For interior beams, the effective flange width may be taken as the least of:

- One-quarter of the effective span length;
- 12.0 times the average depth of the slab, plus the greater of web thickness or one-half the width of the top flange of the girder; or
- The average spacing of adjacent beams.

For exterior beams, the effective flange width may be taken as one-half the effective width of the adjacent interior beam, plus the least of:

- One-eighth of the effective span length;
- 6.0 times the average depth of the slab, plus the greater of one-half the web thickness or one-quarter of the width of the top flange of the basic girder; or
- The width of the overhang.

In calculating the effective flange width for closed steel and precast concrete boxes, the distance between the outside of webs at their tops will be used in lieu of the web thickness, and the spacing will be taken as the spacing between the centerlines of boxes.

For open boxes, the effective flange width of each web should be determined as though each web was an individual supporting element.

For filled grid, partially filled grid, and for unfilled grid composite with reinforced concrete slab, the "slab depth" used should be the full depth of grid and concrete slab, minus a sacrificial depth for grinding, grooving and wear (typically 0.5 in.).

Where a structurally continuous concrete barrier is present and is included in the models used for analysis as permitted in Article 4.5.1, the width of overhang for the purpose of this Article may be extended by:

$$\Delta w = \frac{A_b}{2t_s} \qquad \text{(C4.6.2.6.1-1)}$$

where:

A_b = cross-sectional area of the barrier (in.^{2})

t_s = depth of deck slab (in.)

4.6.2.6.2 Segmental Concrete Box Beams and Single-Cell, Cast-in-Place Box Beams

The effective flange width may be assumed equal to the physical flange width if:

- $b \leq 0.1\, l_i$
- $b \leq 0.3\, d_o$

Otherwise, the effective width of outstanding flanges may be taken as specified in Figures 1 through 4, where:

d_o = depth of superstructure (in.)

b = physical flange width on each side of the web, e.g., b_1, b_2, and b_3, as shown in Figure 3 (in.)

b_e = effective flange width corresponding to the particular position of the section of interest in the span as specified in Figure 1 (in.)

b_m = effective flange width for interior portions of a span as determined from Figure 2; a special case of b_e (in.)

b_s = effective flange width at interior support or for cantilever arm as determined from Figure 2; a special case of b_e (in.)

C4.6.2.6.2

One possible alternative to the procedure specified in this Article is contained in Clause 3-10.2 of the 1991 Ontario Highway Bridge Design Code, which provides an equation for determining the effective flange width for use in calculating flexural resistances and stresses.

Superposition of local two-way slab flexural stresses due to wheel loads and the primary longitudinal flexural stresses is not normally required.

The effective flange widths b_m and b_s are determined as the product of the coefficient in Figure 2 and the physical distance b, as indicated in Figure 3.

a = portion of span subject to a transition in effective flange width taken as the lesser of the physical flange width on each side of the web shown in Figure 3 or one quarter of the span length (in.)

l_i = a notional span length specified in Figure 1 for the purpose of determining effective flange widths using Figure 2

The following interpretations apply:

- In any event, the effective flange width shall not be taken as greater than the physical width.
- The effects of unsymmetrical loading on the effective flange width may be disregarded.
- The value of b_s shall be determined using the greater of the effective span lengths adjacent to the support.
- If b_m is less than b_s in a span, the pattern of the effective width within the span may be determined by the connecting line of the effective widths b_s at adjoining support points.

For the superposition of local and global force effects, the distribution of stresses due to the global force effects may be assumed to have a straight line pattern in accordance with Figure 3c. The linear stress distribution should be determined from the constant stress distribution using the conditions that the flange force remains unchanged and that the maximum width of the linear stress distribution on each side of a web is 2.0 times the effective flange width.

The section properties for normal forces may be based on the pattern according to Figure 4 or determined by more rigorous analysis.

If the linear stress distributions intersect a free edge or each other before reaching the maximum width, the linear stress distribution is a trapezoid; otherwise, it is a triangle. This is shown in Figure 3c.

Figure 4 is intended only for calculation of resistance due to anchorage of post-tensioning tendons and other concentrated forces and may be disregarded in the general analysis to determine force effects.

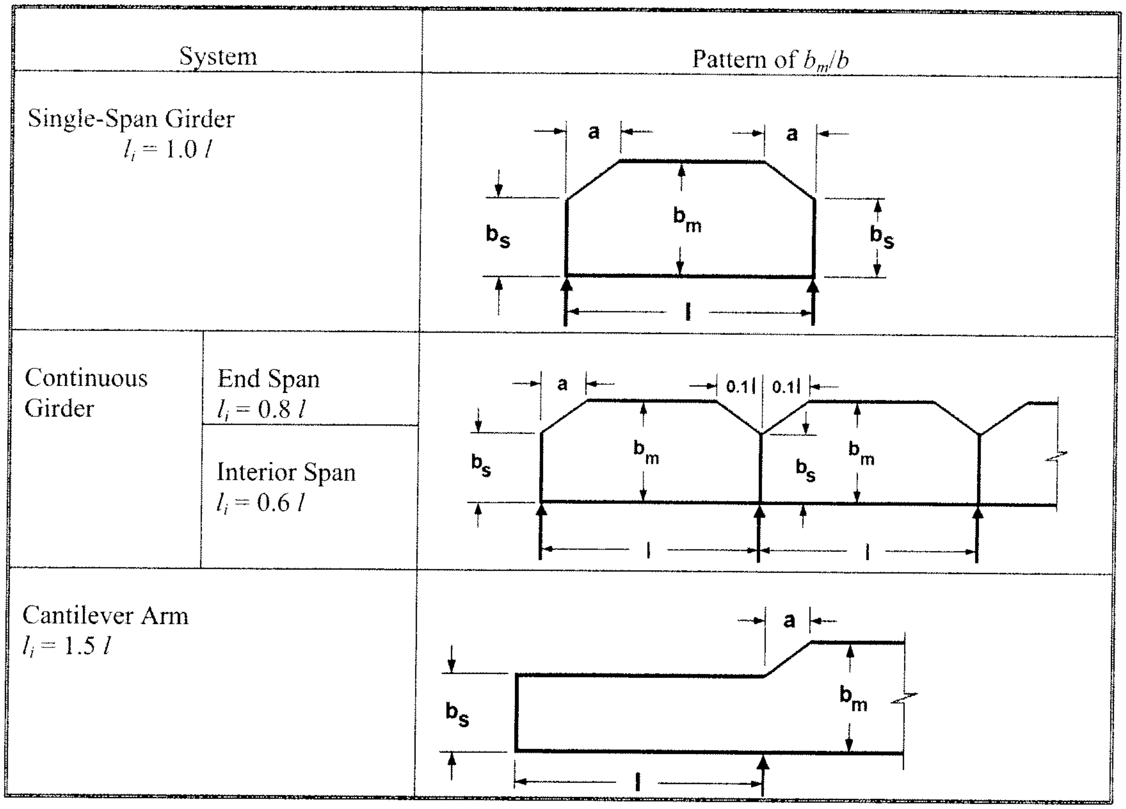


Figure 4.6.2.6.2-1 Pattern of Effective Flange Width, b_e, b_m and b_s.

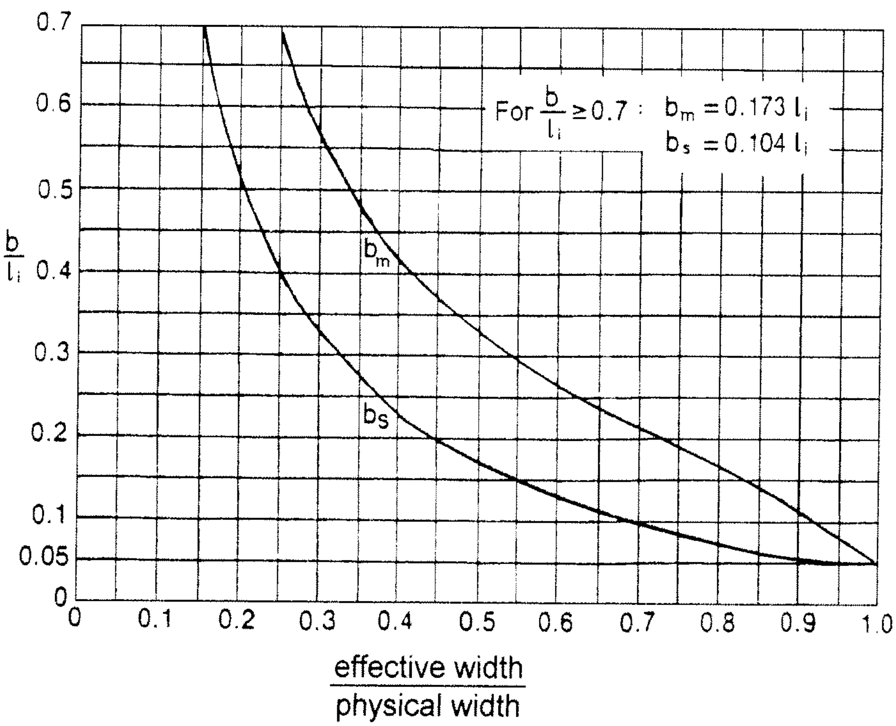


Figure 4.6.2.6.2-2 Values of the Effective Flange Width Coefficients for b_m and b_s for the Given Values of b/l_i.

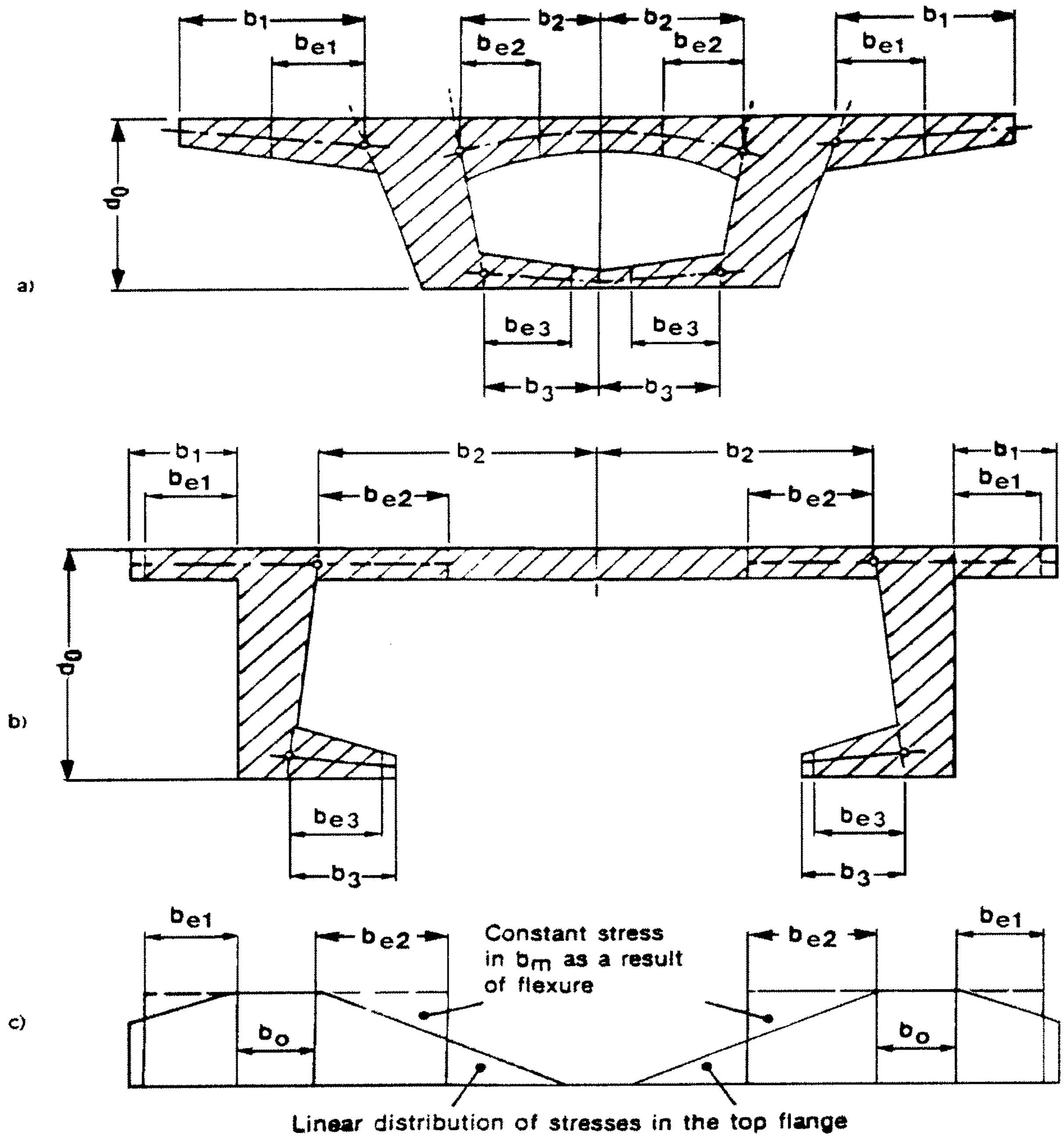


Figure 4.6.2.6.2-3 Cross-Sections and Corresponding Effective Flange Widths, b_e, for Flexure and Shear.

