total shear force in the beam or girder under this single-truck loading shall be limited to $0.58F_y D_t w C$. The constant C, the ratio of the buckling shear stress to the shear yield stress is defined in Article 10.34.4.2 or Article 10.48.8.1.

10.4 EFFECTIVE LENGTH OF SPAN

For the calculation of stresses, span lengths shall be assumed as the distance between centers of bearings or other points of support.

RATIOS

10.5.1 For beams or girders, the ratio of depth to length of span preferably should not be less than $1/30$.

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

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

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

10.5.5 The ratios given above may be used where it is desirable to keep the depth to a minimum for clearance requirements. Where clearance is not critical, the preferable minimum depth ratios of $1/25$ and $1/30$ shall be increased to $1/20$ and $1/25$, respectively. The foregoing requirements as they relate to beam or girder bridges may be exceeded at the discretion of the State Structure and Bridge Engineer.

10.6 DEFLECTION

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

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TABLE 10.3.2A Stress Cycles**

Main (Longitudinal) Load Carrying Members				
Type of Road	Case	ADTT ^a	Truck Loading	Lane Loading ^b
Freeways, Expressways, Major Highways, and Streets	I	2,500 or more	2,000,000 ^c	500,000
Freeways, Expressways, Major Highways, and Streets	II	less than 2,500	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 ^a	Truck Loading	
Freeway----				
Freeways, Expressways Major Highways, and Streets	II	less than 2,500	2,000,000	
Other Highways and Streets	III	---	500,000	

^aAverage Daily Truck Traffic (one direction)

^bLongitudinal members should also be checked for truck loading.

^cMembers should also be investigated for "over 2 million" stress cycles produced by placing a single truck on the bridge distributed to the girders as designated in Article 3.23.2 for one traffic lane loading. The shear in steel girder webs shall not exceed $0.58F_y D_t C$ for this single truck loading.

**See Article 10.3.2 Load Cycles.

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

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

10.6.7 The foregoing requirements as they relate to beam or girder bridges may be exceeded at the discretion of the State Structure and Bridge Engineer.*

10.7 LIMITING LENGTHS OF MEMBERS

10.7.1 For compression members, the slenderness ratio, KL/r , shall not exceed 120 for main members, or those in

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

TABLE 10.3.3A Temperature Zone Designations for Charpy-V Notch Impact Requirements

Minimum Service Temperature	Temperature Zone Designation
0°F and above	1
-1°F to 30°F (Virginia)	2
-31°F to -60°F	3

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

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

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

10.7.4 Actual unbraced length, L , shall be assumed as follows:

For the top chords of half-through trusses, the length between panel points laterally supported as indicated under Article 10.16.12; for other main members, the length between panel point intersections or centers of braced points or centers of end connections; for secondary members, the length between the centers of the end connections of such members or centers of braced points.

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

10.8 MINIMUM THICKNESS OF METAL

10.8.1 Structural steel (including bracing, cross frames, and all types of gusset plates), except for webs of certain rolled shapes, closed ribs in orthotropic decks, fillers, and

in railings, shall be not less than $\frac{5}{16}$ inch in thickness. The web thickness of rolled beams or channels shall not be less than 0.23 inches. The thickness of closed ribs in orthotropic decks shall not be less than $\frac{3}{16}$ inch.

10.8.2 Where the metal will be exposed to marked corrosive influences, it shall be increased in thickness of specially protected against corrosion.

10.8.3 It should be noted that there are other provisions in this section pertaining to thickness for fillers, segments of compression members, gusset plates, etc. As stated above, fillers need not be $\frac{5}{16}$ inch minimum.

10.8.4 For compression members, refer to "Trusses" (Article 10.16).

10.8.5 For stiffeners and other plates, refer to "Plate Girders" (article 10.34).

10.8.6 For stiffeners and outstanding legs of angles, etc., refer to Article 10.10.

10.9 EFFECTIVE AREA OF ANGLES AND TEE SECTIONS IN TENSION

10.9.1 The effective area of a single angle tension member, a tee section tension member, or each angle of a double angle tension member in which the shapes are connected back to back on the same side of a gusset plate shall be assumed as the net area of the connected leg or flange plus one-half of the area of the outstanding leg.

10.9.2 If a double angle or tee section tension member is connected with the angles or flanges back to back on opposite sides of a gusset plate, the full net area of the shapes shall be considered effective.

10.9.3 When angles connect to separate gusset plates, as in the case of a double-webbed truss, and the angles are connected by stay plates located as near the gusset as practicable, or by other adequate means, the full net area of the angles shall be considered effective. If the angles are not so connected, only 80 percent of the net areas shall be considered effective.

10.9.4 Lug angles may be considered as effective in transmitting stress, provided they are connected with at least one-third more fasteners than required by the stress to be carried by the lug angle.

10.10 OUTSTANDING LEGS OF ANGLES

The widths of outstanding legs of angles in compression (except where reinforced by plates) shall not exceed the following:

In main members carrying axial stress, 12 times the thickness.

In bracing and other secondary members, 16 times the thickness.

For other limitations, see Article 10.35.2

10.11 EXPANSION AND CONTRACTION

In all bridges, provisions shall be made in the design to resist thermal stresses induced, or means shall be provided for movement caused by temperature changes. Provisions shall be made for changes in length of span resulting from live load stresses. In spans more than 300 feet long, allowance shall be made for expansion and contraction in the floor. The expansion end shall be secured against lateral movement.

10.12 FLEXURAL MEMBERS

Flexural members shall be designed using the elastic section modulus except when utilizing compact sections under Strength Design as specified in Articles 10.48.1, 10.50.1.1, and 10.50.2.1.

10.13 COVER PLATES

10.13.1 The length of any cover plate added to a rolled beam shall not be less than $(2d+3)$ feet, where (d) is the depth of the beam in feet. Cover plates may be added to simple span rolled beams but shall not be used for continuous spans.

10.13.2 Partial length welded cover plates shall not be used on flanges more than 0.8 inches thick for nonredundant load path structures subjected to repetitive loadings that produce tension or reversal of stress in the member.

10.13.3 Welded cover plates shall be limited to one on any flange. The maximum thickness of a cover plate shall not be greater than two times the thickness of the flange to which the cover plate is attached. Minimum thickness shall be $\frac{1}{24}$ of the width of the plate, but in any case no less than $\frac{5}{8}$ ". The thickness of a cover plate may be varied by butt welding parts of different thickness, with transitions conforming to the requirements of Article 10.18. Such plates shall be assembled and welds ground smooth before attaching to the flange. Cover plates shall be $\frac{1}{2}$ " narrower than the flange to which they are attached.

10.13.4 Any partial length welded cover plate shall extend beyond the theoretical end by the terminal distance and shall extend to within 3 ft. of the centerline of bearing. The theoretical end of the cover plate, when using service load design methods, is the stress at which stress in the flange without that cover plate equals the allowable service

load stress, exclusive of fatigue considerations. When using the strength design methods, the theoretical end of the cover plate is the section at which the flange strength without that cover plate equals the required strength for the design loads, exclusive of fatigue requirements. The terminal distance is two times the nominal cover plate width for cover plates not welded across their ends, and 1½ times for cover plates welded across their ends. Cover plates shall be detailed with square ends. The weld connecting the cover plate to the flange in its terminal distance shall be continuous and of sufficient size to develop a total stress of not less than the computed stress in the cover plate at its theoretical end. All welds connecting cover plates to beam flanges shall be continuous and shall not be smaller than the minimum size permitted by Article 10.23.2.

10.13.5 Any partial length end-bolted cover plate shall extend beyond the theoretical end by a terminal distance equal to the length of the end-bolted portion, and the cover plate shall extend to a section where the stress range in the beam flange is equal to the allowable fatigue stress range for base metal at ends of partial length welded cover plates with high-strength bolted, slip-critical end connections (Table 10.3.1B). Beams with end-bolted cover plates shall be fabricated in the following sequence: drill holes; clean faying surfaces; install bolts; weld. The theoretical end of the end-bolted cover plate is determined in the same manner as that of a welded cover plate, as is specified in Article 10.13.4. The bolts in the slip-critical connections of the cover plate ends to the flange, shall be of sufficient numbers to develop a total force of not less than the computed force in the cover plate at the theoretical end. The slip resistance of the end-bolted connection shall be determined in accordance with Article 10.32.3.2 for service load design, and 10.56.1.4 for load factor design. The longitudinal welds connecting the cover plate to the beam flange shall be continuous and stop a distance equal to one bolt spacing before the first row of bolts in the end-bolted portion.

10.14 CAMBER

Girders should be cambered to compensate for dead load deflections and vertical curvature required by profile grade.

10.15 HEAT-CURVED ROLLER BEAMS AND WELDED PLATE GIRDERS

10.15.1 Scope

This section pertains to rolled beams and welded I-section plate girders heat-curved to obtain a horizontal curvature. Steels that are manufactured to a specified mini-

imum yield point greater than 50,000 psi shall not be heat-curved.

10.15.2 Minimum Radius of Curvature

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

$$R = \frac{14bD}{\sqrt{F_y \Psi} t_w} \tag{10-1}$$

$$R = \frac{7,500b}{F_y \Psi} \tag{10-2}$$

In these equations, F_y is the specified minimum yield point in kips per square inch of steel in the girder web, Ψ is the ratio of the total cross-sectional area to the cross-sectional area of both flanges, b is the widest flange width in inches, D is the clear distance between flanges in inches, t_w is the web thickness in inches, and R is the radius in inches.

10.15.2.2 In addition to the above requirements, the radius shall not be less than 1,000 feet when the flange thickness exceeds 3 inches or the flange width exceeds 30 inches.

10.15.3 Camber

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

$$\Delta = \frac{\Delta_{DL}}{\Delta_M} (\Delta_M + \Delta_R) \tag{10-3}$$

$$\Delta_R = \frac{0.02L^2 F_y}{E Y_o} \left(\frac{1,000 - R}{850} \right)$$

$$\Delta_R = 0 \text{ for radii greater than 1,000}$$

where Δ_{DL} is the camber in inches at any point along the length L calculated by usual procedures to compensate for deflection due to dead loads or any other specified loads; Δ_M is the maximum value of Δ_{DL} in inches within the length L ; E is the modulus of elasticity in ksi; F_y is the specified minimum yield point in ksi of the girder flange; Y_o is the distance from the neutral axis to the extreme

10.17.5 Bottom Struts

The bottom struts of towers shall be strong enough to slide the movable shoes with the structure unloaded, the coefficient of friction being assumed at 0.25. Provision for the expansion of the tower bracing shall be made in the column bearings.

10.18 SPLICES

10.18.1 General

10.18.1.1 The strength of members connected by high-strength bolts ~~and rivets~~ shall be determined by the gross section for compression members. For members primarily in bending, the gross section shall also be used, except that if more than 15 percent of each flange area is removed, that amount removed in excess of 15 percent shall be deducted from the gross area. In no case shall the design tensile stress on the net section exceed $0.50F_u$, when using service load design method or $1.0F_u$, when using strength design method, where F_u equals the minimum tensile strength of the steel, except that for M270 Grades 100/100W steels the design tensile stress on the net section shall not exceed $0.46F_u$ when using the service load design method. Splices may be made ~~with rivets~~, by high-strength bolts, or by the use of welding. Field splices shall be bolted; shop splices may be welded. Splices, except for curved girders (or beams), whether in tension, compression, bending, or shear, shall be designed in the case of service load design for a capacity of based on not less than the average of the calculated design stress at the point of splice and the allowable stress of the member at the same point but, in an event, not less than 75 percent of the allowable stress in the member. Splices in the case of strength design method shall be designed for not less than the average of the required strength at the point of splice and the strength of the member at the same point but, in any event not less than 75 percent of the strength of the member. Where a section changes at a splice, the smaller section is to be used for the above splice requirements.

Splices for curved girders (or beams) shall be designed for 100 percent of the capacity of the member. Where a section changes at a splice, the smaller section is to be used for the above splice requirements.

Where an option is noted on the plans specifying a thicker web to eliminate stiffeners, no change in the web splice design will be made for the thicker web.

Bolted splices shall be designed as slip-critical connections with high-strength bolts (A325). Splices shall be detailed with standard holes. An allowable load of 13.5 ksi (slip load per unit of bolted area) shall be used.

10.18.1.2 If splice plates are not in direct contact with the parts which they connect, the number of fasteners on each side of the joint shall be in excess of the number required for a direct contact splice to the extent of at least two extra transverse lines of fasteners for each intervening plate, except as provided in Article 10.18.1.3 and 10.18.6.

10.18.1.3 Fillers in high-strength bolted slip-critical connections need not be extended and developed, but eccentricity of forces at short, thick fillers must be considered.

10.18.1.4 ~~Riveted and~~ Bolted flange angle splices shall include two angles, one on each side of the flexural member.

10.18.2 Beams and Girders

10.18.2.1 Splices in built-up flexural members with webs greater than 48 inches in depth shall be proportioned as follows: The web splice shall be designed to resist the maximum shear including the effect of the eccentricity introduced by the splice connection.

Splices in rolled flexural members and built-up members having 48 inches or less shall be proportioned the same as built-up flexural members except that the web splice shall be designed to resist the maximum shear neglecting the effect of the eccentricity introduced by the splice connection.

10.18.2.2 The flange splice shall be designed to resist the entire force introduced by bending moment.

10.18.2.3 ~~As an alternate, splices of rolled flexural members may be proportioned for a shear equal to the actual maximum shear multiplied by the ratio of the splice design moment and the actual moment at the splice.~~

Web plates shall be spliced symmetrically by plates on each side. The splice plates for shear shall extend the full depth of the girder (or beam) between flanges. A minimum of two rows of bolts shall be on each side of the joint.