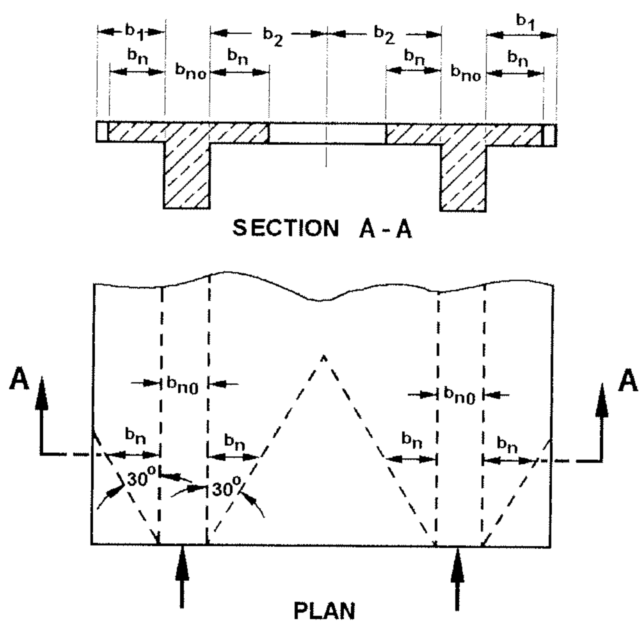


Figure 4.6.2.6.2-4 Effective Flange Widths, b_n, for Normal Forces.

4.6.2.6.3 Cast-in-Place Multicell Superstructures

The effective width for cast-in-place multiweb cellular superstructures may be taken to be as specified in Article 4.6.2.6.1, with each web taken to be a beam, or it may be taken to be the full width of the deck slab. In the latter case, the effects of shear lag in the end zones shall be investigated.

4.6.2.6.4 Orthotropic Steel Decks

The effective width of the deck plate acting as the top flange of one longitudinal stiffener, or one rib, shall be as specified in Table 1.

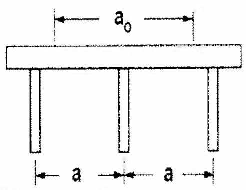
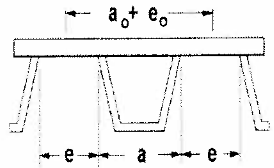
C4.6.2.6.4

The assumption of effective width equal to actual rib spacing is permissible for calculations of relative rigidity ratio by the Pelikan-Esslinger method and for flexural effects of uniformly distributed load. See discussion in Wolchuk (*1963*).

The effective width of the deck plate for flexural effects due to wheel loads is based on unequal loads on individual ribs. The specified value is an average based on more exact calculations.

Note that variation of the effective width of the deck plate does not significantly affect the rib rigidity or section modulus of the rib bottom.

Table 4.6.2.6.4-1 Effective Width of Deck Plate Acting with a Rib.

Calculation of		
Rib section properties for calculation of deck rigidity and flexural effects due to dead loads	$a_0 = a$	$a_0 + e_0 = a + e$
Rib section properties for calculation of flexural effects due to wheel loads	$a_0 = 1.1a$	$a_0 + e_0 = 1.3(a + e)$

The effective width of the deck, including the deck plate and the ribs, acting as the top flange of a longitudinal superstructure component or a transverse beam may be determined by an accepted method of analysis or may be taken as specified in Figure 1.

The effective span, shown as L_1 and L_2 in Figure 1, shall be taken as the actual span for simple spans and the distance between points of dead load inflection for continuous spans.

The development of this Figure is explained in Moffatt and Dowling (*1975* and *1976*); the particular adaptation is from Wolchuk (*1990*).

Figure 1 was originally developed to determine the effective width of deck to be considered active with each web of a box girder but is believed to be adequate for use with other types of beams.

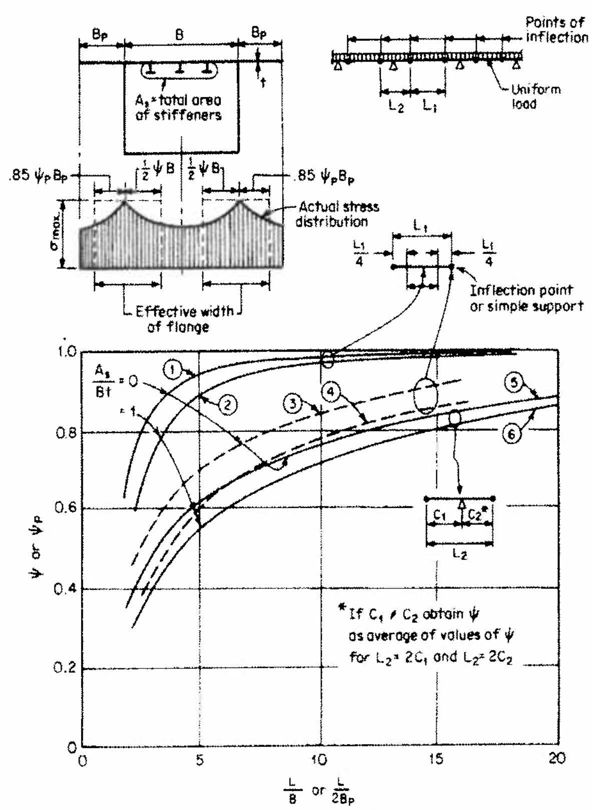


Figure 4.6.2.6.4-1 Effective Width of Deck.

The following notation applies when using Figure 1 to determine the effective width of the deck plate acting with a transverse beam:

B = spacing as shown in Figure 1 (in.)

L_1, L_2 = distances between points of inflection as shown in Figure 1 (in.)

A_s = total area of stiffeners (in.^{2})

t = thickness of flange plate (in.)

For cantilever portions of transverse beams, L shall be taken as 2.0 times the length of the cantilever.

4.6.2.7 Lateral Wind Load Distribution in Multibeam Bridges

4.6.2.7.1 I-Sections

In bridges with composite decks, noncomposite decks with concrete haunches, and other decks that can provide horizontal diaphragm action, wind load on the upper half of the outside beam, the deck, vehicles, barriers, and appurtenances shall be assumed to be directly transmitted to the deck, acting as a lateral diaphragm carrying this load to supports. Wind load on the lower half of the outside beam shall be assumed to be applied laterally to the lower flange.

For bridges with decks that cannot provide horizontal diaphragm action, the lever rule shall apply for distribution of the wind load to the top and bottom flanges.

Bottom and top flanges subjected to lateral wind load shall be assumed to carry that load to adjacent brace points by flexural action. Such brace points occur at wind bracing nodes or at cross-frames and diaphragm locations.

The lateral forces applied at brace points by the flanges shall be transmitted to the supports by one of the following load paths:

- Truss action of horizontal wind bracing in the plane of the flange;
- Frame action of the cross-frames or diaphragms transmitting the forces into the deck or the wind bracing in the plane of the other flange, and then by diaphragm action of the deck, or truss action of the wind bracing, to the supports;
- Lateral bending of the flange subjected to the lateral forces and all other flanges in the same plane, transmitting the forces to the ends of the span, for example, where the deck cannot provide horizontal diaphragm action, and there is no wind bracing in the plane of either flange.

C4.6.2.7.1

Precast concrete plank decks and timber decks are not solid diaphragms and should not be assumed to provide horizontal diaphragm action unless evidence is available to show otherwise.

Unless a more refined analysis is made, the wind force, wind moment, horizontal force to be transmitted by diaphragms and cross-frames, and horizontal force to be transmitted by lateral bracing may be calculated as indicated below. This procedure is presented for beam bridges but may be adapted for other types of bridges.

The wind force, W, may be applied to the flanges of exterior members. For composite members and noncomposite members with cast-in-place concrete or orthotropic steel decks, W need not be applied to the top flange.

$$W = \frac{\eta_i \gamma P_D d}{2} \qquad \text{(C4.6.2.7.1-1)}$$

where:

W = factored wind force per unit length applied to the flange (kip/ft.)

P_D = design horizontal wind pressure specified in Article 3.8.1 (ksf)

d = depth of the member (ft.)

γ = load factor specified in Table 3.4.1-1 for the particular group loading combination

η_i = load modifier relating to ductility, redundancy, and operational importance as specified in Article 1.3.2.1

For the first two load paths, the maximum wind moment on the loaded flange may be determined as:

$$M_w = \frac{WL_b^2}{10} \tag{C4.6.2.7.1-2}$$

where:

M_w = maximum lateral moment in the flange due to the factored wind loading (kip-ft.)

W = factored wind force per unit length applied to the flange (kip/ft.)

L_b = spacing of brace points (ft.)

For the third load path, the maximum wind moment on the loaded flange may be computed as:

$$M_w = \frac{WL_b^2}{10} + \frac{WL^2}{8N_b} \tag{C4.6.2.7.1-3}$$

where:

M_w = total lateral moment in the flange due to the factored wind loading (kip-ft.)

W = factored wind force per unit length applied to the flange (kip/ft.)

L_b = spacing of cross-frames or diaphragms (ft.)

N_b = number of longitudinal members

L = span length (ft.)

Eq. C3 is based on the assumption that cross-frames and diaphragms act as struts in distributing the wind force on the exterior flange to adjacent flanges. If there are no cross-frames or diaphragms, the first term should be taken as 0.0, and N_b should be taken as 1.0.

The horizontal wind force applied to each brace point may be calculated as:

$$P_w = WL_b \tag{C4.6.2.7.1-4}$$

where:

P_w = lateral wind force applied to the brace point (kips)

W = wind force per unit length from Eq. C1 (kip/ft.)

L_b = spacing of diaphragms or cross-frames (ft.)

Lateral bracing systems required to support both flanges due to transfer of wind loading through diaphragms or cross-frames shall be designed for a horizontal force of $2P_w$ at each brace point.

4.6.2.7.2 Box Sections

One quarter of the wind force on a box section shall be applied to the bottom flange of the exterior box beam. The section assumed to resist the wind force shall consist of the bottom flange and a part of the web as determined in Sections 5 and 6. The other three quarters of the wind force on a box section, plus the wind force on vehicles, barriers, and appurtenances, shall be assumed to be transmitted to the supports by diaphragm action of the deck.

Interbox lateral bracing shall be provided if the section assumed to resist the wind force is not adequate.

4.6.2.7.3 Construction

The need for temporary wind bracing during construction shall be investigated for I- and box-section bridges.

4.6.2.8 Seismic Lateral Load Distribution

4.6.2.8.1 Applicability

These provisions shall apply to diaphragms, cross-frames, and lateral bracing, which are part of the seismic lateral force resisting system in common slab-on-girder bridges in Seismic Zones 2, 3, and 4. The provisions of Article 3.10.9.2 shall apply to Seismic Zone 1.

4.6.2.8.2 Design Criteria

The Engineer shall demonstrate that a clear, straightforward load path to the substructure exists and that all components and connections are capable of resisting the imposed load effects consistent with the chosen load path.

The flow of forces in the assumed load path must be accommodated through all affected components and details including, but not limited to, flanges and webs of main beams or girders, cross-frames, connections, slab-to-girder interfaces, and all components of the bearing assembly from top flange interface through the confinement of anchor bolts or similar devices in the substructure.

The analysis and design of end diaphragms and cross-frames shall consider horizontal supports at an appropriate number of bearings. Slenderness and connection requirements of bracing members that are part of the lateral force resisting system shall comply with applicable provisions specified for main member design.

Members of diaphragms and cross-frames identified by the Designer as part of the load path carrying seismic forces from the superstructure to the bearings shall be designed and detailed to remain elastic, based on the applicable gross area criteria, under all design earthquakes, regardless of the type of bearings used. The applicable provisions for the design of main members shall apply.

C4.6.2.8.2

Diaphragms, cross-frames, lateral bracing, bearings, and substructure elements are part of a seismic load resisting system in which the lateral loads and performance of each element are affected by the strength and stiffness characteristics of the other elements. Past earthquakes have shown that when one of these elements responded in a ductile manner or allowed some movement, damage was limited. In the strategy taken herein, it is assumed that ductile plastic hinging in substructure is the primary source of energy dissipation. Alternative design strategies may be considered if approved by the Owner.

4.6.2.8.3 Load Distribution

A viable load path shall be established to transmit lateral loads to the foundation based on the stiffness characteristics of the deck, diaphragms, cross-frames, and lateral bracing. Unless a more refined analysis is made, an approximate load path shall be assumed as noted below.

- In bridges with:
  - A concrete deck that can provide horizontal diaphragm action, or
  - A horizontal bracing system in the plane of the top flange,

  the lateral loads applied to the deck shall be assumed to be transmitted directly to the bearings through end diaphragms or cross-frames. The development and analysis of the load path through the deck or through the top lateral bracing, if present, shall utilize assumed structural actions analogous to those used for the analysis of wind loadings.

- In bridges that have:
  - Decks that cannot provide horizontal diaphragm action and
  - No lateral bracing in the plane of the top flange,

  the lateral loads applied to the deck shall be distributed through the intermediate diaphragms and cross-frames to the bottom lateral bracing or the bottom flange, and then to the bearings, and through the end diaphragms and cross-frames, in proportion to their relative rigidity and the respective tributary mass of the deck.

- If a bottom lateral bracing system is not present, and the bottom flange is not adequate to carry the imposed force effects, the first procedure shall be used, and the deck shall be designed and detailed to provide the necessary horizontal diaphragm action.

C4.6.2.8.3

A continuous path is necessary for the transmission of the superstructure inertia forces to the foundation. Concrete decks have significant rigidity in their horizontal plane, and in short to medium slab-on-girder spans, their response approaches a rigid body motion. Therefore, the lateral loading of the intermediate diaphragms and cross-frames is minimal.

Bearings do not usually resist load simultaneously, and damage to only some of the bearings at one end of a span is not uncommon. When this occurs, high load concentrations can result at the location of the other bearings, which should be taken into account in the design of the end cross-frames or diaphragms. Also, a significant change in the load distribution among end cross-frame members may occur. Although studies of cyclic load behavior of bracing systems have shown that with adequate details, bracing systems can allow for ductile behavior, these design provisions require elastic behavior in end diaphragms (*Astaneh-Asl and Goel, 1984; Astaneh-Asl et al., 1985; Haroun and Sheperd, 1986; Goel and El-Tayem, 1986*).

Because the end diaphragm is required to remain elastic as part of the identified load path, stressing of intermediate cross-frames need not be considered.

4.6.2.9 Analysis of Segmental Concrete Bridges

4.6.2.9.1 General

Elastic analysis and beam theory may be used to determine design moments, shears, and deflections. The effects of creep, shrinkage, and temperature differentials shall be considered as well as the effects of shear lag. Shear lag shall be considered in accordance with the provisions of Article 4.6.2.9.3.

For spans in excess of 250 ft., results of elastic analyses should be evaluated with consideration of possible variations in the modulus of elasticity of the concrete, variations in the concrete creep and shrinkage properties, and the impact of variations in the construction schedule on these and other design parameters.

C4.6.2.9.1

Analysis of concrete segmental bridges requires consideration of variation of design parameters with time as well as a specific construction schedule and method of erection. This, in turn, requires the use of a computer program developed to trace the time-dependent response of segmentally erected, prestressed concrete bridges through construction and under service loads. Among the many programs developed for this purpose, several are in the public domain and may be purchased for a nominal amount, e.g., (*Ketchum, 1986; Shushkewich, 1986; Danon and Gamble, 1977*).

4.6.2.9.2 Strut-and-Tie Models

Strut-and-tie models may be used for analysis in areas of load or geometrical discontinuity.

C4.6.2.9.2

See references for background on transverse analysis of concrete box girder bridges.

4.6.2.9.3 Effective Flange Width

Effective flange width for service load stress calculations may be determined by the provisions of Article 4.6.2.6.2.

The section properties for normal forces may be based on Figure 4.6.2.6.2-4 or determined by more rigorous analysis.

Bending, shear, and normal forces may be evaluated by using the corresponding factored resistances.

The capacity of a cross-section at the strength limit state may be determined by considering the full compression flange width effect.

4.6.2.9.4 Transverse Analysis

The transverse design of box girder segments for flexure shall consider the segment as a rigid box frame. Flanges shall be analyzed as variable depth sections, considering the fillets between the flanges 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. Consideration shall be given to the increase in web shear and other effects on the cross-section resulting from eccentric loading or unsymmetrical structure geometry.

The provisions of Articles 4.6.2.1 and 4.6.3.2, influence surfaces such as those by Homberg (*1968*) and Pucher (*1964*), or other elastic analysis procedures may be used to evaluate live load plus impact moment effects in the top flange of the box section.

Transverse elastic and creep shortening due to prestressing and shrinkage shall be considered in the transverse analysis.

The effect of secondary moments due to prestressing shall be included in stress calculations at the service limit state and construction evaluation. At the strength limit state, the secondary force effects induced by prestressing, with a load factor of 1.0, shall be added algebraically to the force effects due to factored dead and live loads and other applicable loads.

4.6.2.9.5 Longitudinal Analysis

4.6.2.9.5a General

Longitudinal analysis of segmental concrete bridges shall consider a specific construction method and construction schedule as well as the time-related effects of concrete creep, shrinkage, and prestress losses.

The effect of secondary moments due to prestressing shall be included in stress calculations at the service limit state. At the strength limit state, the secondary force effects induced by prestressing, with a load factor of 1.0, shall be added algebraically to other applicable factored loads.

4.6.2.9.5b Erection Analysis

Analysis of the structure during any construction stage shall consider the construction load combinations, stresses, and stability considerations specified in Article 5.14.2.3.

4.6.2.9.5c Analysis of the Final Structural System

The provisions of Article 5.14.2.2.3 shall apply.

4.6.2.10 Equivalent Strip Widths for Box Culverts

4.6.2.10.1 General

This Article shall be applied to box culverts with depths of fill less than 2.0 ft.

C4.6.2.10.1

Design for depths of fill of 2.0 ft. or greater are covered in Article 3.6.1.2.6.

4.6.2.10.2 Case 1: Traffic Travels Parallel to Span

When traffic travels primarily parallel to the span, culverts shall be analyzed for a single loaded lane with the single lane multiple presence factor.

The axle load shall be distributed to the top slab for determining moment, thrust, and shear as follows:

Perpendicular to the span:

$$E = 96 + 1.44S \tag{4.6.2.10.2-1}$$

C4.6.2.10.2

Culverts are designed under the provisions of Section 12. Box culverts are normally analyzed as two-dimensional frames. Equivalent strip widths are used to simplify the analysis of the three-dimensional response to live loads. Eqs. 1 and 2 are based on research (*McGrath et al., 2004*) that investigated the forces in box culverts with spans up to 24.0 ft.

The distribution widths are based on distribution of shear forces. Distribution widths for positive and negative moments are wider; however, using the narrower width in combination with a single lane multiple presence factor provides designs adequate for multiple loaded lanes for all force effects.

Parallel to the span:

$$E_{span} = L_T + LLDF(H) \quad (4.6.2.10.2\text{-}2)$$

where:

E = equivalent distribution width perpendicular to span (in.)

S = clear span (ft.)

E_{span} = equivalent distribution length parallel to span (in.)

L_T = length of tire contact area parallel to span, as specified in Article 3.6.1.2.5 (in.)

$LLDF$ = factor for distribution of live load with depth of fill, 1.15 or 1.00, as specified in Article 3.6.1.2.6

H = depth of fill from top of culvert to top of pavement (in.)

Although past practice has been to ignore the distribution of live load with depth of fill, consideration of this effect, as presented in Eq. 2, produces a more accurate model of the changes in design forces with increasing depth of fill. The increased load length parallel to the span, as allowed by Eq. 2, may be conservatively neglected in design.

4.6.2.10.3 Case 2: Traffic Travels Perpendicular to Span

When traffic travels perpendicular to the span, live load shall be distributed to the top slab using the equations specified in Article 4.6.2.1 for concrete decks with primary strips perpendicular to the direction of traffic.

C4.6.2.10.3

Culverts with traffic traveling perpendicular to the span can have two or more trucks on the same design strip at the same time. This must be considered, with the appropriate multiple presence factor, in analysis of the culvert structural response.

4.6.2.10.4 Precast Box Culverts

For precast box culverts with top slabs having span-to-thickness ratios (*s/t*) of 18 or less and segment lengths greater than or equal to 4 ft in length, shear transfer across the joint need not be provided.

For precast box culverts not satisfying the requirements noted above, the design shall incorporate one of the following:

- **Provide the culvert with a means of shear transfer between the adjacent sections. Shear transfer may be provided by pavement, soil fill, or a physical connection between adjacent sections.**

- **Design the section ends as edge beams in accordance with the provisions of Article 4.6.2.1.4b using the distribution width computed from Eq. 4.6.2.10.2-1. The distribution width shall not exceed the length between two adjacent joints.**

C4.6.2.10.4

Precast box culverts manufactured in accordance with AASHTO M 273 are often installed with joints that do not provide a means of direct shear transfer across the joints of adjacent sections under service load conditions. This practice is based on research (*James, 1984*; *Frederick, et al., 1988*) which indicated significant shear transfer may not be necessary under service loading. The response of the sections tested was typified by small deflections and strains indicating that cracking did not occur under service wheel loads with no earth cover and that the demand on the section was lower than predicted by the design, which was based conservatively on a cracked section. While there are no known service issues with installation of standard box sections without means of shear transfer across joints, analysis (*McGrath et al., 2004*) shows that stresses are substantially higher when a box culvert is subjected to a live load at a free edge than when loaded away from a free edge.

However, research performed on precast box culverts that were loaded at the edge of the section (*Abolmaali and Garg, 2007*) has shown that no means of load transfer across the joint is required when the live load is distributed per Articles 4.6.2.10.2 and 4.6.2.10.3 and the top slab of the box culvert is designed in accordance with Article 5.8.3. The tested boxes were shown to have significantly more shear strength than predicted by Article 5.8.3.

For box culverts outside of the normal ASTM/AASHTO dimensional requirements, some fill or pavement **will** likely provide sufficient shear transfer to distribute live load to adjacent box sections without shear keys to avoid higher stresses due to edge loading. Otherwise, for **box culverts outside of ASTM/AASHTO dimensional requirements** with zero depth of cover, and no pavement, soil, or other means of shear transfer such as shear keys, designers should design the culvert section for the specified reduced distribution widths **lacking a more rigorous design method**.

4.6.3 Refined Methods of Analysis

4.6.3.1 General

Refined methods, listed in Article 4.4, may be used for the analysis of bridges. In such analyses, consideration shall be given to aspect ratios of elements, positioning and number of nodes, and other features of topology that may affect the accuracy of the analytical solution.

A structurally continuous railing, barrier, or median, acting compositely with the supporting components, may be considered to be structurally active at service and fatigue limit states.

When a refined method of analysis is used, a table of live load distribution coefficients for extreme force effects in each span shall be provided in the contract documents to aid in permit issuance and rating of the bridge.

C4.6.3.1

The number of possible locations for positioning the design vehicular live load will be large when determining the extreme force effect in an element using a refined method of analysis. The following are variable:

- The location of the design lanes when the available deck width contains a fraction of a design lane width,
- Which of the design lanes are actually used,
- The longitudinal location of the design vehicular live load in each lane,
- The longitudinal axle spacing of the design vehicular live load,
- The transverse location of the design vehicular live load in each lane.

This provision reflects the experimentally observed response of bridges. This source of stiffness has traditionally been neglected but exists and may be included, provided that full composite behavior is assured.

These live load distribution coefficients should be provided for each combination of component and lane.

4.6.3.2 Decks

4.6.3.2.1 General

Unless otherwise specified, flexural and torsional deformation of the deck shall be considered in the analysis but vertical shear deformation may be neglected.

Locations of flexural discontinuity through which shear may be transmitted should be modeled as hinges.

In the analysis of decks that may crack and/or separate along element boundaries when loaded, Poisson's ratio may be neglected. The wheel loads shall be modeled as patch loads distributed over an area, as specified in Article 3.6.1.2.5, taken at the contact surface. This area may be extended by the thickness of the wearing surface, integral or nonintegral, on all four sides. When such extension is utilized, the thickness of the wearing surface shall be reduced for any possible wear at the time of interest. Other extended patch areas may be utilized with the permission of the Owner provided that such extended area is consistent with the assumptions in, and application of, a particular refined method of analysis.