10.18.2.4 For ~~riveted and~~ bolted flexural members, splices in flange parts shall not be used between field splices except by special permission of the Engineer. In any one flange not more than one part shall be spliced at the same cross section. If practical, splices shall be located at points where there is an excess of section.

10.18.2.5 In continuous spans, splices preferably shall be made at or near points of contraflexure.

10.18.3 Columns

10.18.3.1 Compression members such as columns and chords shall have ends in close contact at ~~riveted and~~ bolted splices. Splices of such members which will be fabricated and erected with close inspection and detailed with milled ends in full contact bearing at the splices may be held in place by means of splice plates and ~~rivets or~~ high-strength bolts proportioned for not less than 50 percent of the lower allowable design stress of the sections spliced.

10.18.3.2 Splices in truss chords and columns shall be located as near to the panel points as practicable and usually on that side where the smaller occurs. The arrangement of plates, angles, or other splice elements shall be such as to make proper provision for the stresses, both axial and bending, in the component parts of the members spliced.

10.18.4 Tension Members

10.18.4.1 For tension members and splice material, the gross section shall be used unless the net section area is less than 85 percent of the corresponding gross area, in which case the amount removed in excess of 15 percent shall be deducted from the gross area.

10.18.4.2 In no case shall the design tensile stress on the net section exceed $0.50F_u$ when using service load design or $1.0F_u$ when using strength design method, where F_u equals the minimum tensile strength of the steel.

10.18.4.3 For M270 Grades 100/100W steels, the design tensile stress on net section shall not exceed $0.46F_u$ when using service load design method.

10.18.4.4 For calculating the net section, the provisions of Article 10.16.14 shall apply.

10.18.5 Welding

10.18.5.1 Tension and compression members may be spliced by means of full penetration welds, preferably without the use of splice plate.

10.18.5.2 ~~Welded field splices preferably should be arranged to minimize overhead welding.~~

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10.18.5.3 In welded splices any filler $\frac{1}{4}$ inch or more in thickness shall extend beyond the edges of the splice plate and shall be welded to the part on which it is fitted, with sufficient weld to transmit the splice plate load applied at the surface of the filler as an eccentric load.

10.18.5.4 The welds joining the splice plate to the filler shall be sufficient to transmit the splice plate load and shall be long enough to avoid overloading the filler along the toe of the weld. Any filler less than $\frac{1}{4}$ inch thick shall have its edges made flush with the edges of the splice plate. The weld size necessary to carry the splice plate load shall be increased by the thickness of the filler plate.

10.18.5.5 Material of different widths spliced by butt welds shall have transitions conforming to Figure 10.18.5A. The type transition selected shall be consistent with the Fatigue Stress Category from Table 10.3.1B for the Groove Welded Connection used in the design of the member. A butt-welded splices joining pieces of different thicknesses, there shall be a uniform slope between the offset surfaces, including the weld, of not more than 1 in 2½.

10.18.5.6 Changes in areas of flange plates or cover plates at welded splices shall ordinarily be made by changing the thickness while maintaining the same width. The ratio of thickness shall preferably be no more than 2:1.

Where it is desirable to change the width of a plate, the ratio of widths shall not exceed 4:3. If both the thickness and width are changed at the same point, the ratio of areas shall preferably be no more than 2:1.

10.18.6 Fillers

When fasteners carrying loads pass through fillers thicker than $\frac{1}{4}$ inch, except in high-strength bolted connections designed as slip-critical connections, the fillers shall be extended beyond the splice material and the filler extension shall be secured by enough additional fasteners to distribute the total stress in the member uniformly over the combined section of the member and the filler. As an alternate, an equivalent number of additional fasteners may be passed through the gusset or splice material without extending the filler. Fillers $\frac{1}{4}$ inch or more in thickness shall consist of not more than two plates, unless special permission is given by the Engineer.

10.18.7 Detailing Practices (Bolted Splices)

10.18.7.1 Beams and girders shall be detailed with

permissible field splices at appropriate points, when necessary, in order that they may be hauled. Lengths of beam or girder up to 135 feet may be hauled by truck on the Interstate of Arterial Systems.

10.18.7.2 Field splices in beams and girders shall be noted at permissible and detailed as bolted splices.

10.18.7.3 Bolted splices for curved beams and girders shall be detailed with a diaphragm (cross frame) as close as practical to the point of splice (normally within one to three feet).

10.19 STRENGTH OF CONNECTIONS

10.19.1 General

10.19.1.1 Except as otherwise provided herein, connections for main members shall be designed in the case of service load design for a capacity based on not less than the average of the calculated design stress in the member at the point of connection and the allowable stress of the member at the same point but, in any event, not less than 75 percent of the allowable stress in the member. Connections for main members in the case of load factor design shall be designed for not less than the average of the required strength at the point of connection and the strength of the member at the same point but, in any event, not less than 75 percent of the strength of the member.

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

10.19.1.3 Members, including bracing, preferably shall be so connected that their gravity axes will intersect in a point. Eccentric connections shall be avoided, if practicable, but if unavoidable the members shall be so proportioned that the combined fiber stresses will not exceed the allowed axial design stress.

10.19.1.4 In the case of connections which transfer total member shear at the end of the member, the gross section shall be taken as the gross section of the connected elements.

10.19.2 End Connections of Floor Beams and Stringers

10.19.2.1 The end connection shall be designed for the calculated member loads. The end connection angles

bracing shall be 3 by 2½ inches. There shall be not less than two fasteners or equivalent weld in each end connection of the angles.

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

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

10.22 CLOSED SECTIONS AND POCKETS

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

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

10.23 WELDING

10.23.1 General

10.23.1.1 Steel base to be welded, weld metal, and welding design details shall conform to the requirements of the *ANSI/AASHTO/AWS D1.5 Bridge Welding Code*.

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

~~**10.23.1.3** Fabrication shall conform to Article 11.4 Division II~~

10.23.2 Effective Size of Fillet Welds

10.23.2.1 Maximum Size of Fillet Welds

The maximum size of a fillet weld that may be assumed in the design of a connection shall be such that the stresses in the adjacent base material do not exceed the values allowed in Article 10.32. The maximum size

that may be used along the edges of connected parts shall be:

- (1) Along edges of material less than ¼ inch thick, the maximum size may be equal to the thickness of the material.
- (2) Along edges of material ¼ inch or more in thickness, the maximum size shall be 1/16 inch less than the thickness of the material, unless the weld is especially designated on the drawings to be built out to obtain full throat thickness.

10.23.2.2 Minimum Size of Fillet Welds

The minimum fillet weld size shall be as shown in the following table.**

Base Metal thickness of Thicker Part Jointed (T)		Minimum Size of fillet Weld*		
in.	mm	in	mm	
T ≤ 3/4	T ≤ 19.0	¼	6	} Single-pass Welds must be used
3/4 < T	19.0 < T	5/16	8	

*Except that the weld size need not exceed the thickness of the thinner part joined. For this exception, particular care should be taken to provide sufficient preheat to ensure weld soundness.

**Smaller fillet welds may be approved by the Engineer based upon applied stress and the use of appropriate preheat.

The minimum size of field weld shall be a 3/16" fillet weld.

10.23.3 Minimum Effect Length of Fillet Welds

The minimum effective length of a fillet weld shall be four times its size and in no case less than 1½ inches.

10.23.4 Fillet Weld End Returns

Fillet welds which support a tensile force that is not parallel to the axis of the weld, or which are proportioned to withstand repeated stress, shall not terminate at corners of parts or members but shall be returned continuously, full size, around the corner for a length equal to twice the weld size where such return can be made in the same plane. End returns shall be indicated on design and detail drawings.

10.23.5 Seal Welds

Seal welding shall preferably be accomplished by a continuous weld combining the functions of sealing and strength, changing section only as the required strength or the requirements of minimum size fillet weld, based on material thickness, may necessitate.

10.24.2.2.2 Short slotted holes may be used in any or all plies of high-strength bolted connections designed on the basis of Table 10.32.3B or Table 10.56A, as applicable, provided the load is applied approximately normal (between 80 and 100 degrees) to the axis of the slot. Short slotted holes may be used without regard for the direction of applied load in any or all plies of connections which satisfy the requirements of Article 10.32.3.2.1 or Article 10.57.3.1, as applicable

10.24.2.2.3 Long slotted holes may be used in one of the connected parts at any individual faying surface in high-strength bolted connections designed on the basis of Table 10.32.3B or Table 10.56A, as applicable, provided the load is applied approximately normal (between 80 and 100 degrees) to the axis of the slot. Long slotted holes may be used in one of the connected parts at any individual faying surface without regard for the direction of applied load on connections which satisfy the requirements of Article 10.32.3.2.1 or Article 10.57.3.1, as applicable.

10.24.3 Washer Requirements

Design details shall provide for washers in high-strength bolted connections as follows:

10.24.3.1 Where the outer face of the bolted parts has a slope greater than 1:20 with respect to a plane normal to the bolt axis, a hardened beveled washer shall be used to compensate for the lack of parallelism.

10.24.3.2 Hardened washers are not required for connection using AASHTO M 164 (ASTM A 325) and AASHTO M 253 (ASTM A 490) bolts except as required in Articles 10.24.3.3 through 10.24.3.7.

10.24.3.3 Hardened washers shall be used under the element turned in tightening when the tightening is to be performed by calibrated wrench method.

10.24.3.4 Irrespective of the tightening method, hardened washers shall be used under both the head and the nut when AASHTO M 253 (ASTM A 490) bolts are to be installed in material having a specified yield point less than 40 ksi.

10.24.3.5 Where AASHTO M 164 (ASTM A 325) bolts of any diameter or AASHTO M 253 (ASTM A 490) bolts equal to or less than 1 inch in diameter are to be installed in an oversize or short slotted hole in an outer ply, a hardened washer conforming to ASTM F 436 shall be used.

10.24.3.6 When AASHTO M 253 (ASMT A 490) bolts over 1 inch in diameter are to be installed in an over-

size or short slotted hole in an outer ply, hardened washers conforming to ASTM F 436 except with $\frac{5}{16}$ inch minimum thickness shall be used under both the head and the nut in lieu of standard thickness hardened washers. Multiple hardened washers with combined thickness equal to or greater than $\frac{5}{16}$ inch do not satisfy this requirement.

10.24.3.7 Where AASHTO M 164 (ASTM A 325) bolts of any diameter or AASHTO M 253 (ASTM A 490) bolts equal to or less than 1 inch in diameter are to be installed in a long slotted hole in an outer ply, a plate washer or continuous bar of at least $\frac{5}{16}$ inch thickness with standard holes shall be provided. These washers or bars shall have a size sufficient to completely cover the slot after installation and shall be of structural grade material, but need not be hardened except as follows. When AASHTO M 253 (ASTM A 490) bolts over 1 inch in diameter are to be used in long slotted holes in external plies, a single hardened washer conforming to ASTM F 436 but with $\frac{5}{16}$ inch minimum thickness shall be used in lieu of washers or bars of structural grade material. Multiple hardened washers with combined thickness equal to or greater than $\frac{5}{16}$ inch do not satisfy this requirement.

10.24.4 Size of Fasteners (~~Rivets or High-Strength Bolts~~)

10.24.4.1 Fasteners shall be of the size shown on the drawings, but generally shall be ~~$\frac{3}{4}$ inch~~ or $\frac{7}{8}$ inch in diameter. Fasteners $\frac{5}{8}$ inch in diameter shall not be used in members carrying calculated stress except in $2\frac{1}{2}$ inch legs of angles and in flanges of sections requiring $\frac{5}{8}$ inch fasteners.

10.24.4.2 The diameter of fasteners in angles carrying calculated stress shall not exceed one-fourth the width of the leg in which they are placed.

10.24.4.3 In angles whose size is not determined by calculated stress, $\frac{5}{8}$ -inch fasteners may be used in 2-inch legs, $\frac{3}{4}$ -inch fasteners in $2\frac{1}{2}$ -inch legs, $\frac{7}{8}$ -inch fasteners in 3-inch legs, and 1-inch fasteners in $3\frac{1}{2}$ -inch legs.

10.24.4.4 Structural shapes which do not admit the use of $\frac{5}{8}$ -inch diameter fasteners shall not be used except in handrails.

10.24.5 Spacing of Fasteners

10.24.5.1 Pitch and Gage of Fasteners

The pitch of fasteners is the distance along the line of principal stress, in inches, between centers of adjacent fas-

10.27.2 Packing of Eyebars

10.27.2.1 The eyebars of a set shall be symmetrical about the central plane of the truss and as nearly parallel as practicable. Bars shall be as close together as practicable and held against lateral movement, but they shall be so arranged that adjacent bars in the same panel will be separated by at least 1/2 inch.

10.27.2.2 Intersecting diagonal bars not far enough apart to clear each other at all times shall be clamped together at the intersection.

10.27.2.3 Steel filling rings shall be provided, if needed, to prevent lateral movement of eyebars or other members connected on the pin.

10.28 FORKED ENDS

Forked ends will be permitted only where unavoidable. There shall be enough pin plates on forked ends to make the section of each jaw equal to that of the member. The pin plates shall be long enough to develop the pin plate beyond the near edge of the stay plate, but not less than the length required by Article 10.25.4.

10.29 FIXED AND EXPANSION BEARINGS

10.29.1 General

10.29.1.1 Fixed ends shall be firmly anchored. Bearings for spans less than 50 feet need have no provision for deflection. Spans of 50 feet or greater shall be provided with a type of bearing employing a hinge, curved bearing plates, elastomeric pads, or pin arrangement for deflection purposes.

10.29.1.2 Spans of less than 50 feet may be arranged to slide upon metal plates with smooth surfaces and no provisions for deflection of the spans need be made. Spans of 50 feet and greater shall be provided with rollers, rockers, or sliding plates for expansion purposes and shall also be provided with a type of bearing employing a hinge, curved bearing plates, or pin arrangement for deflection purposes.

10.29.1.3 In lieu of the above requirements, elastomeric bearings may be used. See Section 14 of this specification.

10.29.2 Bronze or Copper-Alloy Sliding Expansion Bearings

Bronze or copper-alloy sliding plates shall be chamfered at the ends. They shall be held securely in position, usually by being inset into the metal of the pedestals or sole plates. Provisions shall be made against any accumulation of dirt which will obstruct free movement of the span.

10.29.3 Rollers

Expansion rollers shall be connected by substantial side bars and shall be guided by gearing or other effectual means to prevent lateral movement, skewing, and creeping. The rollers and bearing plates shall be protected from dirt and water as far as practicable, and the design shall be such that water will not be retained and that the roller nests may be inspected and clean easily.

10.29.4 Sole Plates and Masonry Plates

10.29.4.1 Sole plates and masonry plates shall have a minimum thickness of 3/4 inch.

10.29.4.2 For spans on inclined grades greater than 1 percent without hinged bearings, the sole plates shall be beveled so that the bottom of the sole plate is level, unless the bottom of the sole plate is radially curved.

10.29.5 Masonry Bearings

Beams, girders, or trusses on masonry shall be so supported that the bottom chords or flanges will be above the bridge seat, preferably not less than 6 inches.

10.29.6 Anchor Bolts

10.29.6.1 Trusses, girders, and rolled beam spans preferably shall be securely anchored to the substructure. Anchor bolts shall be swedged or threaded to secure a satisfactory grip upon the material used to embed them in the holes.

10.29.6.2 The following are the minimum requirements for each bearing:

~~For rolled beam spans the outer beams shall be anchored at each end with 2 bolts, 1 inch in diameter, set 10 inches in the masonry.~~

~~For trusses and girders:~~

Span Length	Number of bolts	Diameter of bolts (in.)	Length set in masonry (in.)
≤ 50'	2	1	10
51' – 100'	2	1 1/4	12
101' – 150'	2	1 1/2	15
> 150'	4	1 1/2	15

For continuous spans, the lengths shown above shall be considered as the length of the longer span supported by any one bearing; however, the total length contributing to horizontal thrust and uplift forces shall be considered in determining the number and size of anchor bolts or overturning of a fixed bearing.

if applicable), a construction load of 50 psf, and the weight of the form. The forms shall be designed to be elastic under construction loads. The elastic deformation caused by the dead load of the forms, plastic concrete and reinforcement,

shall not exceed a deflection of greater than L/180 or one-half inch (½"). For form work spans (L) or 10 feet (10') or less, or a deflection of L/240 or three-quarters inch (¾"), form work for spans L over 10 feet (10').

**Part C
SERVICE LOAD DESIGN METHOD**

ALLOWABLE STRESS DESIGN

10.31 SCOPE

Allowable stress design is a method for proportioning structural members using design loads and forces, allowable stresses, and design limitations for the appropriate material under service conditions. See Part D–Strength Design Method–Load Factor Design for an alternate design procedure.

10.32 ALLOWABLE STRESSES

10.32.1 Steel

Allowable stresses for steel shall be as specified in Table 10.32.1A.

10.32.2 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:

$$F_v = 0.27F_u \quad (10-12)$$

where,

- F_v = allowable basic shear stress;
- F_u = tensile strength of the electrode classification but not greater than the tensile strength of the connected part.

Size of weld (inches)	Strength of weld – pounds per inch		
	Grade 36	A709 Grade 50	Grade 50W
1/4	2770	3100	3340
5/16	3460	3880	4180
3/8	4150	4650	5010
1/2	5540	6200	6680
5/8	6920	7750	8350

For required sizes of welds, see Article 10.23.2.

When detailing fillet welds for quenched and tempered steels—the designer may use the electrode classifications with strengths less than the base metal provided that this requirement is clearly specified on the plans.

Plug Welds:

$F_v = 12,400$ psi for resistance to shear stresses only, where F_v = allowable basic shear stress.

10.32.3 Fasteners (Rivets and Bolts)

Allowable stresses for fasteners shall be as listed in Tables 10.32.3.A and 10.32.3.B, and the allowable force on a slip-critical connection shall be as provided by Article 10.32.3.2.1.

10.32.3.1 General

10.32.3.1.1 In proportioning fasteners for shear or tension, the cross-section area based upon the nominal diameter shall be used except as otherwise noted.

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

10.32.3.1.3 In determining whether the bolt threads are excluded from the shear planes of the contact surfaces, thread length of bolts shall be calculated as two thread pitches greater than the specified thread length as an allowance for thread runout.

10.32.3.1.4 In bearing-type connections, pull-out shear in a plate should be investigated between the end of the plate and the end row of fasteners. (See Table 10.32.3B, footnote g.)

10.32.3.1.5 All bolts except high-strength bolts, tensioned to the requirements of Division II. Table 11.5A

TABLE 10.32.5.1A Allowable Stresses—Cast Steel and Ductile Iron

AASHTO Designation	M 103	M 192	M 192	M 163	None
ASTM Designation	A27	A486	A 486	A 743	A 536
Class or Grade	70-36	70	90 120	CA 15	60-40-18
Minimum Yield Point, F_y	36,000		60,000 95,000	65,000	40,000
Axial Tension	14,500		22,500 34,000	24,000	16,000
Tension in Extreme Fibers	14,500		22,500 34,000	24,000	16,000
Axial Compression, Short Columns	20,000		30,000 45,000	32,000	22,000
Compression in Extreme Fibers	20,000		30,000 45,000	32,000	22,000
Shear	9,000		13,500 21,000	14,000	10,000
Bearing, Steel Parts in Contact	30,000		45,000 68,000	48,000	33,000
Bearing on Pins not Subject to Rotation	26,000		40,000 60,000	43,000	28,000
Bearing on Pins Subject to Rotation (such as used in rockers and hinges)	13,000		20,000 30,000	21,500	14,000

inches (average) beyond the edge of shoe or plate. Otherwise, the unit stresses permitted will be 75 percent of the above amounts.

10.32.6.3 For allowable unit-bearing stress on concrete masonry, refer to Article 8.15.2.1.3.

10.33 ROLLED BEAMS

10.33.1.1 General

10.33.1.1 Rolled beams, including those with welded cover plates, shall be designed by the moment of inertia method. Rolled beams with riveted cover plates shall be designed on the same basis as riveted plate girders.

10.33.1.2 The compression flanges of rolled beams supporting timber floors shall not be considered to be laterally supported by the flooring unless the floor and fastenings are specially designed to provide adequate support.

10.33.1.2 Bearing Stiffeners

Suitable stiffeners shall be provided to stiffen the webs of rolled beams at bearings when the unit shear in the web adjacent to the bearing exceeds 75 percent of the allowable shear for girder webs. See the related provisions of Article 10.34.6.

10.34 PLATE GIRDERS

10.34.1 General

10.34.1 Girders shall be proportioned by the moment of inertia method. For members primarily in bending, the entire gross section shall be used when calculating tensile and compressive stresses. Holes for high-strength bolts or rivets and/or open holes not exceeding 1¼ inches, may be neglected provided the area removed from each flange does not exceed 15 percent of that

flange. The area in excess of 15 percent shall be deducted from the gross area.

10.34.1.2 The compression flanges of plate girders supporting timber floors shall not be considered to be laterally supported by the flooring unless the floor and fastenings are specially designed to provide support.

10.34.2 Flanges

10.34.2.1 Welded Girders

10.34.2.1.1 Each flange may comprise a series of plates joined end to end by full penetration butt welds. Changes in flange areas may be accomplished by varying the thickness and/or width of the flange plate, or by adding cover plates. Where plates of varying thicknesses or widths are connected, the splice shall be made in accordance with Article 10.18 and welds ground smooth before attaching the web.

The minimum thickness of any flange plate shall be ¾”.

The maximum thickness of any flange plate shall be limited to 2½” if the flange plate is to be field welded (butt spliced). Otherwise, the maximum thickness of flange plates shall be in accordance with the limits noted in Table 10.32.1A.

The width at midspan of any compression flange shall preferably be not less than 1/85 of the unsupported length of the shipping piece for handling.

10.34.2.1.2 ~~When cover plates are used, they shall be designed in accordance with Article 10.13.~~

10.34.2.1.3 The ratio of compression flange plate width to thickness shall not exceed the value determined by the formula

$$\frac{b}{t} = \frac{3,250}{\sqrt{f_b}} \quad \text{but in no case shall } b/t \text{ exceed } 24 \quad (10-19)$$

10.34.2.1.4 Where the calculated compressive bending stress equals .55 F_y the (b/t) ratios for the various grades of steel shall not exceed the following:

- 36,000 psi, Y.P.Min. b/t=23
- 50,000 psi, Y.P.Min. b/t=20
- 70,000 psi, Y.P.Min. b/t=17
- 90,000 psi, Y.P.Min. b/t=15
- 100,000 psi, Y.P.Min. b/t=14

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10.37.3 Flange Plates

10.37.3.1 The b/t_f ratio for the width of flange plates between webs shall be not greater than

$$\frac{b'}{t_f} = \frac{4,250}{\sqrt{f_a + f_b}}, \text{ maximum } b/t_f = 47 \quad (10-56)$$

10.37.3.2 The b'/t_f ratio for the overhand width of flange plates shall be not greater than

$$\frac{b'}{t_f} = \frac{1,625}{\sqrt{f_a + f_b}}, \text{ maximum } b'/t_f = 12 \quad (10-57)$$

10.38 COMPOSITE GIRDERS

10.38.1 General

All simple spans with concrete deck slabs shall be designed as composite.

In continuous spans, positive moment areas shall be designed as composite sections. Distribution of moments from superimposed loads shall be made using the gross section of girder and slab throughout the entire length of the girder.

10.38.1.1 This section pertains to structures composed of steel girders with concrete slabs connected by shear connectors.

10.38.1.2 General specifications pertaining to the design of concrete and steel structures shall apply to structures utilizing composite girders where such specifications are applicable. Composite girders and slabs shall be designed and the stresses computed by the composite moment of inertia method and shall be consistent with the predetermined properties of the various materials used.

10.38.1.3 The ratio of the moduli of elasticity of steel (29,000,000 psi) to those of normal weight concrete ($W=145$ pcf) of various design strengths shall be as follows:

f'_c = unit ultimate compressive strength of concrete as determined by cylinder tests at the age of 28 days in pounds per square inch.

n = ratio of modulus of elasticity of steel to that of concrete. The value of n , as a function of the ultimate cylinder strength of concrete, shall be assumed as follows:

$f'_c =$	2,000 – 2,300	$n = 11$
	2,400 – 2,800	$= 10$
	2,900 – 3,500	$= 9$
	3,600 – 4,500	$= 8$
	4,600 – 5,900	$= 7$
	6,000 or more	$= 6$

10.38.1.4 The effect of creep shall be considered in the design of composite girders which have dead loads acting on the composite section. In such structures, stresses and horizontal shears produced by dead loads acting on the composite section shall be computed for n as given above or for this value multiplied by 3, whichever gives the higher stresses and shears.

10.38.1.5 If concrete with expansive characteristics is used, composite design should be used with caution and provision must be made in the design to accommodate the expansion.

10.38.1.6 Composite sections in simple spans and the positive moment regions of continuous spans should preferably be proportioned so that the neutral axis lies below the top surface of the steel beam. Concrete on the tension side of the neutral axis shall not be considered in calculating resisting moments. In the negative moment regions of continuous spans, only the slab reinforcement can be considered to act compositely with the steel beams in calculating resisting moments. Mechanical anchorages shall be provided in the composite regions to develop stresses on the plane joining the concrete and the steel. Concrete on the tension side of the neutral axis may be considered in computing moments of inertia for deflection calculations and for determining stiffness factors used in calculating moment and shears.

10.38.1.7 The steel beams or girders, especially if not supported by intermediate falsework, shall be investigated for stability and strength for the loading applied during the time the concrete is in place and before it has hardened. The casting or placing sequence specified in the plans for the composite concrete deck shall be considered when calculating the moments and shears on the steel section. The maximum flange compression stress shall not exceed the value specified in Table 10.32.1A for partially supported or unsupported compression flanges. The maximum shear in the web shall not exceed the shear-buckling capacity of the web nor the shear yield strength calculated as

$$F_v = \frac{CF_y}{3} \leq \frac{F_y}{3} \quad (10-57a)$$

where C is specified in Article 10.34.4.2.

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10.38.2 Shear Connectors

10.38.2.1 The mechanical means used at the junction of the girder and slab for the purpose of developing the shear resistance necessary to produce composite action shall conform to the specifications of the respective materials as provided in Division II. The shear connectors shall be of types that permit a thorough compaction of the concrete in order to ensure that their entire surfaces are in contact with the concrete. They shall be capable of resisting both horizontal and vertical movement between the concrete and the steel.

10.38.2.2 The capacity of stud and channel shear connectors welded to the girders is given in Article 10.38.5. Channel shear connectors shall have at least 3/16-inch fillet welds placed along the heel and toe of the channel.

10.38.2.3 The clear depth of concrete cover over the tops of the shear connectors shall be not less than 2 inches.

Shear connectors shall penetrate at least 2 inches above bottom of slab.