C4.6.3.2.1

In many solid decks, the wheel load-carrying contribution of torsion is comparable to that of flexure. Large torsional moments exist in the end zones of skewed girder bridges due to differential deflection. In most deck types, shear stresses are rather low, and their contribution to vertical deflection is not significant. In-plane shear deformations, which gave rise to the concept of effective width for composite bridge decks, should not be neglected.

4.6.3.2.2 Isotropic Plate Model

For the purpose of this section, bridge decks that are solid, have uniform or close to uniform depth, and whose stiffness is close to equal in every in-plane direction shall be considered isotropic.

C4.6.3.2.2

Analysis is rather insensitive to small deviations in constant depth, such as those due to superelevation, crown, and haunches. In slightly cracked concrete slabs, even a large difference in the reinforcement ratio will not cause significant changes in load distribution.

The torsional stiffness of the deck may be estimated using Eq. C4.6.2.2.1-1 with b equal to 1.0.

4.6.3.2.3 Orthotropic Plate Model

In orthotropic plate modeling, the flexural rigidity of the elements may be uniformly distributed along the cross-section of the deck. Where the torsional stiffness of the deck is not contributed solely by a solid plate of uniform thickness, the torsional rigidity should be established by physical testing, three-dimensional analysis, or generally accepted and verified approximations.

C4.6.3.2.3

The accuracy of the orthotropic plate analysis is sharply reduced for systems consisting of a small number of elements subjected to concentrated loads.

4.6.3.3 Beam-Slab Bridges

4.6.3.3.1 General

The aspect ratio of finite elements and grid panels should not exceed 5.0. Abrupt changes in size and/or shape of finite elements and grid panels should be avoided.

Nodal loads shall be statically equivalent to the actual loads being applied.

C4.6.3.3.1

More restrictive limits for aspect ratio may be specified for the software used.

In the absence of other information, the following guidelines may be used at the discretion of the Engineer:

- A minimum of five, and preferably nine, nodes per beam span should be used.

- For finite element analyses involving plate and beam elements, it is preferable to maintain the relative vertical distances between various elements. If this is not possible, longitudinal and transverse elements may be positioned at the midthickness of the plate-bending elements, provided that the eccentricities are included in the equivalent properties of those sections that are composite.

- For grid analysis or finite element and finite difference analyses of live load, the slab shall be assumed to be effective for stiffness in both positive and negative flexure. In a filled or partially filled grid system, composite section properties should be used.

- In finite element analysis, an element should have membrane capability with discretization sufficient to properly account for shear lag. The force effects so computed should be applied to the appropriate composite or noncomposite section for computing resistance.

- For longitudinal composite members in grid analyses, stiffness should be computed by assuming a width of the slab to be effective, but it need not be less than that specified in Article 4.6.2.6.

- For K-frame and X-frame diaphragms, equivalent beam flexure and shear stiffnesses should be computed. For bridges with widely spaced diaphragms, it may be desirable to use notional transverse beam members to model the deck. The number of such beams is to some extent discretionary. The significance of shear lag in the transverse beam-slab width as it relates to lateral load distribution can be evaluated qualitatively by varying the stiffness of the beam-slab elements within reasonable limits and observing the results. Such a sensitivity study often shows that this effect is not significant.

- Live load force effects in diaphragms should be calculated by the grid or finite element analysis. The easiest way to establish extreme force effects is by using influence surfaces analogous to those developed for the main longitudinal members.

- The St. Venant torsional inertia may be determined using the equation in Article C4.6.2.2.1. Transformation of concrete and steel to a common material should be on the basis of shear modulus, G, which can be taken as $G = 0.5E/(1+\mu)$. It is recommended that the St. Venant rigidity of composite sections utilize only one-half of the effective width of the flexural section, as described above, before transformation.

4.6.3.3.2 Curved Steel Bridges

Refined analysis methods should be used for the analysis of curved steel bridges unless the Engineer ascertains that approximate analysis methods are appropriate according to the provisions of Article 4.6.2.2.4.

C4.6.3.3.2

Refined analysis methods, identified in Article 4.4, are generally computer-based. The finite strip and finite element methods have been the most common. The finite strip method is less rigorous than the finite element method and has fallen into disuse with the advent of more powerful computers. Finite element programs may provide grid analyses using a series of beam elements connected in a plane. Refinements of the grid model may include offset elements. Frequently, the torsional warping degree of freedom is not available in beam elements. The finite element method may be applied to a three-dimensional model of the superstructure. A variety of elements may be used in this type of model. The three-dimensional model may be made capable of recognizing warping torsion by modeling each girder cross-section with a series of elements.

The stiffness of supports, including lateral restraint such as integral abutments or integral piers, should be recognized in the analysis. Since bearing restraint is offset from the neutral axis of the girders, large lateral forces at the bearings often occur and may create significant bending in the girders, which may lead to lower girder moments than would be computed if the restraints were not present. The Engineer should ascertain that any such benefit recognized in the design will be present throughout the useful life of the bridge.

Loads may be applied directly to the structural model, or applied to influence lines or influence surfaces. Only where small-deflection elastic solutions are used are influence surfaces or influence lines appropriate. The Engineer should ascertain that dead loads are applied as accurately as possible.

4.6.3.4 Cellular and Box Bridges

A refined analysis of cellular bridges may be made by any of the analytic methods specified in Article 4.4, except the yield line method, which accounts for the two dimensions seen in plan view and for the modeling of boundary conditions. Models intended to quantify torsional warping and/or transverse frame action should be fully three-dimensional.

For single box cross-sections, the superstructure may be analyzed as a spine beam for both flexural and torsional effects. A steel box should not be considered to be torsionally rigid unless internal bracing is provided to maintain the box cross-section. The transverse position of bearings shall be modeled.

4.6.3.5 Truss Bridges

A refined plane frame or space frame analysis shall include consideration for the following:

- Composite action with the deck or deck system;
- Continuity among the components;
- Force effects due to self-weight of components, change in geometry due to deformation, and axial offset at panel points; and
- In-plane and out-of-plane buckling of components including original out-of-straightness, continuity among the components and the effect axial forces present in those components.

Out-of-plane buckling of the upper chords of pony truss bridges shall be investigated. If the truss derives its lateral stability from transverse frames, of which the floorbeams are a part, the deformation of the floorbeams due to vehicular loading shall be considered.

C4.6.3.5

Load applied to deck or floorbeams instead of to truss joints will yield results that more completely quantify out-of-plane actions.

Experience has shown that dead load force effects calculated using either plane frame or space frame analysis in a truss with properly cambered primary and secondary members and detailed to minimize eccentricity at joints, will be quite close to those calculated by the conventional approximations. In many cases, a complete three-dimensional frame analysis may be the only way to accurately calculate forces in secondary members, particularly live load force effects.

4.6.3.6 Arch Bridges

The provisions of Article 4.6.3.5 shall apply where applicable.

The effect of the extension of cable hangers shall be considered in the analysis of an arch tie.

Where not controlled through proper detailing, rib shortening should be investigated.

The use of large deflection analysis of arches of longer spans should be considered in lieu of the moment magnification correction as specified in Article 4.5.3.2.2c.

When the distribution of stresses between the top and bottom chords of trussed arches is dependent on the manner of erection, the manner of erection shall be indicated in the contract documents.

C4.6.3.6

Rib shortening and arch design and construction are discussed by Nettleton (*1977*).

Any single-step correction factor cannot be expected to accurately model deflection effects over a wide range of stiffnesses.

If a hinge is provided at the crown of the rib in addition to hinges at the abutment, the arch becomes statically determinate, and stresses due to change of temperature and rib shortening are essentially eliminated.

Arches may be analyzed, designed, and constructed as hinged under dead load or portions of dead load and as fixed at some hinged locations for the remaining design loads.

In trussed arches, considerable latitude is available in design for distribution of stresses between the top and bottom chords dependent on the manner of erection. In such cases, the manner of erection should be indicated in the contract documents.

4.6.3.7 Cable-Stayed Bridges

The distribution of force effects to the components of a cable-stayed bridge may be determined by either spatial or planar structural analysis if justified by consideration of tower geometry, number of planes of stays, and the torsional stiffness of the deck superstructure.

Cable-stayed bridges shall be investigated for nonlinear effects that may result from:

- The change in cable sag at all limit states,
- Deformation of deck superstructure and towers at all limit states, and
- Material nonlinearity at the extreme event limit states.

Cable sag may be investigated using an equivalent member modeled as a chord with modified modulus of elasticity given by Eq. 1 for instantaneous stiffness and Eq. 2, applied iteratively, for changing cable loads.

$$E_{MOD} = E\left[1 + \frac{EAW^2(\cos\alpha)^5}{12H^3}\right]^{-1} \qquad (4.6.3.7\text{-}1)$$

$$E_{MOD} = E\left[1 + \frac{(H_1 + H_2)EAW^2(\cos\alpha)^5}{24H_1^2H_2^2}\right]^{-1} \qquad (4.6.3.7\text{-}2)$$

where:

E = modulus of elasticity of the cable (ksi)

W = total weight of cable (kip)

A = cross-sectional area of cable (in.^{2})

α = angle between cable and horizontal (°)

H, H_1, H_2 = horizontal component of cable force (kip)

The change in force effects due to deflection may be investigated using any method that satisfies the provisions of Article 4.5.3.2.1 and accounts for the change in orientation of the ends of cable stays.

Cable-stayed bridges shall be investigated for the loss of any one cable stay.

C4.6.3.7

Nonlinear effects on cable-stayed bridges are treated in several texts, e.g., (*Podolny and Scalzi, 1986; Troitsky, 1977*), and a report by the ASCE Committee on Cable Suspended Bridges (*ASCE, 1991*), from which the particular forms of Eqs. 1 and 2 were taken.

4.6.3.8 Suspension Bridges

Force effects in suspension bridges shall be analyzed by the large deflection theory for vertical loads. The effects of wind loads shall be analyzed, with consideration of the tension stiffening of the cables. The torsional rigidity of the deck may be neglected in assigning forces to cables, suspenders, and components of stiffening trusses.

C4.6.3.8

In the past, short suspension bridges have been analyzed by conventional small deflection theories. Correction factor methods have been used on short- to moderate-span bridges to account for the effect of deflection, which is especially significant for calculating deck system moments. Any contemporary suspension bridge would have a span such that the large deflection theory should be used. Suitable computer programs are commercially available. Therefore, there is little rationale to use anything other than the large deflection solution.

For the same economic reasons, the span would probably be long enough that the influence of the torsional rigidity of the deck, combined with the relatively small effect of live load compared to dead load, will make the simple sum-of-moments technique suitable to assign loads to the cables and suspenders and usually even to the deck system, e.g., a stiffening truss.

4.6.4 Redistribution of Negative Moments in Continuous Beam Bridges

4.6.4.1 General

The Owner may permit the redistribution of force effects in multispan, multibeam, or girder superstructures. Inelastic behavior shall be restricted to the flexure of beams or girders, and inelastic behavior due to shear and/or uncontrolled buckling shall not be permitted. Redistribution of loads shall not be considered in the transverse direction.

The reduction of negative moments over the internal supports due to the redistribution shall be accompanied by a commensurate increase in the positive moments in the spans.

4.6.4.2 Refined Method

The negative moments over the support, as established by linear elastic analysis, may be decreased by a redistribution process considering the moment-rotation characteristics of the cross-section or by a recognized mechanism method. The moment-rotation relationship shall be established using material characteristics, as specified herein, and/or verified by physical testing.

4.6.4.3 Approximate Procedure

In lieu of the analysis described in Article 4.6.4.2, simplified redistribution procedures for concrete and steel beams, as specified in Sections 5 and 6, respectively, may be used.

4.6.5 Stability

The investigation of stability shall utilize the large deflection theory.

4.6.6 Analysis for Temperature Gradient

Where determination of force effects due to vertical temperature gradient is required, the analysis should consider axial extension, flexural deformation, and internal stresses.

Gradients shall be as specified in Article 3.12.3.

C4.6.6

The response of a structure to a temperature gradient can be divided into three effects as follows:

- AXIAL EXPANSION—This is due to the uniform component of the temperature distribution that should be considered simultaneously with the uniform temperature specified in Article 3.12.2. It may be calculated as:

$$T_{UG} = \frac{1}{A_c} \iint T_G \, dw \, dz \qquad \text{(C4.6.6-1)}$$

The corresponding uniform axial strain is:

$$\varepsilon_u = \alpha\left(T_{UG} + T_u\right) \qquad \text{(C4.6.6-2)}$$

- FLEXURAL DEFORMATION—Because plane sections remain plane, a curvature is imposed on the superstructure to accommodate the linearly variable component of the temperature gradient. The rotation per unit length corresponding to this curvature may be determined as:

$$\phi = \frac{\alpha}{I_c} \iint T_G z \, dw \, dz = \frac{1}{R} \qquad \text{(C4.6.6-3)}$$

If the structure is externally unrestrained, i.e., simply supported or cantilevered, no external force effects are developed due to this superimposed deformation.

The axial strain and curvature may be used in both flexibility and stiffness formulations. In the former, ε_u may be used in place of P/AE, and ϕ may be used in place of M/EI in traditional displacement calculations. In the latter, the fixed-end force effects for a prismatic frame element may be determined as:

$$N = EA_c \varepsilon_u \qquad \text{(C4.6.6-4)}$$

$$M = EI_c \phi \qquad \text{(C4.6.6-5)}$$

An expanded discussion with examples may be found in Ghali and Neville (*1989*).

Strains induced by other effects, such as shrinkage and creep, may be treated in a similar manner.

- INTERNAL STRESS—Using the sign convention that compression is positive, internal stresses in addition to those corresponding to the restrained axial expansion and/or rotation may be calculated as:

$$\sigma_E = E\left[\alpha T_G - \alpha T_{UG} - \phi z\right] \quad \text{(C4.6.6-6)}$$

where:

T_G = temperature gradient (Δ°F)

T_{UG} = temperature averaged across the cross-section (°F)

T_u = uniform specified temperature (°F)

A_c = cross-section area—transformed for steel beams (in.^{2})

I_c = inertia of cross-section—transformed for steel beams (in.^{4})

α = coefficient of thermal expansion (in./in./°F)

E = modulus of elasticity (ksi)

R = radius of curvature (ft.)

w = width of element in cross-section (in.)

z = vertical distance from center of gravity of cross-section (in.)

For example, the flexural deformation part of the gradient flexes a prismatic superstructure into a segment of a circle in the vertical plane. For a two-span structure with span length, L, in ft., the unrestrained beam would lift off from the central support by $\Delta = 6\ L^2/R$ in in. Forcing the beam down to eliminate Δ would develop a moment whose value at the pier would be:

$$M_c = \frac{3}{2} EI_c \phi \quad \text{(C4.6.6-7)}$$

Therefore, the moment is a function of the beam rigidity and imposed flexure. As rigidity approaches 0.0 at the strength limit state, M_c tends to disappear. This behavior also indicates the need for ductility to ensure structural integrity as rigidity decreases.

4.7 DYNAMIC ANALYSIS

4.7.1 Basic Requirements of Structural Dynamics

4.7.1.1 General

For analysis of the dynamic behavior of bridges, the stiffness, mass, and damping characteristics of the structural components shall be modeled.

The minimum number of degrees-of-freedom included in the analysis shall be based upon the number of natural frequencies to be obtained and the reliability of the assumed mode shapes. The model shall be compatible with the accuracy of the solution method. Dynamic models shall include relevant aspects of the structure and the excitation. The relevant aspects of the structure may include the:

- Distribution of mass,
- Distribution of stiffness, and
- Damping characteristics.

The relevant aspects of excitation may include the:

- Frequency of the forcing function,
- Duration of application, and
- Direction of application.

C4.7.1.1

Typically, analysis for vehicle- and wind-induced vibrations is not to be considered in bridge design. Although a vehicle crossing a bridge is not a static situation, the bridge is analyzed by statically placing the vehicle at various locations along the bridge and applying a dynamic load allowance, as specified in Article 3.6.2, to account for the dynamic responses caused by the moving vehicle. However, in flexible bridges and long slender components of bridges that may be excited by bridge movement, dynamic force effects may exceed the allowance for impact given in Article 3.6.2. In most observed bridge vibration problems, the natural structural damping has been very low. Flexible continuous bridges may be especially susceptible to vibrations. These cases may require analysis for moving live load.

If the number of degrees-of-freedom in the model exceeds the number of dynamic degrees-of-freedom used, a standard condensation procedure may be employed.

Condensation procedures may be used to reduce the number of degrees-of-freedom prior to the dynamic analysis. Accuracy of the higher modes can be compromised with condensation. Thus if higher modes are required, such procedures should be used with caution.

The number of frequencies and mode shapes necessary to complete a dynamic analysis should be estimated in advance or determined as an early step in a multistep approach. Having determined that number, the model should be developed to have a larger number of applicable degrees-of-freedom.

Sufficient degrees-of-freedom should be included to represent the mode shapes relevant to the response sought. One rule-of-thumb is that there should be twice as many degrees-of-freedom as required frequencies.

The number of degrees-of-freedom and the associated masses should be selected in a manner that approximates the actual distributive nature of mass. The number of required frequencies also depends on the frequency content of the forcing function.

4.7.1.2 Distribution of Masses

The modeling of mass shall be made with consideration of the degree of discretization in the model and the anticipated motions.

C4.7.1.2

The distribution of stiffness and mass should be modeled in a dynamic analysis. The discretization of the model should account for geometric and material variation in stiffness and mass.

The selection of the consistent or lump mass formulation is a function of the system and the response sought and is difficult to generalize. For distributive mass systems modeled with polynomial shape functions in which the mass is associated with distributive stiffness, such as a beam, a consistent mass formulation is recommended (*Paz, 1985*). In lieu of a consistent formulation, lumped masses may be associated at the translational degrees-of-freedom, a manner that approximates the distributive nature of the mass (*Clough and Penzian, 1975*).

For systems with distributive mass associated with larger stiffness, such as in-plane stiffness of a bridge deck, the mass may be properly modeled as lumped. The rotational inertia effects should be included where significant.

In seismic analysis, nonlinear effects, such as inelastic deformation and cracking, which decrease the stiffness, should be considered.

4.7.1.3 Stiffness

The bridge shall be modeled to be consistent with the degrees-of-freedom chosen to represent the natural modes and frequencies of vibration. The stiffness of the elements of the model shall be defined to be consistent with the bridge being modeled.

4.7.1.4 Damping

Equivalent viscous damping may be used to represent energy dissipation.

C4.7.1.4

Damping may be neglected in the calculation of natural frequencies and associated nodal displacements. The effects of damping should be considered where a transient response is sought.

Suitable damping values may be obtained from field measurement of induced free vibration or by forced vibration tests. In lieu of measurements, the following values may be used for the equivalent viscous damping ratio:

- Concrete construction: 2 percent
- Welded and bolted steel construction: 1 percent
- Timber: 5 percent

4.7.1.5 Natural Frequencies

For the purpose of Article 4.7.2, and unless otherwise specified by the Owner, elastic undamped natural modes and frequencies of vibration shall be used. For the purpose of Articles 4.7.4 and 4.7.5, all relevant damped modes and frequencies shall be considered.

4.7.2 Elastic Dynamic Responses

4.7.2.1 Vehicle-Induced Vibration

When an analysis for dynamic interaction between a bridge and the live load is required, the Owner shall specify and/or approve surface roughness, speed, and dynamic characteristics of the vehicles to be employed for the analysis. Impact shall be derived as a ratio of the extreme dynamic force effect to the corresponding static force effect.

In no case shall the dynamic load allowance used in design be less than 50 percent of the dynamic load allowance specified in Table 3.6.2.1-1, except that no reduction shall be allowed for deck joints.

C4.7.2.1

The limitation on the dynamic load allowance reflects the fact that deck surface roughness is a major factor in vehicle/bridge interaction and that it is difficult to estimate long-term deck deterioration effects thereof at the design stage.

The proper application of the provision for reducing the dynamic load allowance is:

$$IM_{CALC} \geq 0.5 IM_{Table\ 3\text{-}6} \qquad \text{(C4.7.2.1-1)}$$

not:

$$\left(1+\frac{IM}{100}\right)_{CALC} \geq 0.5\left(1+\frac{IM}{100}\right) \qquad \text{(C4.7.2.1-2)}$$

4.7.2.2 Wind-Induced Vibration

4.7.2.2.1 Wind Velocities

For **critical or essential** structures, which may be expected to be sensitive to wind effects, the location and magnitude of extreme pressure and suction values shall be established by simulated wind tunnel tests.

4.7.2.2.2 Dynamic Effects

Wind-sensitive structures shall be analyzed for dynamic effects, such as buffeting by turbulent or gusting winds, and unstable wind-structure interaction, such as galloping and flutter. Slender or torsionally flexible structures shall be analyzed for lateral buckling, excessive thrust, and divergence.

4.7.2.2.3 Design Considerations

Oscillatory deformations under wind that may lead to excessive stress levels, structural fatigue, and user inconvenience or discomfort shall be avoided. Bridge decks, cable stays, and hanger cables shall be protected against excessive vortex and wind-rain-induced oscillations. Where practical, the employment of dampers shall be considered to control excessive dynamic responses. Where dampers or shape modification are not practical, the structural system shall be changed to achieve such control.

C4.7.2.2.3

Additional information on design for wind may be found in AASHTO (*1985*); Scanlan (*1975*); Simiu and Scanlan (*1978*); Basu and Chi (*1981a*); Basu and Chi (*1981b*); ASCE (*1961*); and ASCE (*1991*).

4.7.3 Inelastic Dynamic Responses

4.7.3.1 General

During a major earthquake or ship collision, energy may be dissipated by one or more of the following mechanisms:

- Elastic and inelastic deformation of the object that may collide with the structure,
- Inelastic deformation of the structure and its attachments,
- Permanent displacement of the masses of the structure and its attachments, and
- Inelastic deformation of special-purpose mechanical energy dissipators.

4.7.3.2 Plastic Hinges and Yield Lines

For the purpose of analysis, energy absorbed by inelastic deformation in a structural component may be assumed to be concentrated in plastic hinges and yield lines. The location of these sections may be established by successive approximation to obtain a lower bound solution for the energy absorbed. For these sections, moment-rotation hysteresis curves may be determined by using verified analytic material models.

4.7.4 Analysis for Earthquake Loads

4.7.4.1 General

Minimum analysis requirements for seismic effects shall be as specified in Table 4.7.4.3.1-1.

For the modal methods of analysis, specified in Articles 4.7.4.3.2 and 4.7.4.3.3, the design response spectrum specified in Figure 3.10.4.1-1 and Eqs. 3.10.4.2-1, 3.10.4.2-3, and 3.10.4.2.4 shall be used.

Bridges in Seismic Zone 1 need not be analyzed for seismic loads, regardless of their **operational classification** and geometry. However, the minimum requirements, as specified in Articles 4.7.4.4 and 3.10.9, shall apply.

4.7.4.2 Single-Span Bridges

Seismic analysis is not required for single-span bridges, regardless of seismic zone.

Connections between the bridge superstructure and the abutments shall be designed for the minimum force requirements as specified in Article 3.10.9.

Minimum support length requirements shall be satisfied at each abutment as specified in Article 4.7.4.4.

C4.7.4.2

A single-span bridge is comprised of a superstructure unit supported by two abutments with no intermediate piers.

4.7.4.3 Multispan Bridges

4.7.4.3.1 Selection of Method

For multispan structures, the minimum analysis requirements shall be as specified in Table 1 in which:

* = no seismic analysis required

UL = uniform load elastic method

SM = single-mode elastic method

MM = multimode elastic method

TH = time history method

C4.7.4.3.1

The selection of the method of analysis depends on seismic zone, regularity, and **operational classification** of the bridge.

Regularity 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. A more rigorous analysis procedure may be used in lieu of the recommended minimum.

Table 4.7.4.3.1-1 Minimum Analysis Requirements for Seismic Effects.

Seismic Zone	Single-Span Bridges	Multispan Bridges					
		Other Bridges		Essential Bridges		Critical Bridges	
		regular	irregular	regular	irregular	regular	irregular
1	No seismic analysis required	*	*	*	*	*	*
2		SM/UL	SM	SM/UL	MM	MM	MM
3		SM/UL	MM	MM	MM	MM	TH
4		SM/UL	MM	MM	MM	TH	TH

Except as specified below, bridges satisfying the requirements of Table 2 may be taken as "regular" bridges. Bridges not satisfying the requirements of Table 2 shall be taken as "irregular" bridges.

Table 4.7.4.3.1-2 Regular Bridge Requirements.