10.38.2.4 The clear distance between the edge of a girder flange and the edge of the shear connectors shall be not less than 1 inch. Adjacent stud shear connectors shall not be closer than 4 diameters center to center.

10.38.3 Effective Flange Width

10.38.3.1 In composite girder construction the assumed effective width of the slab as a T-beam flange shall not exceed the following:

- (1) One-fourth of the span length of the girder.
- (2) The distance center to center of girders.
- (3) Twelve times the least thickness of the slab.

10.38.3.2 For girders having a flange on one side only, the effective flange width shall not exceed one-twelfth of the span length of the girder, or six times the thickness of the slab, or one-half the distance center to center of the next girder.

10.38.4 Stresses

10.38.4.1 Maximum compressive and tensile stresses in girders that are not provided with temporary supports during the placing of the permanent dead load shall be the sum of the stresses produced by the dead loads acting on the steel girders alone and the stresses produced by the superimposed loads acting on the composite girder. When girders are provided with the effective intermediate supports that are kept in place until the concrete has attained 75 percent of its required 28-day strength, the dead and live load stresses shall be computed on the basis of the composite section.

10.38.4.2 Continuous composite bridges shall be designed with shear connectors throughout the length of the bridge. Shear connectors shall be provided in accordance with Article 10.38.5.

10.38.4.3 In the negative moment region(s) of continuous spans, the longitudinal reinforcement, including that required by Article 3.24.10, shall equal or exceed 1 percent of the cross-sectional area of the concrete slab. The reinforcement in excess of that required by Article 3.24.10 is to be placed in the top layer and shall be anchored beyond the points of dead load contra-flexure. Longitudinal reinforcement is not included in composite section properties.

10.38.4.4 (deleted)

10.38.5 Shear

10.38.5.1 Horizontal Shear

The maximum pitch of shear connectors shall not exceed 24". At simply supported ends (simple and continuous spans), shear connectors shall be as close as practicable to the end of the girder and have a maximum pitch of 6" within a distance of 36" from the end. Shear connectors shall be provided in the negative moment portion in which the reinforcing steel embedded in the concrete is considered a part of the composite section.

Resistance to horizontal shear shall be provided by mechanical shear connectors at the junction of the concrete slab and the steel girder. The shear connectors shall be mechanical devices placed transversely across the flange of the girder spaced at regular or variable intervals. The shear connectors shall be designed for fatigue* and checked for ultimate strength.

10.38.5.1.1 Fatigue

The range of horizontal shear shall be computed by the formula

$$S_r = \frac{V_r Q}{I} \quad (10-58)$$

where,

S_r = range of horizontal shear, in kips per inch, at the junction of the slab and girder at the point in the span under consideration.

V_r = range of shear due to live loads and impact in kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads);

Q = statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches.

I = moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

(In the formula, the compressive concrete area is transformed into an equivalent area of steel by dividing the effective concrete flange width by the modular ratio, n .)

*Reference is made to the paper titled "Fatigue Strength of Shear Connectors," by Robert G. Slutter and John W. Fisher, in *Highway Research Record No. 147*, published by the Highway Research Board, Washington, D.C., 1966.

The allowable range of horizontal shear, Z_r , in pounds on an individual connector is as follows:

Channels:
$$Z_r = Bw \quad (10-59)$$

Welded studs (for $H/d \geq 4$):
$$Z_r = \alpha d^2 \quad (10-60)$$

where,

w = length of a channel shear connector, in inches, measured in a transverse direction on the flange of a girder;

d = diameter of stud in inches.

α = 13,000 for 100,000 cycles
 10,600 for 500,000 cycles
 7,850 for 2,000,000 cycles
 5,500 for over 2,000,000 cycles

B = 4,000 for 100,000 cycles
 3,000 for 500,000 cycles
 2,400 for 2,000,000 cycles
 2,100 for over 2,000,000 cycles

H = height of stud in inches

The required pitch of shear connectors is determined by dividing the allowable range of horizontal shear of all connectors at one transverse girder cross-section (ΣZ_r) by the horizontal range of shear S_r . Over the interior supports of continuous beams the pitch may be modified to avoid placing the connectors at locations of high stresses in the tension flange provided that the total number of connectors remains unchanged.

For simple spans, S_r may be computed using V_r , Q and I at the supports and a uniform spacing of connectors used throughout the beam or girder, between the end group and midspan. Use the following values for Z_r :

No. of cycles	Z_r for $\frac{7}{8}''\phi$
500,000	8120
2,000,000	6010
over 2,000,000	4210

10.38.5.1.2 Ultimate Strength

The number of connectors so provided for fatigue shall be checked to ensure that adequate connectors are provided for ultimate strength.

The number of shear connectors required shall equal or exceed the number given by the formula:

$$N_1 = \frac{P}{\Phi S_u} \quad (10-61)$$

where:

N_1 = number of connectors between points of maximum positive moment and adjacent end supports.

S_u = ultimate strength of the shear connector as given below;

Φ = reduction factor = 0.85;

P = force in the slab as defined hereafter as P_1 or P_2 .

At points of maximum positive moment, the force in the slab is taken as the smaller value of the formulas:

$$P_1 = A_s F_y \quad (10-62)$$

or:

$$P_2 = 0.85 f'_c b t_s \quad (10-63)$$

where:

A_s = total area of the steel section including cover-plates;

F_y = specified minimum yield point of the steel being used;

f'_c = compressive strength of concrete at age of 28 days;

b = effective flange width given in Article 10.38.3;

t_s = thickness of the concrete slab.

The number of connectors, N_2 , required between the points of maximum positive moment and points of adjacent maximum negative moment shall equal or exceed the number given by the formula:

$$N_2 = \frac{P + P_3}{\Phi S_u} \quad (10-64)$$

At points of maximum negative moment the force in the slab is taken as:

$$P_3 = A_s^r F_y^* \quad (10-65)$$

where:

A_s^r = total area of longitudinal reinforcing steel at the interior support within the effective flange width.;

F_y^* = specified minimum yield point of the reinforcing steel.

*When reinforcement steel embedded in the top slab is not used in computing section properties for negative moments, P_3 is equal to zero.

The ultimate strength of the shear connector is given as follows:

Channels:

$$S_u = 550 \left(h + \frac{t}{2} \right) W \sqrt{f'_c} \quad (10-66)$$

Welded studs (for $H/d > 4$):

$$S_u = 0.4d_2 \sqrt{f'_c E_c} \quad (10-67)$$

Use the following value for S_u where $w = 145$ pounds per cu.ft. and $f'_c = 4000$ psi:

$$\text{For } \frac{7}{8} \phi, S_u = 36,970 \text{ lbs}$$

where:

E_c = modulus of elasticity of the concrete in pounds per square inch;

$$E_c = w^{3/2} 33 \sqrt{f'_c} \quad (10-68)$$

S_u = ultimate strength of individual shear connector in pounds;

H = average flange thickness of the channel flange in inches.

t = thickness of the web of a channel in inches;

W = length of a channel shear connectors in inches;

f'_c = compressive strength of the concrete in 28 days in pounds per square inch;

d = diameter of stud in inches;

w = unit weight of concrete in pounds per cubic foot.

10.38.5.1.3 Additional Connectors to Develop Slab Stresses

The number of additional connectors required at points of contraflexure when reinforcing steel embedded in the concrete is not used in computing section properties for negative moments shall be computed by the formula:

$$N_c = A_r^s f_r Z_r \quad (10-69)$$

where,

N_c = number of additional connectors for each beam at point of contraflexure;

A_r^s = total area of longitudinal slab reinforcing steel for each beam over interior support;

*When reinforcement steel embedded in the top slab is not used in computing section properties for negative moments, P_3 is equal to zero.

f_r = range of stress due to live load plus impact in the slab reinforcement over the support (in lieu of more accurate computations, f_r may be taken as equal to 10,000 psi);

Z_r = allowable range of horizontal shear on an individual shear connector.

The additional connectors, N_c , shall be placed adjacent to the point of dead load contraflexure within a distance equal to one-third the effective slab width, i.e., placed either side of this point or centered about it. It is preferable to locate field splices so that they clear the connectors.

10.38.5.2 Vertical Shear

The intensity of unit-shearing stress in a composite girder may be determined on the basis that the web of the steel girder carries the total external shear, neglecting the effects of the steel flanges and of the concrete slab. The shear may be assumed to be uniformly distributed throughout the gross area of the web.

10.38.6 Deflection

10.38.6.1 The provisions of Article 10.6 in regard to deflections from live load plus impact also shall be applicable to composite girders.

10.38.6.2 When the girders are not provided with falsework or other effective intermediate support during the placing of the concrete slab, the deflection due to the weight of the slab and other permanent dead loads added before the concrete has attained 75 percent of its required 28-day strength shall be computed on the basis of non-composite action.

10.39 COMPOSITE BOX GIRDERS

10.39.1 General

10.39.1.1 This section pertains to the design of simple and continuous bridges of moderate length supported by two or more single cell composite box girders. The distance center-to-center of flanges of each box should be the same and the average distance center-to-center of flanges of adjacent boxes shall be not greater than 1.2 times and not less than 0.8 times the distance center-to-center of flanges of each box. In addition to the above, when nonparallel girders are used, the distance center-to-center of adjacent flanges at supports shall be not greater than 1.35 times and not less than 0.65 times the distance center-to-center of flanges of each box. The cantilever overhand of the deck slab, including curbs and parapets, shall be limited to 60 percent of the average distance center-to-center

For sawn lumber beams, further adjustment by the shear stress factor may be applicable as described in the footnotes to Table 13.5.1A.

For structural composite lumber, more restrictive adjustments to the tabulated shear stress parallel to grain shall be as recommended by the material manufacturer.

13.6.6 Compression Perpendicular to Grain

13.6.6.1 General

When calculating the bearing stress in compression perpendicular to grain at beam ends, a uniform stress distribution shall be assumed.

13.6.6.2 Allowable Stress

The allowable unit stress in compression perpendicular to grain shall be the tabulated stress adjusted by the applicable adjustment factors given in the following equation:

$$F_{c\perp}' = F_{c\perp} C_M C_b \quad (13-12)$$

where

- $F_{c\perp}'$ = allowable unit stress in compression perpendicular to grain, in psi.
- $F_{c\perp}$ = tabulated unit stress in compression perpendicular to grain, in psi.
- C_M = wet service factor from Article 13.5.5.1;
- C_b = bearing area factor from Article 13.6.6.3.

13.6.6.3 Bearing Area Factor, C_b

Tabulated values in compression perpendicular to grain apply to bearings of any length at beam ends, and to all bearings 6 inches or more in length at any other location. For bearings less than 6 inches in length and not nearer than 3 inches to the end of a member, the tabulated values shall be adjusted by the bearing area factor, C_b , given by the following equation:

$$C_b = \frac{I_b + 0.375}{I_b} \quad (13-13)$$

where I_b is the length of bearing in inches, measured parallel to the wood grain. For round washers, or other round bearing areas, the length of bearing shall be the diameter of the bearing area.

The multiplying factors for bearing lengths on small areas such as plates and washers are given in Table 13.6.1A.

TABLE 13.6.1A Values of the Bearing Area Factor, C_b , for Small Bearing Areas

Length of Bearing, I_b (in.)	1/2	1	1-1/2	2	3	4	6 or more
Bearing Area Factor, C_b	1.75	1.38	1.25	1.19	1.13	1.10	1.00

13.6.7 Bearing on Inclined Surfaces

For bearing on an inclined surface, the allowable unit stress in bearing shall be as given by the following equation:

$$F_{\theta}' = \frac{F_g' F_{c\perp}'}{F_g' \sin^2 \theta + F_{c\perp}' \cos^2 \theta} \quad (13-14)$$

where:

- F_{θ}' = allowable unit stress for bearing on an inclined surface, in psi;
- F_g' = allowable unit stress in bearing parallel to grain from Article 13.7.4;
- $F_{c\perp}'$ = allowable unit stress in compression perpendicular to the grain from Article 13.6.6;
- θ = angle in degrees between the direction of load and the direction of grain.

13.7 COMPRESSION MEMBERS

13.7.1 General

13.7.1.1 The provisions of this article apply to simple solid columns consisting of a single piece of sawn lumber, piling, structural composite lumber, or glued laminated timber. Refer to the 1991 Edition of the NDS[®] for design requirements for built-up columns, consisting of a number of solid members joined together with mechanical fasteners, and for spaced columns consisting of two or more individual members with their longitudinal axes parallel, separated and fastened at the ends and at one or more interior points by blocking.

13.7.1.2 The term “column” refers to all types of compression members, including members forming part of a truss or other structural components.

13.7.1.3 Column bracing shall be provided where necessary to provide lateral stability and resist wind or other lateral forces.

TABLE 13.7.1A Support Condition Coefficients for Tapered Columns

Support Condition	Support Condition Coefficient, a
Large end fixed, small end unsupported	0.70
Small end fixed, large end unsupported	0.30
Both ends simply supported	
Tapered toward one end	0.50
Tapered towards both ends	0.70

where:

- F_c^* = tabulated stress in compression parallel to grain adjusted by all applicable modification factors given in Equation 13-14 except C_p ;
- K_{cE} = 0.300 for visually graded sawn lumber; 0.418 for glued laminated timber, structural composite lumber, and machine stress-rated lumber;
- c = 0.80 for sawn lumber
0.85 for round piles;
0.90 for glued laminated timber and structural composite lumber.

For especially severe service conditions or extraordinary hazardous conditions, the use of lower design values than those obtained above may be necessary. Refer to the 1991 Edition of the NDS[®].