Parameter	Value				
Number of Spans	2	3	4	5	6
Maximum subtended angle for a 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

Curved bridges comprised of multiple simple-spans shall be considered to be "irregular" if the subtended angle in plan is greater than 20°. Such bridges shall be analyzed by either the multimode elastic method or the time-history method.

A curved continuous-girder bridge may be analyzed as if it were straight, provided all of the following requirements are satisfied:

- The bridge is "regular" as defined in Table 2, except that for a two-span bridge the maximum span length ratio from span to span must not exceed 2;

- The subtended angle in plan is not greater than 90°; and
- 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 continuous-girder bridges must be analyzed using the actual curved geometry.

4.7.4.3.2 Single-Mode Methods of Analysis

4.7.4.3.2a General

Either of the two single-mode methods of analysis specified herein may be used where appropriate.

4.7.4.3.2b Single-Mode Spectral Method

The single-mode method of spectral analysis shall be based on the fundamental mode of vibration in either the longitudinal or transverse direction. For regular bridges, the fundamental modes of vibration in the horizontal plane coincide with the longitudinal and transverse axes of the bridge structure. This mode shape may be found by applying a uniform horizontal load to the structure and calculating the corresponding deformed shape. The natural period may be calculated by equating the maximum potential and kinetic energies associated with the fundamental mode shape. The amplitude of the displaced shape may be found from the elastic seismic response coefficient, C_{sm}, specified in Article 3.10.4.2, and the corresponding spectral displacement. This amplitude shall be used to determine force effects.

C4.7.4.3.2b

The single-mode spectral analysis method described in the following steps may be used for both transverse and longitudinal earthquake motions. Examples illustrating its application are given in AASHTO (*1983*) and ATC (*1981*).

- Calculate the static displacements $v_s(x)$ due to an assumed uniform loading p_o as shown in Figure C1:

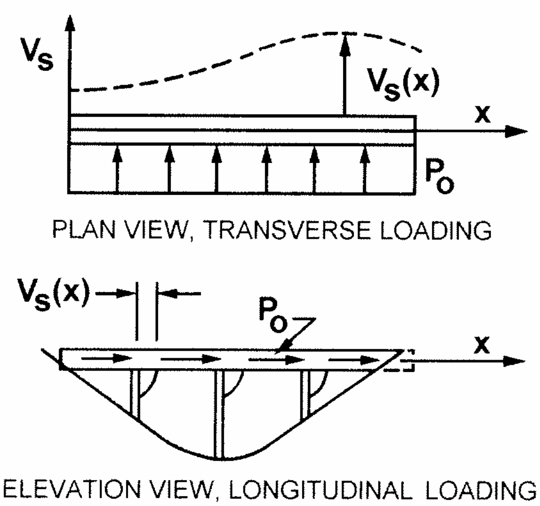


Figure C4.7.4.3.2b-1 Bridge Deck Subjected to Assumed Transverse and Longitudinal Loading.

- Calculate factors α, β, and γ as:

$$\alpha = \int v_s(x)\,dx \tag{C4.7.4.3.2b-1}$$

$$\beta = \int w(x)\,v_s(x)\,dx \tag{C4.7.4.3.2b-2}$$

$$\gamma = \int w(x) v_s^2(x)\, dx \qquad \text{(C4.7.4.3.2b-3)}$$

where:

p_o = a uniform load arbitrarily set equal to 1.0 (kip/ft.)

$v_s(x)$ = deformation corresponding to p_o (ft.)

$w(x)$ = nominal, unfactored dead load of the bridge superstructure and tributary substructure (kip/ft.)

The computed factors, α, β, and γ have units of (ft.^{2}), (kip-ft.), and (kip-ft.^{2}), respectively.

- Calculate the period of the bridge as:

$$T_m = 2\pi \sqrt{\frac{\gamma}{p_o g \alpha}} \qquad \text{(C4.7.4.3.2b-4)}$$

where:

g = acceleration of gravity (ft./sec.^{2})

- Using T_m and Eq. 3.10.6.1-1, calculate C_{sm}
- Calculate the equivalent static earthquake loading $p_e(x)$ as:

$$p_e(x) = \frac{\beta C_{sm}}{\gamma} w(x) v_s(x) \qquad \text{(C4.7.4.3.2b-5)}$$

where:

C_{sm} = the dimensionless elastic seismic response coefficient given by Eq. 3.10.6.1-1

$p_e(x)$ = the intensity of the equivalent static seismic loading applied to represent the primary mode of vibration (kip/ft.)

- Apply loading $p_e(x)$ to the structure, and determine the resulting member force effects.

4.7.4.3.2c Uniform Load Method

The uniform load method shall be based on the fundamental mode of vibration in either the longitudinal or transverse direction. The period of this mode of vibration shall be taken as that of an equivalent single mass-spring oscillator. The stiffness of this equivalent spring shall be calculated using the maximum displacement that occurs when an arbitrary uniform lateral load is applied to the bridge. The elastic seismic response coefficient, C_{sm}, specified in Article 3.10.6 shall be used to calculate the equivalent uniform seismic load from which seismic force effects are found.

C4.7.4.3.2c

The uniform load method, described in the following steps, may be used for both transverse and longitudinal earthquake motions. It is essentially an equivalent static method of analysis that uses a uniform lateral load to approximate the effect of seismic loads. The method is suitable for regular bridges that respond principally in their fundamental mode of vibration. Whereas all displacements and most member forces are calculated with good accuracy, the method is known to overestimate the transverse shears at the abutments by up to 100 percent. If such conservatism is undesirable, then the single-mode spectral analysis method specified in Article 4.7.4.3.2b is recommended.

- Calculate the static displacements $v_s(x)$ due to an assumed uniform load p_o, as shown in Figure C4.7.4.3.2b-1. The uniform loading p_o is applied over the length of the bridge; it has units of force per unit length and may be arbitrarily set equal to 1.0. The static displacement $v_s(x)$ has units of length.

- Calculate the bridge lateral stiffness, K, and total weight, W, from the following expressions:

$$K = \frac{p_o L}{v_{s,MAX}} \quad \text{(C4.7.4.3.2c-1)}$$

$$W = \int w(x)dx \quad \text{(C4.7.4.3.2c-2)}$$

where:

L = total length of the bridge (ft.)

$v_{s,MAX}$ = maximum value of $v_s(x)$ (ft.)

$w(x)$ = nominal, unfactored dead load of the bridge superstructure and tributary substructure (kip/ft.)

The weight should take into account structural elements and other relevant loads including, but not limited to, pier caps, abutments, columns, and footings. Other loads, such as live loads, may be included. Generally, the inertia effects of live loads are not included in the analysis; however, the probability of a large live load being on the bridge during an earthquake should be considered when designing bridges with high live-to-dead load ratios that are located in metropolitan areas where traffic congestion is likely to occur.

- Calculate the period of the bridge, T_m, using the expression:

$$T_m = 2\pi\sqrt{\frac{W}{gK}} \quad \text{(C4.7.4.3.2c-3)}$$

where:

g = acceleration of gravity (ft./sec.^{2})

- Calculate the equivalent static earthquake loading p_e from the expression:

$$p_e = \frac{C_{sm} W}{L} \quad \text{(C4.7.4.3.2c-4)}$$

where:

C_{sm} = the dimensionless elastic seismic response coefficient given by Eq. 3.10.6.1-1

p_e = equivalent uniform static seismic loading per unit length of bridge applied to represent the primary mode of vibration (kip/ft.)

- Calculate the displacements and member forces for use in design either by applying p_e to the structure and performing a second static analysis or by scaling the results of the first step above by the ratio p_e/p_o.

4.7.4.3.3 Multimode Spectral Method

The multimode spectral analysis method shall be used for bridges in which coupling occurs in more than one of the three coordinate directions within each mode of vibration. As a minimum, linear dynamic analysis using a three-dimensional model shall be used to represent the structure.

The number of modes included in the analysis should be at least three times the number of spans in the model. The elastic seismic response spectrum as specified in Article 3.10.6 shall be used for each mode.

The member forces and displacements may be estimated by combining the respective response quantities (moment, force, displacement, or relative displacement) from the individual modes by the Complete Quadratic Combination (CQC) method.

C4.7.4.3.3

Member forces and displacements obtained using the CQC combination method are generally adequate for most bridge systems (*Wilson et al., 1981*).

If the CQC method is not readily available, alternative methods include the square root of the sum of the squares method (SRSS), but this method is best suited for combining responses from well-separated modes. For closely spaced modes, the absolute sum of the modal responses should be used.

4.7.4.3.4 Time-History Method

Any step-by-step time-history method of analysis used for either elastic or inelastic analysis shall satisfy the requirements of Article 4.7.

The sensitivity of the numerical solution to the size of the time step used for the analysis shall be determined. A sensitivity study shall also be carried out to investigate the effects of variations in assumed material hysteretic properties.

The time histories of input acceleration used to describe the earthquake loads shall be selected in consultation with the Owner. Unless otherwise directed, five spectrum-compatible time histories shall be used when site-specific time histories are not available. The spectrum used to generate these five time histories shall be the same as that used for the modal methods, as specified in Article 3.10.6, modified for the appropriate soil profile.

C4.7.4.3.4

Rigorous methods of analysis are required for critical structures, which are defined in Article 3.10.3, and/or those that are geometrically complex or close to active earthquake faults. Time history methods of analysis are recommended for this purpose, provided care is taken with both the modeling of the structure and the selection of the input time histories of ground acceleration.

Site-specific spectrum is preferred, if available.

4.7.4.4 Minimum Displacement Requirements

Bridge seat widths at expansion bearings without restrainers, STUs, or dampers shall either accommodate the greater of the maximum displacement calculated in accordance with the provisions of Article 4.7.4.3, except for bridges in Zone 1, or a percentage of the empirical seat width, N, specified by Eq. 1. Otherwise, longitudinal restrainers complying with Article 3.10.9.5 shall be provided. Bearings restrained for longitudinal movement shall be designed in compliance with Article 3.10.9. The percentages of N, applicable to each seismic zone, shall be as specified in Table 1.

The empirical seat width shall be taken as:

$$N = (8 + 0.02L + 0.08H)(1 + 0.000125S^2) \qquad (4.7.4.4\text{-}1)$$

where:

N = minimum support length measured normal to the centerline of bearing (in.)

L = length of the bridge deck to the adjacent expansion joint, or to the end of the bridge deck; for hinges within a span, L shall be the sum of the distances to either side of the hinge; for single-span bridges, L equals the length of the bridge deck (ft.)

H = for abutments, average height of columns supporting the bridge deck to the next expansion joint (ft.)

for columns and/or piers, column, or pier height (ft.)

for hinges within a span, average height of the adjacent two columns or piers (ft.)

0.0 for single-span bridges (ft.)

S = skew of support measured from line normal to span (°)

Table 4.7.4.4-1 Percentage N by Zone and Acceleration Coefficient.

Zone	Acceleration Coefficient	Soil Type	Percent N
1	<0.025	I or II	≥50
1	<0.025	III or IV	100
1	>0.025	All	100
2	All Applicable	All	100
3	All Applicable	All	150
4	All Applicable	All	150

for columns and/or piers, column, or pier height (ft.)

for hinges within a span, average height of the adjacent two columns or piers (ft.)

0.0 for single-span bridges (ft.)

S = skew of support measured from line normal to span (°)

Table 4.7.4.4-1 Percentage N by Zone and Acceleration Coefficient A_S, Specified in Eq. 3.10.4.2-2.

Zone	Acceleration Coefficient, A_S	Percent, N
1	≤ 0.05	≥ 75
1	**≥ 0.05**	100
2	All Applicable	150
3	All Applicable	150
4	All Applicable	150

4.7.4.5 *P*-Δ Requirements

The displacement of any column or pier in the longitudinal or transverse direction shall satisfy:

$$\Delta P_u < 0.25\phi M_n \quad (4.7.4.5\text{-}1)$$

in which:

$$\Delta = R_d \Delta_e \quad (4.7.4.5\text{-}2)$$

- If $T < 1.25T_s$, then:

$$R_d = \left(1 - \frac{1}{R}\right)\frac{1.25T_s}{T} + \frac{1}{R} \quad (4.7.4.5\text{-}3)$$

- If $T \geq 1.25T_s$, then:

$$R_d = 1 \quad (4.7.4.5\text{-}4)$$

where:

Δ = displacement of the point of contraflexure in the column or pier relative to the point of fixity for the foundation (in.)

Δ_e = displacement calculated from elastic seismic analysis (in.)

T = period of fundamental mode of vibration (sec.)

T_S = reference period specified in Article 3.10.4.2 (sec.)

C4.7.4.5

Bridges subject to earthquake ground motion may be susceptible to instability due to *P*-Δ effects. Inadequate strength can result in ratcheting of structural displacements to larger and larger values causing excessive ductility demand on plastic hinges in the columns, large residual deformations, and possibly collapse. The maximum value for Δ given in this Article is intended to limit the displacements such that *P*-Δ effects will not significantly affect the response of the bridge during an earthquake.

P-Δ effects lead to a loss in strength once yielding occurs in the columns of a bridge. In severe cases, this can result in the force-displacement relationship having a negative slope once yield is fully developed. The value for Δ given by Eq. 1 is such that this reduction in strength is limited to 25 percent of the yield strength of the pier or bent.

An explicit *P*-Δ check was not required in the previous edition of these Specifications but has been introduced herein because two conservative provisions have been relaxed in this revised edition. These are:

- The shape of the response spectrum (Figure 3.10.4.1-1) has been changed from being proportional to $1/T^{2/3}$ to $1/T$. The reason for the $1/T^{2/3}$ provision in the previous edition was to give conservative estimates of force and displacement in bridges with longer periods (>1.0 secs) which, in an indirect way, provided for such effects as *P*-Δ. With the change of the spectrum to being proportional to $1/T$, an explicit check for *P*-Δ is now required.

R = R-factor specified in Article 3.10.7

P_u = axial load on column or pier (kip)

ϕ = flexural resistance factor for column specified in Article 5.10.11.4.1b

M_n = nominal flexural strength of column or pier calculated at the axial load on the column or pier(kip-ft.)

- The flexural resistance factor, ϕ, for seismic design of columns with high axial loads has been increased from a minimum value of 0.5 to 0.9 (Article 5.10.11.4.1b). Use of a low resistance factor led to additional strength being provided in heavily loaded columns that could be used to offset reductions due to P-Δ, in the previous edition. The increased value for ϕ now permitted in Section 5 is a second reason for requiring an explicit check for P-Δ.

4.7.5 Analysis for Collision Loads

Where permitted by the provisions of Section 3, dynamic analysis for ship collision may be replaced by an equivalent static elastic analysis. Where an inelastic analysis is specified, the effect of other loads that may also be present shall be considered.

4.7.6 Analysis of Blast Effects

As a minimum, bridge components analyzed for blast forces should be designed for the dynamic effects resulting from the blast pressure on the structure. The results of an equivalent static analysis shall not be used for this purpose.

4.7.6

Localized spall and breach damage should be accounted for when designing bridge components for blast forces. Data available at the time these provisions were developed are not sufficient to develop expressions for estimating the extent of spall/breach in concrete columns; however, spall and breach damage can be estimated for other types of components using guidelines found in Department of the Army (1986).

Due to the uncertainties that exist when considering likely attack scenarios and associated blast loads, an appropriate equivalent static load can not be used for design. Moreover, the highly impulsive nature of blast loads warrants the consideration of inertial effects during the analysis of a structural component. Therefore, an equivalent static analysis is not acceptable for the design of any structural member. Information on designing structures to resist blast loads may be found in ASCE (*1997*), Department of the Army (*1990*), E. J. Conrath, et al. (*1999*), J. M. Biggs (*1964*), and W. Bounds (*1998*).

4.8 ANALYSIS BY PHYSICAL MODELS

4.8.1 Scale Model Testing

To establish and/or to verify structural behavior, the Owner may require the testing of scale models of structures and/or parts thereof. The dimensional and material properties of the structure, as well as its boundary conditions and loads, shall be modeled as accurately as possible. For dynamic analysis, inertial scaling, load/excitation, and damping functions shall be applied as appropriate. For strength limit state tests, factored dead load shall be simulated. The instrumentation shall not significantly influence the response of the model.

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4.8.2 Bridge Testing

Existing bridges may be instrumented and results obtained under various conditions of traffic and/or environmental loads or load tested with special purpose vehicles to establish force effects and/or the load-carrying capacity of the bridge.

C4.8.2

These measured force effects may be used to project fatigue life, to serve as a basis for similar designs, to establish permissible weight limits, to aid in issuing permits, or to establish a basis of prioritizing rehabilitation or retrofit.

REFERENCES

AASHTO. 1983. *Guide Specification for Seismic Design of Highway Bridges*. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2000. *Guide Specifications for Seismic Isolation Design*, 2nd Edition, GSID-2. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2001. *Standard Specifications for Structural Supports for Highway Signs, Luminaries, and Traffic Signals*, 4th Edition, LTS-4. American Association of State Highway and Transportation Officials, Washington, DC.

ACI Committee 435. 1986. *State-of-Art Report on Temperature-Induced Deflections of Reinforced Concrete Members*. SP-86-1. ACI 435.7R-85. American Concrete Institute, Farmington Hills, MI.

ACI. 2002. *Building Code Requirements for Structural Concrete and Commentary*, ACI 318-02 and ACI 318R-02. American Concrete Institute, Farmington, Hill, MI.

AISC. 1993. "Load and Resistance Factor Design." *Specification for Structural Steel Buildings and Commentary*, 2nd Edition. American Institute of Steel Construction, Chicago, IL.

AISC. 1999. *LRFD Specifications for Structural Steel Buildings*, 3rd Edition. American Institute of Steel Construction, Chicago, IL.

Aristizabal, J. D. 1987. "Tapered Beam and Column Elements in Unbraced Frame Structures." *Journal of Computing in Civil Engineering*, Vol. 1, No. 1, January 1987, pp. 35–49.

ASCE. 1961. "Wind Forces on Structures." *Transactions of the ASCE*, American Society of Civil Engineers, New York, NY, Vol. 126, No. 3269.

ASCE. 1971. "Guide for Design of Transmission Towers." *Manuals and Reports on Engineering Practice*, No. 52, American Society of Civil Engineers, New York, NY, pp. 1–47.

ASCE Committee on Cable-Suspended Bridges. 1991. *Guidelines for Design of Cable-Stayed Bridges*. Committee on Cable-Suspended Bridges, American Society of Civil Engineers, New York, NY.

ASCE Task Committee on Effective Length. 1997. *Effective Length and Notional Load Approaches for Assessing Frame Stability: Implementation for American Steel Design*. Task Committee on Effective Length, American Society of Civil Engineers, Reston, VA.

Astaneh-Asl, A., and S. C. Goel. 1984. "Cyclic In-Plane Buckling of Double Angle Bracing." *Journal of Structural Engineering*, American Society of Civil Engineers, New York, NY, Vol. 110, No. 9, September 1984, pp. 2036–2055.

Astaneh-Asl, A., S. C. Goel, and R. D. Hanson. 1985. "Cyclic Out-of-Plane Buckling of Double Angle Bracing." *Journal of Structural Engineering*, American Society of Civil Engineers, New York, NY, Vol. 111, No. 5, May 1985, pp. 1135–1153.

ATC. 1981. *Seismic Design Guidelines for Highway Bridges*. ATC-6. Applied Technology Council, Berkeley, CA.

Basu, S., and M. Chi. 1981a. *Analytic Study for Fatigue of Highway Bridge Cables*, FHWA-RD-81-090. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Basu, S., and M. Chi. 1981b. *Design Manual for Bridge Structural Members under Wind-Induced Excitation*, FHWA-TS-81-206. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Chen, W. F., and E. M. Lui. 1991. *Stability Design of Steel Frames*. CRC Press, Boca Raton, FL.

Clough, R. W., and J. Penzian. 1975. *Dynamics of Structures*. McGraw Hill, New York, NY.

Danon, J. R. and W. L. Gamble. 1977. "Time-Dependent Deformations and Losses in Concrete Bridges Built by the Cantilever Method." *Civil Engineering Studies, Structural Research Series*. University of Illinois at Urbana–Champaign, Department of Civil Engineering, No. 437, January 1977, p. 169.

Davis, R., J. Kozak, and C. Scheffey. 1965. *Structural Behavior of a Box Girder Bridge*. State of California Highway Transportation Agency, Department of Public Works, Division of Highways, Bridge Department, in cooperation with U.S. Department of Commerce, Bureau of Public Roads, and the University of California, Berkeley, CA, May 1965.

Disque, R. O. 1973. "Inelastic K-Factor in Design." *AISC Engineering Journal*, American Institute of Steel Construction, Chicago, IL, Vol. 10, 2nd Qtr., p. 33.

Duan, L. and W. F. Chen. 1988. "Effective Length Factor for Columns in Braced Frames." *Journal of Structural Engineering*, American Society of Civil Engineers, Vol. 114, No. 10, October, 1988, pp. 2357–2370.

Duan, L., and W. F. Chen. 1989. "Effective Length Factor for Columns in Unbraced Frames." *Journal of Structural Engineering*, American Society of Civil Engineers, New York, NY, Vol. 115, No. 1, January 1989, pp. 149–165.

Duan, L., W. S. King, and W. F. Chen. 1993. "K-factor Equation to Alignment Charts for Column Design." *ACI Structural Journal*, Vol. 90, No. 3, May–June, 1993, pp. 242–248.

Eby, C. C., J. M. Kulicki, C. N. Kostem, and M. A. Zellin. 1973. *The Evaluation of St. Venant Torsional Constants for Prestressed Concrete I-Beams*. Fritz Laboratory Report No. 400.12. Lehigh University, Bethlehem, PA.

Frederick, G. R., C. V. Ardis, K. M. Tarhini, and B. Koo. 1988. "Investigation of the Structural Adequacy of C 850 Box Culverts," *Transportation Research Record* 1191, Transportation Research Board, National Research Council, Washington, DC.

Galambos, T. V., ed. 1998. *Guide to Stability Design for Metal Structures*, 5th Edition. Structural Stability Research Council. John Wiley and Sons, Inc., New York, NY.

Ghali, A., and A. M. Neville. 1989. *Structural Analysis: A Unified Classical and Matrix Approach*, 3rd Edition. Chapman Hall, New York, NY.

Goel, S. C., and A. A. El-Tayem. 1986. "Cyclic Load Behavior of Angle X-Bracing." *Journal of Structural Engineering*, American Society of Civil Engineers, New York, NY, Vol. 112, No. 11, November 1986, pp. 2528–2539.

Guyan, R. J. 1965. "Reduction of Stiffness and Mass Matrices." *AIAA Journal*, American Institute of Aeronautics and Astronautics, Reston, VA, Vol. 3, No. 2, February 1965, p. 380.

Hall, D. H., and C. H. Yoo. 1996. *I-Girder Curvature Study*. Interim Report, NCHRP Project 12-38 submitted to NCHRP, Transportation Research Board, Washington, DC, pp. 1–72 (or see Appendix A of NCHRP Report 424: Improved Design Specifications for Horizontally Curved Steel Highway Bridges, pp. 49–74).

Haroun, N. M., and R. Sheperd. 1986. "Inelastic Behavior of X-Bracing in Plane Frames." *Journal of Structural Engineering*, American Society of Civil Engineers, New York, NY, Vol. 112, No. 4, April 1986, pp. 764–780.

Higgins, C. 2003. "LRFD Orthotropic Plate Model for Determining Live Load Moments in Concrete Filled Grid Bridge Decks." *Journal of Bridge Engineering*, American Society of Civil Engineers, Reston, VA, January/February 2003, pp. 20–28.

Highway Engineering Division. 1991. *Ontario Highway Bridge Design Code*. Highway Engineering Division, Ministry of Transportation and Communications, Toronto, Canada.

Homberg, H. 1968. *Fahrbahnplatten mit Verandlicher Dicke*. Springer-Verlag, New York, NY.

James, R. W. 1984. "Behavior of ASTM C 850 Concrete Box Culverts Without Shear Connectors," *Transportation Research Record* 1001, Transportation Research Board, National Research Council, Washington, DC.

Johnston, S. B., and A. H. Mattock. 1967. *Lateral Distribution of Load in Composite Box Girder Bridges*. Highway Research Record No. 167, Highway Research Board, Washington, DC.

Karabalis, D. L. 1983. "Static, Dynamic and Stability Analysis of Structures Composed of Tapered Beams." *Computers and Structures*, Vol. 16, No. 6, pp. 731–748.

Ketchum, M. S. 1986. "Short Cuts for Calculating Deflections." *Structural Engineering Practice: Analysis, Design, Management*, Vol. 3, No. 2, pp. 83–91.

King, Csagoly P. F., and J. W. Fisher. 1975. *Field Testing of the Aquasabon River Bridge*. Ontario, Canada.

Liu, H. 1991. *Wind Engineering: A Handbook for Structural Engineers*. Prentice Hall, Englewood Cliffs, NJ.