13.7.3.4 Tapered Columns

13.7.3.4.1 For rectangular columns tapered at one or both ends, the cross-sectional area shall be based on the representative dimension of each tapered face. The representative dimension, d_{rep} , of each tapered face shall be based on the support condition coefficient given in Table 13.7.1a.

13.7.3.4.2 For support conditions given in Table 13.7.1A, the representative dimension d_{rep} , of each tapered face shall be as given by the following equation:

$$d_{rep} = d_{min} + (d_{max} - d_{min}) \left[a - 0.15 - \left(1 - \frac{d_{min}}{d_{max}} \right) \right] \quad (13-19)$$

where:

- d_{rep} = representative dimension for a tapered column face, in inches;
- d_{min} = minimum column face dimension, in inches;
- d_{max} = maximum column face dimension, in inches;
- a = coefficient based on support conditions.

13.7.3.4.3 For support conditions other than those in Table 13.7.1A, the representative dimension of each tapered face shall be as given by the following equation:

$$d_{rep} = d_{min} + 0.33(d_{max} - d_{min}) \quad (13-20)$$

13.7.3.4.4 For any tapered column, the actual stress in compression parallel to grain, f_c , shall not exceed the allowable stress determined by Equation 13-14, assuming the column stability factor $C_p = 1.0$.

13.7.3.5 Round Columns

The design of a round column shall be based on the design of a square column of the same cross-sectional area with the same degree of taper.

13.7.4 Bearing Parallel to Grain

13.7.4.1 The actual stress in bearing parallel to grain shall be based on the net area and shall not exceed the tabulated stress for bearing parallel to grain adjusted by the applicable adjustment factor given in the following equation:

$$F'_g = F_g C_D \quad (13-21)$$

where:

- F'_g = allowable unit stress in bearing parallel to grain in psi;
- F_g = tabulated unit stress in bearing parallel to grain from Table 13.5.2A, in psi;
- C_D = load duration factor from Article 13.5.5.2.

13.7.4.2 When the bearing load is at an angle to the grain, the allowable bearing stress shall be determined by Equation 13-14, using the design values for end-grain bearing parallel to grain and design values in compression perpendicular to grain.

13.7.4.3 When bearing parallel to grain exceeds 75% of the allowable value determined by Equation 13-21, bearing shall be on a metal plate or on other durable, rigid, homogeneous material of adequate strength and stiffness to distribute applied loads over the entire bearing area.

13.8 TENSION MEMBERS

13.8.1 Tension Parallel to Grain

The allowable unit stress in tension parallel to grain shall be the tabulated value adjusted by the applicable adjustment factors given in the following equation:

$$F'_t = F_t C_M C_D C_F \quad (13-22)$$

where

- D = diameter of the projection of the loaded surface of the bearing in the horizontal plane (in)
- W = length of the cylinder (in)

- σ_{PFE} = maximum average contact stress permitted on the PTFE by Table 14.6.2.4-1.
- ψ = subtended semi-angle of the curved surface.

The two surfaces of a sliding interface shall have equal radii.

14.6.3.2 Resistance to Lateral Load

In bearings which are required to resist horizontal loads, either an external restraint system shall be provided, or for a cylindrical sliding surface the horizontal load shall be limited to

$$H_m \leq 2RW \sigma_{PTFE} \sin(\Psi - \beta - \theta_m) \sin \beta \quad (14.6.3.2-1)$$

and for a spherical surface the horizontal load shall satisfy

$$H_m \leq \pi R^2 \sigma_{PTFE} \sin^2(\psi - \beta - \theta_m) \sin \beta \quad (14.6.3.2-2)$$

Where:

$$\beta = \tan^{-1} \left(\frac{H_m}{P_D} \right) \quad (14.6.3.2-3)$$

and

$$\psi = \sin^{-1} \left(\frac{L}{2R} \right) \quad (14.6.3.2-4)$$

and:

- H_m = maximum horizontal load.
- L = projected length of the sliding surface perpendicular to the rotation axis.
- P_D = compressive load due to permanent loads.
- R = radius of curved sliding surface.
- w = length of the cylindrical surface.
- β = angle between the vertical and applied loads.
- θ_m = maximum design rotation angle. See Article 14.4.1

14.6.4 Pot Bearings

14.6.4.1 General

See VDOT Special Provision 408.3, High Load Multi-Rotational Bearings and Manual of the Structure and Bridge Division, Volume V - Part 3 (also Part 3M), standard drawings BBD-6 and BBD-7 (Notes to Designer) for design parameters.

Where pot bearings are provided with a PTFE slider to provide for both rotation and horizontal movement, such slicing surfaces and any guidance systems shall be designed in accordance with the appropriate Articles 14.6.2 and 14.6.9.

The rotational elements of pot bearings shall satisfy the requirements of this section. They shall consist of at least a pot, a piston, an elastomeric disc, and sealing rings.

For the purpose of establishing the forces and deformations imposed on a pot bearing, the axis of rotation shall be taken as lying in the horizontal plane at mid-height of the elastomeric disc.

14.6.4.2 Materials

The elastomeric disc shall be made from a compound based on virgin natural rubber or virgin neoprene. Its nominal hardness shall lie between 50 and 60 on the Shore 'A' scale.

The pot and piston shall be made from structural steel conforming to AASHTO M 270 (ASTM A 709) Grades 36, 50 or 50W, or from stainless steel conforming to ASTM A 240. The finish of surfaces in contact

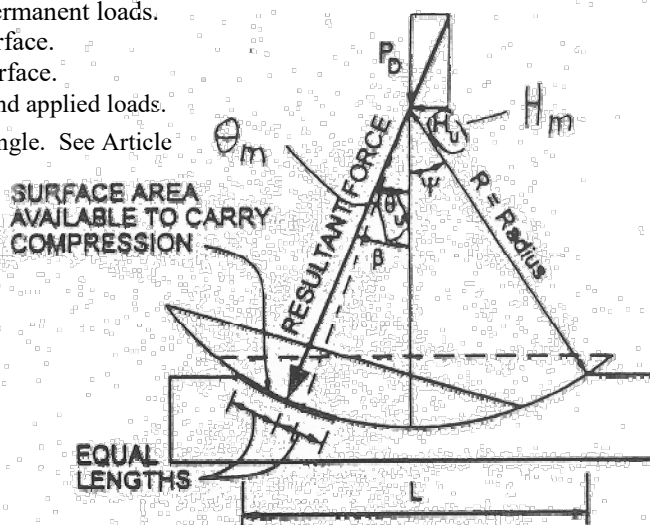


FIGURE 14.6.3.2-1

where

- D_p = internal diameter of pot (in)
- w = height of piston rim(in)
- θ_m = design rotation specified in Article 14.4.1 (rad)

14.6.4.8 Lateral Loads

Pot bearings which are subjected to lateral loads shall be proportioned so that the thickness, t , of the pot wall and the pot base shall satisfy

$$t \geq \sqrt{\frac{40H_m \theta_m}{F_y}} \quad (14.6.4.8-1)$$

For pot bearings which transfer lateral load through the piston

$$w \geq \frac{2.5H_m}{D_p F_y} \quad (14.6.4.8-2)$$

and

$$w \geq \frac{1}{8}''$$

where w is the rim thickness of the piston which is in contact with the pot wall.

~~**14.6.5 Steel Reinforced Elastomeric Bearings Method B**~~

~~**14.6.5.1 General**~~

~~Steel reinforced elastomeric bearings shall consist of alternate layers of steel reinforcement and elastomer, bonded together. Tapered elastomer layers shall not be used. All internal layers of elastomer shall be of the same thickness. The top and bottom cover layers shall be no thicker than 70% of the internal layers. In addition to any internal reinforcement, bearings may have external steel~~

NOTE:

Method B is not used by VDOT. Method A is used by VDOT to design steel reinforced elastomeric bearings. See Article 14.6.6. A computer program is available to assist in the design of these bearings.

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load plates bonded to the upper or lower elastomer layers or both.

14.6.5.2 Material Properties

The elastomer shall have a shear modulus between 0.08 and 0.175 ksi and a nominal hardness between 50 and 60 on the Shore A scale.

The shear modulus of the elastomer at 73°F shall be used as the basis for design. If the elastomer is specified explicitly by its shear modulus, then that value shall be used in design and the other properties shall be obtained from Table 14.6.5.2-1. If the material is specified by its hardness, the shear modulus shall be taken as the least favorable value from the range for that hardness given in Table 14.6.5.2-1. Intermediate values shall in all cases be obtained by interpolation.

For the purposes of bearing design, all bridge sites shall be classified as being in temperature Zones A, B, C, D or E. Characteristics for each zone are given in Table 14.6.5.2-2. In the absence of more precise information, Figure 14.6.5.2-2 may be used as a guide in selecting the zone required for a given region.

Bearings shall be made from AASHTO low temperature grades of elastomer as defined in Section 18 of Division II. The minimum grade of elastomer required for each low temperature zone is specified in Table 14.6.5.2-2.

Any of the three design options listed below may be used:

- specify the elastomer with the minimum low temperature grade indicated in Table 14.6.5.2-2 and determine the shear force transmitted by the bearing as specified in Article 14.5.3.1
- specify the elastomer with the minimum low temperature grade for use when special force provisions are incorporated in the design and provide a low friction sliding surface, in which case the special force provision is that the bridge components shall be designed to withstand twice the design shear force specified in Article 14.5.3.1, or
- specify the elastomer with the minimum low temperature grade for use when special force provisions are incorporated in the design, but do not provide a

Table 14.6.5.2-1 Elastomer properties at different hardnesses.

Hardness (Shore 'A')	50	60	70
Shear modulus at 73°F (psi)	95-130	130-200	200-300
Creep deflection at 25 yrs			
Instantaneous deflection	25%	35%	45%

low friction sliding surface, in which case the components of the bridge shall be designed to resist four times the design shear force as specified in Article 14.5.3.1.

14.6.5.3 Design Requirements

14.6.5.3.1 Scope

Bearings designed by the provisions of this section shall be subsequently tested in accordance with the requirements for steel reinforced elastomeric bearings of Article 18.7 of Division II of this Specification. Steel reinforced elastomeric bearings may also be designed under the provisions of Article 14.6.6

14.6.5.3.2 Compressive stress

In any bearing layer, the average compressive stress (ksi) shall satisfy the following:

- for bearings subject to shear deformation

$$\begin{aligned} \sigma_{TL} &\leq 1.6\text{KSI} \\ \sigma_{TL} &\leq 1.66\text{GS} \\ \sigma_L &\leq 0.66\text{GS} \end{aligned} \quad (14.6.5.3.2-1)$$

- for bearings fixed against shear deformation

$$\begin{aligned} \sigma_{TL} &\leq 1.75\text{KSI} \\ \sigma_{TL} &\leq 2.00\text{GS} \\ \sigma_L &\leq 1.00\text{GS} \end{aligned} \quad (14.6.5.3.2-2)$$

where

- σ_L = average compressive stress due to the live load (KSI)
- σ_{TL} = average compressive stress due to total dead plus live load (KSI)
- G = shear modulus of elastomer (KSI)
- S = shape factor of the thickest layer of the bearing

14.6.5.3.3 Compressive Deflection

Deflections due to total load and to live load alone shall be considered separately.

Instantaneous deflection shall be calculated as follows:

$$\delta = \sum \epsilon_i h_{ri} \quad (14.6.5.3.3-1)$$

where:

- ϵ_i = instantaneous compressive strain in the i^{th} elastomer layer of a laminated elastomeric bearing
- h_{ri} = thickness of i^{th} elastomeric layer in elastomeric bearing (in)

Table 14.6.52.2 Low temperature zones and elastomer grades

Low Temperature Zone	A	B	C	D	E
50 year low temperature (*F)	0	-20	-30	-45	all others
Max. no. of days below 32°F	3	7	14	N/A	N/A
Low temp. elastomer grade without special provision	0	2	3	4	5
Low temp. elastomer grade with special provisions	0	0	2	3	5

Values for ϵ_i shall be determined from test results or by rational analysis. The effects of creep of the elastomer shall be added to the instantaneous deflection when considering long-term deflections. They should be computed from information relevant to the elastomeric compound used. In the absence of material-specific data, the values given in Article 14.6.5.2 shall be used.

14.6.5.3.4 Shear

The horizontal movement of the bridge superstructure, Δ_o , shall be taken as the maximum possible displacement caused by creep, shrinkage, post-tensioning, combined with thermal effects computed in accordance with this Specification. The maximum shear deformation of the bearing, Δ_s , shall be taken as Δ_o , modified to account for the pier flexibility and construction procedures. If a low friction surface is installed, Δ_s , need not be taken larger than the deformation corresponding to first slip.

The bearing shall be designed so that

$$h_{ri} \geq 2\Delta_s \quad (14.6.5.3.4-1)$$

where

- h_{ri} = total elastomeric thickness (in)
- Δ_s = maximum service shear deformation of the elastomer (in)

14.6.5.3.5 Combined Compression and Rotation

Rotations shall be taken as the maximum possible difference in slope between the top and bottom surfaces of the bearing. They shall include the effects of initial lack-of-parallelism and subsequent girder end rotation due to imposed loads and movements. Bearings shall be designed so that uplift does not occur under any combination of loads and corresponding rotation.