McGrath, T. J., A. A. Liepins, J. L. Beaver, and B. P. Strohman. 2004. *Live Load Distribution Widths for Reinforced Concrete Box Culverts*. A Study for the Pennsylvania Department of Transportation, Simpson Gumpertz & Heger Inc., Waltham, MA.

Modjeski and Masters, Inc. 1994. *Report to Pennsylvania Department of Transportation*. Harrisburg, PA.

Moffatt, K. R., and P. J. Dowling. 1975. "Shear Lag in Steel Box Girder Bridges." *The Structural Engineer*, October 1975, pp. 439–447.

Moffatt, K. R., and P. J. Dowling. 1976. "Discussion." *The Structural Engineer*, August 1976, pp. 285–297.

Nettleton, D. A. 1977. *Arch Bridges*. Bridge Division, Office of Engineering, Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Paz, M. 1984. "Dynamic Condensation." *AIAA Journal*, American Institute of Aeronautics and Astronautics, Reston, VA, Vol. 22, No. 5, May 1984, pp. 724–727.

Paz, M. 1985. *Structural Dynamics*, 2nd Edition. Van Nosstrand Reinhold Company, New York, NY.

Peck, R. B., W. E. Hanson, and T. H. Thornburn. 1974. *Foundation Engineering*, 2nd Edition. John Wiley and Sons, Inc., New York, NY, p. 514.

Podolny, W., and J. B. Scalzi. 1986. *Construction and Design of Cable-Stayed Bridges*, 2nd Edition. Wiley-Interscience, New York, NY.

Przemieniecki, J. S. 1968. *Theory of Matrix Structural Analysis*. McGraw Hill, New York, NY.

Pucher, A. 1964. *Influence Surfaces of Elastic Plates*, 4th Edition. Springer-Verlag, New York, NY.

Richardson, Gordon and Associates (presently HDR, Pittsburgh office). 1976. "Curved Girder Workshop Lecture Notes." Prepared under Contract No. DOT-FH-11-8815. Federal Highway Administration, U.S. Department of Transportation. Four-day workshop presented in Albany, Denver, and Portland, September–October 1976, pp. 31–38.

Salmon, C. G., and J. E. Johnson. 1990. *Steel Structures: Design and Behavior, Emphasizing Load, and Resistance Factor Design*, 3rd Edition. Harper and Row, New York, NY.

Scanlan, R. H. 1975. *Recent Methods in the Application of Test Results to the Wind Design of Long Suspended-Span Bridges*, FHWA-RD-75-115. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Shushkewich, K. W. 1986. "Time-Dependent Analysis of Segmental Bridges." *Computers and Structures*, Vol. 23, No. 1, pp. 95–118.

Simiu, E. 1973. "Logarithmic Profiles and Design Wind Speeds." *Journal of the Mechanics Division*, American Society of Civil Engineers, New York, NY, Vol. 99, No. EM5, October 1973, pp. 1073–1083.

Simiu, E. 1976. "Equivalent Static Wind Loads for Tall Building Design." *Journal of the Structures Division*, American Society of Civil Engineers, New York, NY, Vol. 102, No. ST4, April 1976, pp. 719–737.

Simiu, E., and R. H. Scanlan. 1978. *Wind Effects on Structures*. Wiley-Interscience, New York, NY.

Smith, Jr., C. V. 1976. "On Inelastic Column Buckling." *AISC Engineering Journal*, American Institute of Steel Construction, Chicago, IL, Vol. 13, 3rd Qtr., pp. 86–88.

Song, S. T., Y. H. Chai, and S. E. Hida. 2001. *Live Load Distribution in Multi-Cell Box-Girder Bridges and its Comparisons with AASHTO LRFD Bridge Design Specifications*, UCD-STR-01-1, University of California, Davis, CA, July 2001.

Song, S. T., Y. H. Chai, and S. E. Hida. 2003. "Live Load Distribution Factors for Concrete Box-Girder Bridges." *Journal of Bridge Engineering*, American Society of Civil Engineers, Vol. 8, No. 5, pp. 273–280.

Troitsky, M. S. 1977. *Cable-Stayed Bridges*. Crosby Lockwood Staples, London, England, p. 385.

Tung, D. H. H., and R. S. Fountain. 1970. "Approximate Torsional Analysis of Curved Box Girders by the M/R-Method." *Engineering Journal*, American Institute of Steel Construction, Vol. 7, No. 3, pp. 65–74.

United States Steel. 1984. "V-Load Analysis." Available from the National Steel Bridge Alliance, Chicago, IL, pp. 1–56.

White, D. W., and J. F. Hajjar. 1991. "Application of Second-Order Elastic Analysis in LRFD: Research to Practice." *AISC Engineering Journal*, American Institute of Steel Construction, Chicago, IL, Vol. 28, No. 4, pp. 133–148.

Wilson, E. L., A. Der Kiureghian, and E. P. Bayo. 1981. "A Replacement for the SRSS Method in Seismic Analysis." *International Journal of Earthquake Engineering and Structural Dynamics*, Vol. 9, pp. 187–194.

Wolchuk, R. 1963. *Design Manual for Orthotropic Steel Plate Deck Bridges*. American Institute of Steel Construction, Chicago, IL.

Wolchuk, R. 1990. "Steel-Plate-Deck Bridges." In *Structural Engineering Handbook*, 3rd Edition. E. H. Gaylord and C. N. Gaylord, eds. McGraw-Hill, New York, NY, pp. 19-1–19-28.

Wright, R. N., and S. R. Abdel-Samad. 1968. "BEF Analogy for Analysis of Box Girders." *Journal of the Structural Division*, American Society of Civil Engineers, Vol. 94, No. ST7, pp. 1719–1743.

Yura, J. A. 1971. "The Effective Length of Columns in Unbraced Frames." *AISC Engineering Journal*, American Institute of Steel Construction, Chicago, IL, Vol. 8, No. 2., April 1971, pp. 37–42.

Yen, B. T., T. Huang, and D. V. VanHorn. 1995. *Field Testing of a Steel Bridge and a Prestressed Concrete Bridge*, Research Project No. 86-05, Final Report, Vol. II, Pennsylvania Department of Transportation Office of Research and Special Studies, Fritz Engineering Laboratory Report No. 519.2, Lehigh University, Bethlehem, PA.

Zokaie, T. 1998, 1999, 2000. Private Correspondence.

Zokaie, T., T. A. Osterkamp, and R. A. Imbsen. 1991. *Distribution of Wheel Loads on Highway Bridges*, NCHRP Report 12-2611. Transportation Research Board, National Research Council, Washington, DC.

APPENDIX A4 DECK SLAB DESIGN TABLE

Table 1 may be used in determining the design moments for different girder arrangements. The following assumptions and limitations were used in developing this table and should be considered when using the listed values for design:

- The moments are calculated using the equivalent strip method as applied to concrete slabs supported on parallel girders.
- Multiple presence factors and the dynamic load allowance are included in the tabulated values.
- See Article 4.6.2.1.6 for the distance between the center of the girders to the location of the design sections for negative moments in the deck. Interpolation between the listed values may be used for distances other than those listed in Table 1.
- The moments are applicable for decks supported on at least three girders and having a width of not less than 14.0 ft. between the centerlines of the exterior girders.
- The moments represent the upper bound for the moments in the interior regions of the slab and, for any specific girder spacing, were taken as the maximum value calculated, assuming different number of girders in the bridge cross-section. For each combination of girder spacing and number of girders, the following two cases of overhang width were considered:

  (a) Minimum total overhang width of 21.0 in. measured from the center of the exterior girder, and

  (b) Maximum total overhang width equal to the smaller of 0.625 times the girder spacing and 6.0 ft.

  A railing system width of 21.0 in. was used to determine the clear overhang width. For other widths of railing systems, the difference in the moments in the interior regions of the deck is expected to be within the acceptable limits for practical design.

- The moments do not apply to the deck overhangs and the adjacent regions of the deck that need to be designed taking into account the provisions of Article A13.4.1.

Table A4-1 Maximum Live Load Moments Per Unit Width, kip-ft./ft.

S		Positive Moment	NEGATIVE MOMENT						
			Distance from CL of Girder to Design Section for Negative Moment						
			0.0 in.	3 in.	6 in.	9 in.	12 in.	18 in.	24 in.
4'	-0"	4.68	2.68	2.07	1.74	1.60	1.50	1.34	1.25
4'	-3"	4.66	2.73	2.25	1.95	1.74	1.57	1.33	1.20
4'	-6"	4.63	3.00	2.58	2.19	1.90	1.65	1.32	1.18
4'	-9"	4.64	3.38	2.90	2.43	2.07	1.74	1.29	1.20
5'	-0"	4.65	3.74	3.20	2.66	2.24	1.83	1.26	1.12
5'	-3"	4.67	4.06	3.47	2.89	2.41	1.95	1.28	0.98
5'	-6"	4.71	4.36	3.73	3.11	2.58	2.07	1.30	0.99
5'	-9"	4.77	4.63	3.97	3.31	2.73	2.19	1.32	1.02
6'	-0"	4.83	4.88	4.19	3.50	2.88	2.31	1.39	1.07
6'	-3"	4.91	5.10	4.39	3.68	3.02	2.42	1.45	1.13
6'	-6"	5.00	5.31	4.57	3.84	3.15	2.53	1.50	1.20
6'	-9"	5.10	5.50	4.74	3.99	3.27	2.64	1.58	1.28
7'	-0"	5.21	5.98	5.17	4.36	3.56	2.84	1.63	1.37
7'	-3"	5.32	6.13	5.31	4.49	3.68	2.96	1.65	1.51
7'	-6"	5.44	6.26	5.43	4.61	3.78	3.15	1.88	1.72
7'	-9"	5.56	6.38	5.54	4.71	3.88	3.30	2.21	1.94
8'	-0"	5.69	6.48	5.65	4.81	3.98	3.43	2.49	2.16
8'	-3"	5.83	6.58	5.74	4.90	4.06	3.53	2.74	2.37
8'	-6"	5.99	6.66	5.82	4.98	4.14	3.61	2.96	2.58
8'	-9"	6.14	6.74	5.90	5.06	4.22	3.67	3.15	2.79
9'	-0"	6.29	6.81	5.97	5.13	4.28	3.71	3.31	3.00
9'	-3"	6.44	6.87	6.03	5.19	4.40	3.82	3.47	3.20
9'	-6"	6.59	7.15	6.31	5.46	4.66	4.04	3.68	3.39
9'	-9"	6.74	7.51	6.65	5.80	4.94	4.21	3.89	3.58
10'	-0"	6.89	7.85	6.99	6.13	5.26	4.41	4.09	3.77
10'	-3"	7.03	8.19	7.32	6.45	5.58	4.71	4.29	3.96
10'	-6"	7.17	8.52	7.64	6.77	5.89	5.02	4.48	4.15
10'	-9"	7.32	8.83	7.95	7.08	6.20	5.32	4.68	4.34
11'	-0"	7.46	9.14	8.26	7.38	6.50	5.62	4.86	4.52
11'	-3"	7.60	9.44	8.55	7.67	6.79	5.91	5.04	4.70
11'	-6"	7.74	9.72	8.84	7.96	7.07	6.19	5.22	4.87
11'	-9"	7.88	10.01	9.12	8.24	7.36	6.47	5.40	5.05
12'	-0"	8.01	10.28	9.40	8.51	7.63	6.74	5.56	5.21
12'	-3"	8.15	10.55	9.67	8.78	7.90	7.02	5.75	5.38
12'	-6"	8.28	10.81	9.93	9.04	8.16	7.28	5.97	5.54
12'	-9"	8.41	11.06	10.18	9.30	8.42	7.54	6.18	5.70
13'	-0"	8.54	11.31	10.43	9.55	8.67	7.79	6.38	5.86
13'	-3"	8.66	11.55	10.67	9.80	8.92	8.04	6.59	6.01
13'	-6"	8.78	11.79	10.91	10.03	9.16	8.28	6.79	6.16
13'	-9"	8.90	12.02	11.14	10.27	9.40	8.52	6.99	6.30
14'	-0"	9.02	12.24	11.37	10.50	9.63	8.76	7.18	6.45
14'	-3"	9.14	12.46	11.59	10.72	9.85	8.99	7.38	6.58
14'	-6"	9.25	12.67	11.81	10.94	10.08	9.21	7.57	6.72
14'	-9"	9.36	12.88	12.02	11.16	10.30	9.44	7.76	6.86
15'	-0"	9.47	13.09	12.23	11.37	10.51	9.65	7.94	7.02