All rectangular bearings shall satisfy

$$\sigma_{TL} \geq 1.0GS \left(\frac{\theta_m}{n} \right) \left(\frac{B}{h_{ri}} \right)^2 \quad (14.6.5.3.5-1)$$

A rectangular bearing subject to shear deformation shall also satisfy Equation 14.6.5.3.5-2; those fixed against shear deformation shall also satisfy Equation 14.6.5.3.5-3.

$$\sigma_{TL} \leq 1.875GS \left(1 - 0.200 \left(\frac{\theta_m}{n} \right) \left(\frac{B}{h_{ri}} \right)^2 \right) \quad (14.6.5.3.5-2)$$

$$\sigma_{TL} \leq 2.250GS \left(1 - 0.167 \left(\frac{\theta_m}{n} \right) \left(\frac{B}{h_{ri}} \right)^2 \right) \quad (14.6.5.3.5-3)$$

where

- B = length of pad if rotation is about its transverse axis, or width of pad if rotation is about its longitudinal axis (in)

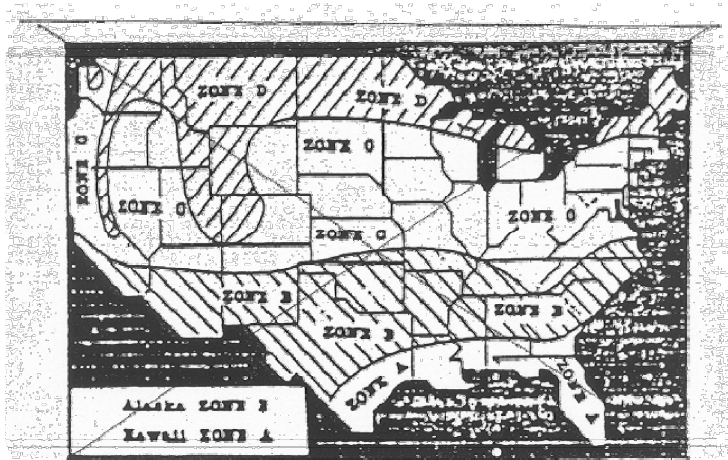


FIGURE 14.6.52-1 Map of Low Temperature Zones

- G = shear modulus of elastomer (ksi)
- h_{ri} = thickness of the i^{th} layer of elastomer (in)
- n = number of layers of elastomer
- S = shape factor of the thickest layer of the bearing
- θ_m = component of maximum service rotation in direction of interest (rad)
- σ_{TL} = average compressive stress due to the total dead plus live load (ksi)

All circular bearings shall satisfy

$$\sigma_{TL} < 0.75GS \left(\frac{\theta_m}{n} \right) \left(\frac{D}{h_{ri}} \right)^2 \quad (14.6.5.3.5-4)$$

A circular bearing subject to shear deformation shall also satisfy Equation 14.6.5.3.5-5; those fixed against shear deformation shall also satisfy Equation 14.6.5.3.5-6.

$$\sigma_{TL} < 2.5GS \left(1 - 0.15 \left(\frac{\theta_m}{n} \right) \left(\frac{D}{h_{ri}} \right)^2 \right) \quad (14.6.5.3.5-5)$$

$$\sigma_{TL} < 3.0GS \left(1 - 0.125 \left(\frac{\theta_m}{n} \right) \left(\frac{D}{h_{ri}} \right)^2 \right) \quad (14.6.5.3.5-6)$$

where

D = diameter of pad (in)

14.6.5.3.6 Stability

Bearings shall be proportioned to avoid instability. If

$$\frac{3.84(h_{ri}/L)}{S\sqrt{1+2L/w}} \leq \frac{2.67}{S(S+2)(1+L/4w)} \quad (14.6.5.3.6-1)$$

the bearing is stable for all allowable loads in this specification and no further consideration of stability is required.

For rectangular bearings not satisfying equation 14.6.5.3.6-1, the average compressive stress due to dead and live load shall satisfy:

- if the bridge deck is free to translate horizontally

$$\sigma_{TL} \leq \frac{G}{\left(\frac{3.84(h_{ri}/L)}{S\sqrt{1+2L/W}} - \frac{2.67}{S(S+2)(1+L/4W)} \right)} \quad (14.6.5.3.6-2)$$

- if the bridge deck is not free to translate horizontally

$$\sigma_{TL} \leq \frac{G}{\left(\frac{1.92(h_{ri}/L)}{S\sqrt{1+2L/W}} - \frac{2.67}{S(S+2)(1+L/4W)} \right)} \quad (14.6.5.3.6-3)$$

If L is greater than W for a rectangular bearing, stability shall be checked by the above formulas with L and W interchanged.

For circular bearings, stability may be evaluated by using the equations for a square bearing with $W=L=0.8D$.

14.6.5.3.7 Reinforcement

The thickness of the reinforcement h_s shall satisfy the requirements

$$h_s > \frac{3.0h_{rmax}\sigma_{TL}}{F_{sr}} \quad (14.6.5.3.7-1)$$

and

$$h_s > \frac{2.0h_{rmax}\sigma_L}{F_{sr}} \quad (14.6.5.3.7-2)$$

where

h_s = thickness of steel laminate (in)

F_{sr} = allowable fatigue stress range for over 2,000,000 cycles (ksi)

If holes exist in the reinforcement, the minimum thickness shall be increased by a factor of 2 (gross width)/(net width).

14.6.6 Elastomeric Pads and Steel Reinforced Elastomeric Bearings – Method A

14.6.6.1 General

This section of the specification covers the design of plain elastomeric pads, PEP, pads reinforced with discrete layers of fiberglass, FGP, and pads reinforced with closely spaced layers of cotton duck; CDP and steel reinforced elastomeric bearings. Layer thickness in FGP may be different from one another. For steel reinforced elastomeric bearings designed in accordance with the provisions of this section, internal layers shall be the same thickness and cover layers shall be no more than 70% of the thickness of internal layers.

14.6.6.2 Material Properties

The materials shall satisfy the requirements of Article 14.6.5.2, except that the shear modulus shall lie between 0.080 and 0.250 ksi and the nominal hardness shall lie between 50 and 70 on the Shore 'A' scale. This exception shall not apply to steel reinforced elastomeric bearings designed in accordance with the provisions of this section.

than that of the elastomer at 73°F. Effects of relaxation shall be ignored.

If the design shear force, H_m , due to pad deformation exceeds one-fifth of the minimum vertical force, the pad shall be secured against horizontal movement.

The pad shall not be permitted to sustain uplift forces.

14.6.7 Bronze or Copper Alloy Sliding Surfaces

Bronze or Copper Alloy may be used in

- flat sliding surfaces to accommodate translational movements,
- curved sliding surfaces to accommodate translation and limited rotation,
- pins or cylinders for shaft bushings of rocker bearings or other bearings with large rotations.

14.6.7.1 Materials

Bronze sliding surfaces or castings shall conform to AASHTO M 107 (ASTM B 22) and shall be made of Alloy C90500, C91100 or C86300 unless otherwise specified. The mating surface shall be structural steel which has a Rockwell hardness value at least 100 points greater than that of the bronze.

Cooper alloy 913 or 911 or copper alloy plates, AASHTO M 108 (ASTM B 100), shall be used unless otherwise specified.

14.6.7.2 Coefficient of Friction

The design coefficient of friction shall be determined by applying an appropriate safety factor to the measured coefficient of friction obtained using a rational test procedure. In lieu of such test data, the design coefficient of friction may be taken as 0.1 for self-lubricating bronze components and 0.4 for other types.

14.6.7.3 Limits on Load and Geometry

The nominal bearing stress due to combined dead and live load shall be no greater than 2.0 ksi.

14.6.7.4 Clearances and Mating Surface

The mating surface shall be steel which is accurately machined to match the geometry of the bronze surface and provide uniform bearing and contact.

14.6.8 Disc Bearings

14.6.8.1 General

See VDOT special Provision 408.3, High Load Multi-Rotational Bearings and Manual of the Structure and Bridge Division, Volume V – Part 3 (also Part 3M), standard drawings BBD-6 and BBD-7 (Notes to Designer) for design parameters.

For the purpose of establishing the forces and deformations imposed on a disc bearing, the axis of rotation may be taken as lying in the horizontal plane at mid-height of the disc. The urethane disc shall be held in place by a positive location device.

The disc bearing shall be designed for the design rotation θ_m defined in Article 14.4.1.

14.6.8.2 Materials

The elastomeric disc shall be made from a compound based on polyether urethane, using only virgin materials. The hardness shall lie between 45 and 65 on the shore D scale.

The metal components of the bearing shall be made from structural steel conforming to AASHTO M 270 (ASTM A 709) Grades 36, 50, or 50W, or from stainless steel conforming to ASTM A 240.

14.6.8.3 Overall Geometric Requirements

The dimensions of the components shall be such that hard contact between metal components which prevents further displacement or rotation will not occur under the least favorable combination of design displacements and rotations.

14.6.8.4 Elastomeric Disc

The elastomeric disc shall be held in location by a positive locator device. The disc shall be designed so that

- its instantaneous deflection under total load does not exceed 10% of the thickness of the unstressed disc, and the additional deflection due to creep does not exceed 8% of the thickness of the unstressed disc;
- the average compressive stress due to the maximum load, P_m , on the disc does not exceed 5.0 ksi. If the outer surface of the disc is not vertical, the stress shall be computed using the smallest plan area of the disc.

If a PTFE slider is used

- the stresses on the PTFE slider do not exceed 75% of the allowable values for average and edge stresses given in Article 14.6.2. The effect of moments induced by the urethane disc shall be included in the stress analysis.

14.6.8.5 Shear Resisting Mechanism

In fixed and guided bearings, a shear-resisting mechanism shall be provided to transmit horizontal forces between

the upper and lower steel plates. It shall be capable of resisting a horizontal force in any direction equal to the larger of the design shear force and 10% of the design vertical load.

The horizontal design clearance between the upper and lower components of the shear-restricting mechanism shall not exceed the value for guide bars given in Article 14.6.9.

14.6.8.6 Steel Plates

The thickness of the upper and lower steel plates shall not be less than $0.045 D_d$ if the plate is in direct contact with a steel girder or distribution plate, or $0.06 D_d$ if it bears directly on grout or concrete.

14.6.9 Guides and restraints

14.6.9.1 General

Guides may be used to prevent movement in one direction. Restraints may be used to permit only limited movement in one or more directions. Guides and restraints shall have a low-friction material at their sliding contact surfaces.

14.6.9.2 Design Loads

The guide or restraint shall be designed using the maximum load combinations for the larger of

- the horizontal design load, or
- 10% of the maximum vertical load acting on all the bearings at the bent divided by the number of guided bearings at the bent.

14.6.9.3 Materials

For steel bearings, the guide or restraint shall be made from steel conforming to AASHTO M 270 (ASTM A 709) Grades 36, 50 or 50W, or stainless steel conforming to ASTM A 240. The guide for aluminum bearings may also be aluminum.

The low-friction interface material shall be approved by the Engineer.

14.6.9.4 Geometric Requirements

Guides shall be parallel, long enough to accommodate the full design displacement of the bearing in the sliding direction, and shall permit a minimum of $1/32$ -in and a maximum of $1/16$ -in free slip in the restrained direction. Guides shall be designed to avoid binding under all design loads and displacements, including rotations.

14.6.95 Design Basis

14.6.9.5.1 Load Location

The horizontal load acting on the guide or restraint shall be assumed to act at the centroid of the low-friction interface material. Design of the connection between the guide or restraint and the body of the bearing system shall take into account both shear and overturning moment.

14.6.9.5.2 Contact stress

The contact stress on the low-friction material shall not exceed that recommended by the manufacturer. For PTFE, the stresses due to the maximum loads, P_m and H_m , shall not exceed those given in Table 14.6.2.4.1 under sustained loading or 1.25 times those stresses for short-term loading.

14.6.9.6 Attachment of Low-Friction Material

The low-friction material shall be attached by at least two of the following three methods:

- mechanical fastening
- bonding
- mechanical interlocking with a metal substrate.

14.6.10 Other Bearing Systems

Bearing systems made from components not described in Articles 14.6.1 through 14.6.8 may also be used, subject to the approval of the Engineer. Such bearings shall be adequate to resist the forces and deformations imposed on them without material distress and without inducing deformations large enough to threaten their proper functioning.

The dimensions of the bearing shall be chosen to provide for adequate movements at all times. The materials used shall have sufficient strength, stiffness, and resistance to creep and decay to ensure the proper functioning of the bearing throughout the design life of the bridge.

The Engineer shall determine the tests which the bearing must satisfy. The tests shall be designed to demonstrate any potential weakness in the system under individual compression, shear or rotational loading or combinations thereof. Testing under sustained or cyclic loading shall be required.

14.7 LOAD PLATES AND ANCHORAGE FOR BEARINGS

14.7.1 Plates for Load Distribution

The bearing, together with any additional plates, shall be designed so that

- the combined system is stiff enough to prevent distortions of the bearing which would impair its proper functioning;
- the stresses imposed on the supporting structure satisfy the limits specified by the Engineer. Allowable stresses on concrete and grout beds shall be assumed.

17.5.3.5 Footing Design

Design shall include consideration of differential horizontal and vertical movements and footing rotations. Footing design shall conform to Article 4.4.

17.6 REINFORCED CONCRETE BOX, CAST-IN-PLACE

17.6.1 Application

This specification is intended for use in the design of cast-in-place reinforced concrete box culverts.

17.6.2 Materials

17.6.2.1 Concrete

Concrete shall conform to Article 8.2 except that evaluation of f'_c may be based on test beams.

17.6.2.2 Reinforcement

Reinforcement shall meet the requirements of Article 8.3 except that for welded wire fabric a yield strength of 65,000 psi may be used. For wire fabric, the spacing of longitudinal wires shall be a maximum of 8 inches.

17.6.3 Concrete Cover for Reinforcement

The minimum concrete cover for reinforcement shall conform to Article 8.22. ~~The top slab shall be considered a bridge slab for concrete cover considerations.~~

17.6.4 Design

17.6.4.1 General Requirements

Designs shall conform to applicable sections of these specifications except as provided otherwise in this Section. For design loads and loading conditions see Section 3. For distribution of concentrated loads through earth for culverts with less than 2 feet of cover, see Article 3.24.3, Case B, and for requirements for bottom distribution reinforcement in top slabs of such culverts see Article 3.24.10. For distribution of wheel loads to culverts with 2 feet or more of cover see Article 6.4. For reinforced concrete design requirements, see Section 8.

17.6.4.2 Modification of Earth Loads for Soil Structure Interaction

The effects of soil structure interaction shall be taken into account and shall be based on the design earth cover, sidefill compaction, and bedding characteristics. These parameters may be determined by a soil-structure interaction analysis of the system. The loads given in Article 6.2 may be used, if they are multiplied by a soil-structure interaction factor, F_{cs} , that accounts for the type and conditions of installation as defined in Figure 17.6A, so that the total earth load, W_E on the box section is

$$W_E = F_{cs} W_B H \quad (17-16)$$

F_{cs} may be determined by the Marston-Spangler Theory of earth loads, as follows

17.6.4.2.1 Embankment Installations

$$F_{ci} = 1 + 0.20 \frac{H}{B_c} \quad (17-17)$$

F_{ci} need not be greater than 1.15 for installations with compacted fill at the sides of the box section, and need not be greater than 1.4 for installations with uncompacted fill at the sides of the box section.

17.6.4.2.2 Trench Installations

$$F_{e2} = \frac{C_d B_d^2}{H B_c}$$

Values of C_d can be obtained from Figure 17.4B for normally encountered soils. The maximum value of F_{e2} need not exceed F_{e1} .

The soil-structure interaction factor F_{cs} is not applicable if the Service Load Design Method is used.

17.6.4.3 Distribution of Concentrated Load Effects to Bottom Slab

The width of top slab strip used for distribution of concentrated wheel loads may be increased by twice the box height and used for the distribution of loads to the bottom slab.

17.6.4.4 Distribution of Concentrated Loads in Skewed Culverts

Wheel loads on skewed culverts shall be distributed using the same provisions as given for culverts with main reinforcement parallel to traffic.

17.7.3 Concrete Cover for Reinforcement

The minimum concrete cover for reinforcement shall be provided in accordance with Article 8.22. Except in slabs as noted in Article 8.22.6, the specified cover may be reduced by not more than $\frac{1}{2}$ inch, provided the reinforcement is epoxy-coated or that the concrete surfaces adjacent to the reinforcement are waterproofed conforming to Section 416 of the VDOT Road and Bridge Specifications

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Section 3

GENERAL REQUIREMENTS

3.1 APPLICABILITY OF SPECIFICATIONS

These Specifications are for the design and construction of new bridges to resist the effect of earthquake motions. The provisions apply to bridges of conventional steel and concrete girder and box girder construction with spans not exceeding 500 ft (150 m). Suspension bridges, cable-stayed bridges, arch type and movable bridges are not covered by these Specifications. Seismic design is usually not required for buried type (culvert) bridges.

The provisions contained in these Specifications are minimum requirements.

No detailed seismic analysis is required for any single span bridge or for any bridge in Seismic Performance Category A. For single span bridges (Article 3.11) and bridges classified as SPC A (Section 5) the connections must be designed for specified forces and must also meet minimum support length requirements.

3.2 ACCELERATION COEFFICIENT

The Acceleration Coefficient (A) to be used in the application of these provisions shall be determined from the contour maps of Figures 3.2A and 3.2. For Acceleration Coefficients for Virginia, see Figure 3.2C (Point Page 398.1). (Note: An en-

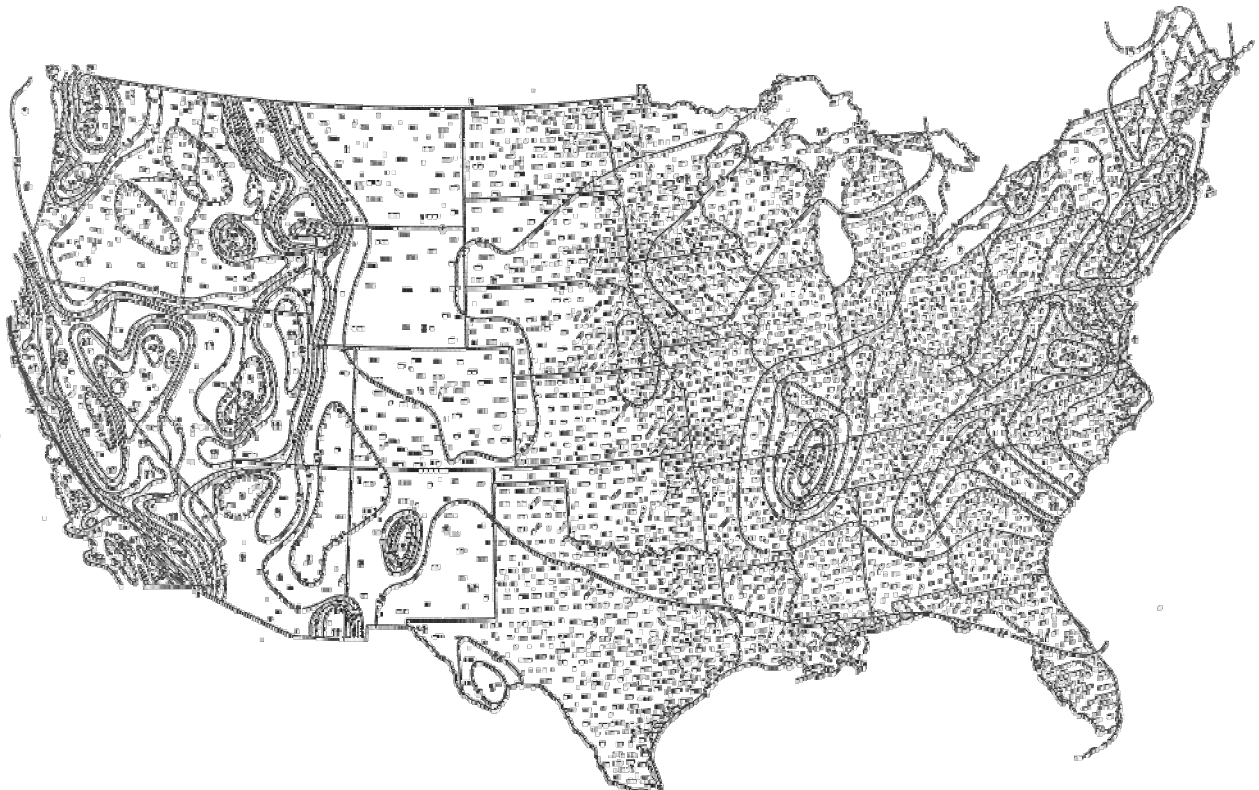
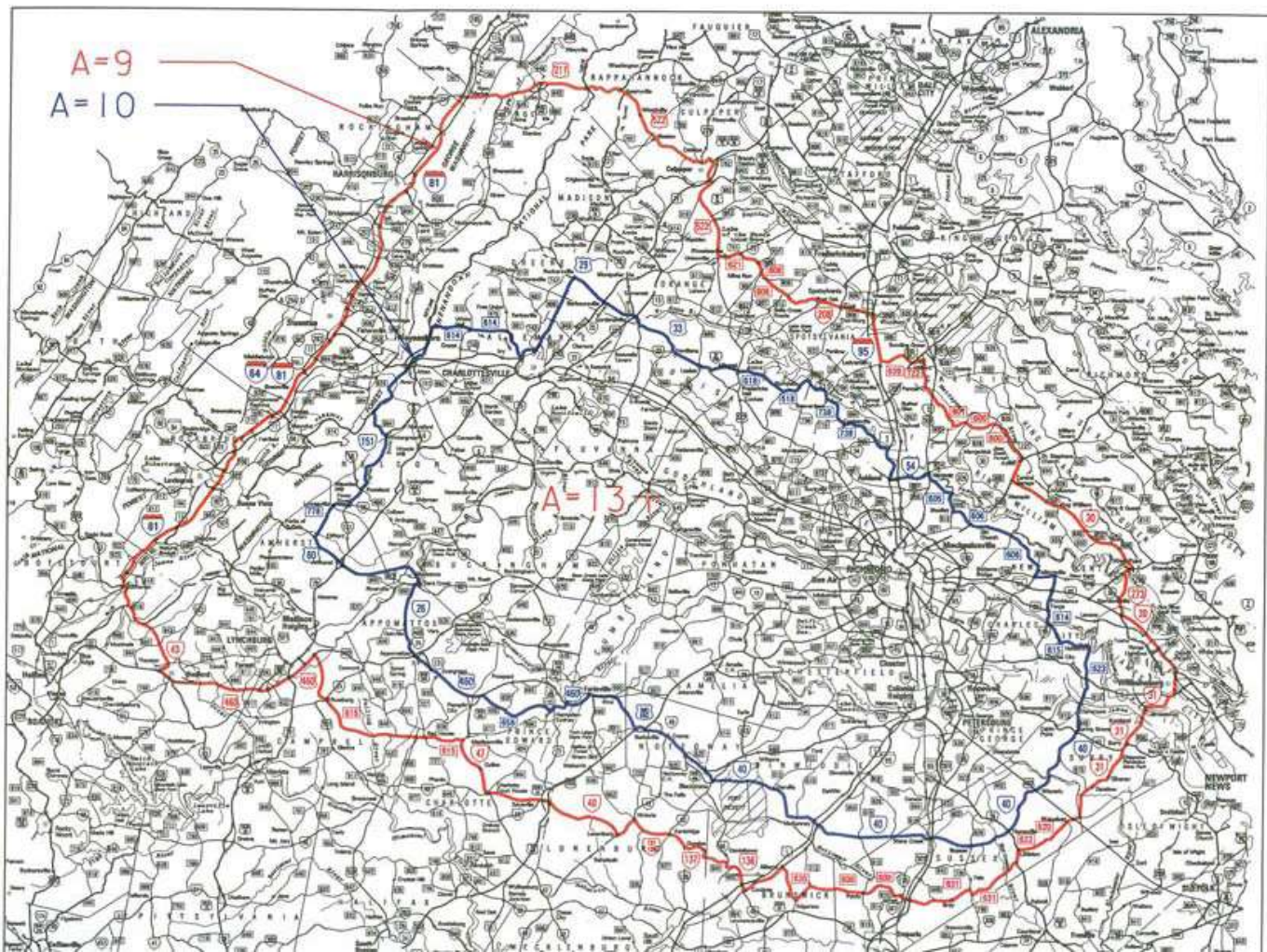


FIGURE 3.2A Acceleration Coefficient – Continental United States
(An enlarged version of this map, including counties, is given at the end of Division – I-A.)



larged version of Figure 3.2A is given at the end of Division I-A.) Values given in Figures 3.2A, 3.2B and 3.2C are expressed in percent. Numerical values for the coefficient A are obtained by dividing contour values by 100.0. Local maxima and minima are given inside the highest (and lowest) contour for a particular region. Linear interpolation shall be used for sites located between contour lines and between a contour line and the local maximum (or minimum). The seismic loads represented by the acceleration coefficients in Figures 3.2A, 3.2B and 3.2C have a 10-percent probability of exceedance in 50 years (which is approximately equivalent to a 15-percent probability of exceedance in 75 years). This corresponds to a return period of approximately 475 years. Special studies to determine site- and structure-specific acceleration co-efficients shall be performed by a qualified professional if any one of the following conditions exist:

- (a) The site is located close to an active fault.
- (b) Long duration earthquakes are expected in the region.
- (c) The importance of the bridge is such that a longer exposure period (and therefore return period) should be considered.

The effect of soil conditions at the site are considered in Article 3.5.

3.3 IMPORTANCE CLASSIFICATION

An Importance Classification (IC) shall be assigned for all bridges with an Acceleration Coefficient greater than 0.29 for the purpose of determining the Seismic Performance Category (SPC) in Article 3.4 as follows:

- 1. Essential bridges – IC = I
- 2. Other bridges – IC = II

Bridges shall be classified on the basis of Social/Survival and Security/Defense requirements, guidelines for which are given in the Commentary.

3.4 SEISMIC PERFORMANCE CATEGORIES

Each bridge shall be assigned to one of four Seismic Performance Categories (SPC), A through D, based on the Acceleration Coefficient (A) and the Importance Classification (IC), as shown in Table 3.4 Minimum analysis and design requirements are governed by the SPC.

3.5 SITE EFFECTS

The effects of site conditions on bridge response shall be determined from a Site Coefficient (S) based on soil profile types defined as follows:

TABLE 3.4 Seismic Performance Category (SPC)

Acceleration Coefficient	Importance Classification (IC)	
	I	II
$A \leq 0.09$	A	A
$0.09 < A \leq 0.19$	B	B
$0.19 < A \leq 0.29$	C	C
$0.29 < A$	D	D

SOIL PROFILE TYPE I is a profile with either—

- 1. Rock of any characteristic, either shale-like or crystalline in nature (such material may be characterized by a shear wave velocity greater than 2,500 ft/s (760 m/s), or by other appropriate means of classification); or
- 2. Stiff soil conditions where the soil depth is less than 200 ft (60 m) and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

SOIL PROFILE TYPE II is a profile with stiff clay or deep cohesionless conditions where the soil depth exceeds 200 ft (60m) and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

SOIL PROFILE TYPE III is a profile with soft to medium-stiff clays and sands, characterized by 30 ft. (9m) or more of soft to medium-stiff clays with or without intervening layers of sand or other cohesionless soils.

SOIL PROFILE TYPE IV is a profile with soft clays or silts greater than 40 ft (12 m) in depth. These materials may be characterized by a shear wave velocity less than 500 ft/s (150) m/s) and might include loose natural deposits or synthetic, nonengineered fill.

In locations where the soil properties are not known in sufficient detail to determine the soil profile type with confidence, the Engineer’s judgement shall be used to select a site coefficient from Table 3.5.1 that conservatively represents the amplification effects of the site. The soil profile coefficients apply to all foundation types including pile supported and spread footings.

A site coefficient need not be explicitly identified if a site-specific seismic response coefficient is developed by a qualified professional (Article 3.6).

3.5.1 Site Coefficient

The Site Coefficient (S) approximates the effects of the site conditions on the elastic response coefficient or spectrum of Article 3.6 and is given in Table 3.5.1.

Section 4 ANALYSIS REQUIREMENTS

4.1 GENERAL

The requirements of this section shall control the selection and method of seismic analysis of bridges. Four analysis procedures are presented.

- Procedure 1. Uniform Load Method
- Procedure 2. Single-Mode Spectral Method
- Procedure 3. Multimode Spectral Method
- Procedure 4. Time History Method

In each method, all fixed column, pier, or abutment supports are assumed to have the same ground motion at the same instant in time. At movable supports, displacements determined from the analysis prescribed in this chapter, which exceed the minimum seat width requirements as specified in Article 6.3 or 7.3, shall be used in design without reduction by the Response Modification Factor (Article 3.7).

Procedure 3 using SEISAB, STAAD or a similar finite element analysis program with seismic design capabilities is the preferred method of analysis for all bridges classified as SPC B.

4.2 SELECTION OF ANALYSIS METHOD

Minimum requirements for the selection of an analysis method for a particular bridge type are given in Table 4.2. Applicability is determined by the “regularity” of a bridge which is a function of the number of spans and the distribution of weight and stiffness. Regular bridges have less than seven spans, no abrupt or unusual changes in weight, stiffness, or geometry and no large changes in these parameters from span-to-span or support-to-support (abutments excluded). The are defined in Table 4.2B. Any bridge not satisfying the requirements of Table 4.2B

Seismic Performance Category	Regular Bridges With 2 Through 6 Spans	Not Regular Bridges with 2 or More Spans
A	Not required	Not required
B, C, D	Use Procedure 1 or 2	Use Procedure 3

is considered to be “not regular.” A more rigorous, generally accepted analysis procedure may be used in lieu of the recommended minimum such as the Time History Method (Procedure 4).

4.2.1 Special Requirements for Single-Span Bridges and Bridges in SPCA

Notwithstanding the above requirements, detailed seismic analysis is not required for a single-span bridge or for bridges classified as SPC A.

4.2.2 Special Requirements for Curved Bridges

A curved bridge may be analyzed as if it were straight provided all of the following requirements are satisfied:

- (a) the bridge is regular as defined in Table 4.2B except that for a two-span bridge the maximum span length ratio from span-to-span must not exceed 2;
- (b) the subtended angle in plan is not greater than 30°; and
- (c) the span lengths of the equivalent straight bridge are equal to the arc lengths of the curved bridge.

If these requirements are not satisfied, then curved bridges must be analyzed using the actual curved geometry.

TABLE 4.2B Regular Bridge Requirements

Parameter	Value				
Number of Spans	2	3	4	5	6
Maximum subtended Angle (curved bridge)	90°	90°	90°	90°	90°
Maximum span length ratio from span-to-span	3	2	2	1.5	1.5
Maximum bent/pier stiffness ratio from span-to-span (excluding abutments)	—	4	4	3	2

Note: All ratios expressed in terms of the smaller value.

Division II CONSTRUCTION

NOTE: Division II: Construction is replaced by the VDOT *Road and Bridge Specifications*, 2002, including Special Provisions and Copied Notes or The VDOT *Metric Road and Bridge Specifications*, January 1997, including the Special Provisions and Copied Notes.

LOADING – HS 20-44 (MS18)

TABLE OF MAXIMUM MOMENTS, SHEARS, AND REACTIONS— SIMPLE SPANS, ONE LANE

Spans in feet; moments in thousands of foot-pounds; shears and reactions in thousands of pounds.
These values are subject to specifications reduction for loading of multiple lanes.
Impact not included.

Span	Moment		End shear and end Reaction (a)		Span	Moment		End shear And end Reaction (a)
1		8.0(b,d)		32.0(b,d)	42	485.3(b,d)	56.0(b,d)	
2		16.0(b,d)		32.0(b,d)	44	520.9(b,d)	56.7(b,d)	
3		24.0(b,d)		32.0(b,d)	46	556.6(b,d)	57.3(b,d)	
4		32.0(b,d)		32.0(b,d)	48	592.1(b,d)	58.0(b,d)	
5		40.0(b,d)		32.0(b,d)	50	627.9(b,d)	58.5(b,d)	
6		48.0(b,d)	32.0(c)	32.0(b,d)	52	663.6(b,d)	59.1(b,d)	
7		56.0(b,d)	34.3(c)	32.0(b,d)	54	699.3(b,d)	59.6(b,d)	
8		64.0(b,d)	36.0(c)	32.0(b,d)	56	735.1(b,d)	60.0(b,d)	
9		72.0(b,d)	37.3(c)	32.0(b,d)	58	770.8(b,d)	60.4(b,d)	
10	80.0(c)	80.0(b,d)	38.4(c)	32.0(b,d)	60	806.5(b,d)	60.8(b,d)	
11	88.4(c)	88.0(b,d)	39.3(c)	32.0(b,d)	62	842.4(b,d)	61.2(b,d)	
12	100.0(c)	96.0(b,d)	40.0(c)	32.0(b,d)	64	878.1(b,d)	61.5(b,d)	
13	111.7(c)	104.0(b,d)	40.6(c)	32.0(b,d)	66	914.0(b,d)	61.9(b,d)	
14	123.4(c)	112.0(b,d)	41.1(c)	32.0(b,d)	68	949.7(b,d)	62.1(b,d)	
15	135.2(c)	120.0(b,d)	41.6(c)	34.1(b,d)	70	985.6(b,d)	62.4(b,d)	
16	147.0(c)	128.0(b,d)	42.0(c)	36.0(b,d)	75	1075.1(b,d)	63.1(b,d)	
17	158.8(c)	136.0(b,d)	42.4(c)	37.7(b,d)	80	1164.9(b,d)	63.6(b,d)	
18	170.7(c)	144.0(b,d)	42.7(c)	39.1(b,d)	85	1254.7(b,d)	64.1(b,d)	
19	182.5(c)	152.0(b,d)	42.9(c)	40.4(b,d)	90	1344.4(b,d)	64.5(b,d)	
20	194.4(c)	160.0(b,d)	43.2(c)	41.6(b,d)	95	1434.1(b,d)	64.9(b,d)	
21	206.3(c)	168.0(b,d)	43.4(c)	42.7(b,d)	100	1524.0(b,d)	65.3(b,d)	
22	218.2(c)	176.0(b,d)	43.6(c)	43.6(b,d)	110	1703.6(b,d)	65.9(b,d)	
23	230.1(c)	184.0(b,d)		44.5(b,d)	120	1883.3(b,d)	66.4(b,d)	
24	242.0(c)	192.0(b,d)		45.3(b,d)	130	2063.1(b,d)	67.6	
25	253.9(c)	207.4(b,d)		46.1(b,d)	140	2242.8(b,d)	70.8	
26	265.8(c)	222.2(b,d)		46.8(b,d)	150	2475.1	74.0	
27	277.8(c)	237.0(b,d)		47.4(b,d)	160	2768.0	77.2	
28	289.7(c)	252.0(b,d)		48.0(b,d)	170	3077.1	80.4	
29	301.7(c)	267.0(b,d)		48.8(b,d)	180	3402.1	83.6	
30	313.6(c)	282.1(b,d)		49.6(b,d)	190	3743.1	86.8	
31	325.5(c)	297.3(b,d)		50.3(b,d)	200	4100.0	90.0	
32	337.5(c)	312.5(b,d)		51.0(b,d)	220	4862.0	96.4	
33	349.5(c)	327.8(b,d)		51.6(b,d)	240	5688.0	102.8	
34	361.4(c)	343.5(b,d)		52.2(b,d)	260	6578.0	109.2	
35	373.4(c)	361.2(b,d)		52.8(b,d)	280	7582.0	115.6	
36	385.3(c)	378.9(b,d)		53.3(b,d)	300	8550.0	122.0	
37	397.3(c)	396.6(b,d)		53.8(b,d)				
38	414.3(c)	414.3(b,d)		54.3(b,d)				
39		432.1(b,d)		54.8(b,d)				
40		449.8(b,d)		55.2(b,d)				

- (a) Concentrated load is considered placed at the support. Loads are stipulated for shear.
- (b) Maximum value determined by Standard Truck Loading (one HS truck).
- (c) Alternate Military Load consists of two 24,000 pounds axles, spaced 4 feet apart. Shift for moment is approx. 1' from midspan.
- (d) Maximum moment occurs approx. 2.33 feet from midspan. Therefore, a parabolic moment curve should be shifted off center by this amount when used for determination of plate/cover plate cut-offs, bar bend-ups, etc.

cess, and hence abutments must be considered as a vital link in the overall seismic design process for bridges.

The nature of abutment movement or damage during past earthquakes has been well documented in the literature. Evans²² examined the abutments of 39 bridges within 30 miles (48.3 km) of the 1968 M7 Inangahua earthquake in New Zealand, of which 23 showed measurable movement and 15 were damaged. Movements of free standing abutments followed the general pattern of outward motion and rotation about the top after contact with and restraint by the superstructures. Fill settlements were observed to be 10 to 15% of the fill height. Damage effects on bridge abutments in the M7 Madang earthquake in New Guinea reported by Ellison²³ were similar, abutment movements as much as 20 in. (500 mm) were noted. Damage to abutments in the 1971 San Fernando earthquake is described by Fung et al.²⁴ Numerous instances of abutment displacement and associated damage have been reported in publications on the Niigata and Alaskan earthquakes. However, these failures were primarily associated with liquefaction of foundation soils.

Design features of abutments vary tremendously, and depend on the nature of the bridge site, foundation soils, bridge span length and load magnitudes. Abutment types include free-standing gravity walls, cantilever walls, tied back walls, and monolithic diaphragms. Foundation support may use spread footings, vertical piles or battered piles, while connection details to the superstructure may incorporate roller supports, elastomeric bearings or fixed bolted connections. Considering the number of potential design variables together with the complex nature of soil-abutment-superstructure interaction during earthquakes, it is clear that the seismic design of abutments necessitates many simplifying assumptions.

C6.4.3(A), C7.4.3(A), and C7.4.5 Free-Standing Abutments

For free-standing abutments such as gravity or cantilever walls, which are able to yield laterally during an earthquake (i.e., superstructure supported by bearings which are able to slide freely) the well-established Mononobe-Okabe pseudo-static approach outlined below, is widely used to compute earth pressures induced by earthquakes.

For free-standing abutments in highly seismic areas, design of abutments to provide zero displacement under peak ground accelerations may be unrealistic, and design for an acceptable small lateral displacement may be preferable. A recently developed method for computing the magnitude of relative wall displacement during a given earthquake is outlined in this subsection. Based on this simplified approach, recommendations are made for the selection of a pseudo-static seismic coefficient and the

corresponding displacement level for a given effective peak ground acceleration.

Mononobe-Okabe Analysis

The method most frequently used for the calculation of the seismic soil forces acting on a bridge abutment is a static approach developed in the 1920's by Mononobe²⁵ and Okabe.²⁶ The Mononobe-Okabe analysis is an extension of the Coulomb sliding-wedge theory taking into account horizontal and vertical inertia forces acting on the soil. The analysis is described in detail by Seed and Whitman²⁷ and Richards and Elms.²⁸ The following assumptions are made:

1. The abutment is free to yield sufficiently to enable full soil strength or active pressure conditions to be mobilized. If the abutment is rigidly fixed and unable to move, the soil forces will be much higher than those predicted by the Mononobe-Okabe analysis.
2. The backfill is cohesionless, with a friction angle of ϕ .
3. The backfill is unsaturated, so that liquefaction problems will not arise.

Equilibrium considerations of the soil wedge behind the abutment (figure CA10) then lead to a value, E_{AE} , of the active force exerted on the soil mass by the abutment (and vice versa), when the abutment is at the point of failure, E_{AE} is given by the expression:

$$E_{AE} = 1/2 \gamma H^2 (1 - k_v) K_{AE} \quad (CA-3)$$

where the seismic active pressure coefficient K_{AE} is:

$$K_{AE} = \frac{\cos^2(\Phi - \theta - \beta)}{\Psi \cos \theta \cos^2 \beta \cos(\delta + \beta + \theta)} \quad (CA-4)$$

and where:

- γ = unit weight of soil
- H = height of soil face
- Φ = angle of friction of soil
- θ = arc tan $(kh/(1 - k_v))$
- δ = angle of friction between soil and abutment
- k_h = horizontal acceleration coefficient
- k_v = vertical acceleration coefficient
- i = backfill slope angle
- β = slope of soil face.

$$\Psi = \left[1 + \frac{\sin(\Phi + \delta) \sin(\Phi - \theta - i)}{\cos(\delta + \beta + \theta) \cos(i - \beta)} \right]^2$$

The equivalent expression for passive force if the abutment is being pushed into the backfill is: