SECTION 3: LOADS AND LOAD FACTORS

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LOADS AND LOAD FACTORS

3.1 SCOPE

This section specifies minimum requirements for loads and forces, the limits of their application, load factors, and load combinations used for the design of new bridges. The load provisions may also be applied to the structural evaluation of existing bridges.

Where multiple performance levels are provided, the selection of the design performance level is the responsibility of the Owner.

A minimum load factor is specified for force effects that may develop during construction. Additional requirements for construction of segmental concrete bridges are specified in Article 5.14.2.

C3.1

This section includes, in addition to traditional loads, the force effects due to collisions, earthquakes, and settlement and distortion of the structure.

Vehicle and vessel collisions, earthquakes, and aeroelastic instability develop force effects that are dependent upon structural response. Therefore, such force effects cannot be determined without analysis and/or testing.

With the exception of segmental concrete bridges, construction loads are not provided, but the Designer should obtain pertinent information from prospective contractors.

3.2 DEFINITIONS

Active Earth Pressure—Lateral pressure resulting from the retention of the earth by a structure or component that is tending to move away from the soil mass.

Active Earth Wedge—Wedge of earth with a tendency to become mobile if not retained by a structure or component.

Aeroelastic Vibration—Periodic, elastic response of a structure to wind.

Apparent Earth Pressure—Lateral pressure distribution for anchored walls constructed from the top down.

Axle Unit—Single axle or tandem axle.

Berm—An earthwork used to redirect or slow down impinging vehicles or vessels and to stabilize fill, embankment, or soft ground and cut slopes.

Centrifugal Force—A lateral force resulting from a change in the direction of a vehicle's movement.

Damper—A device that transfers and reduces forces between superstructure elements and/or superstructure and substructure elements, while permitting thermal movements. The device provides damping by dissipating energy under seismic, braking or other dynamic loads.

Deep Draft Waterways—A navigable waterway used by merchant ships with loaded drafts of 14–60+ ft.

Design Lane—A notional traffic lane positioned transversely on the roadway.

Design Thermal Movement Range—The structure movement range resulting from the difference between the maximum design temperature and minimum design temperature as defined in Article 3.12.

Design Water Depth—Depth of water at mean high water.

Distortion—Change in structural geometry.

Dolphin—Protective object that may have its own fender system and that is usually circular in plan and structurally independent from the bridge.

Dynamic Load Allowance—An increase in the applied static force effects to account for the dynamic interaction between the bridge and moving vehicles.

Equivalent Fluid— A notional substance whose density is such that it would exert the same pressure as the soil it is seen to replace for computational purposes.

Exposed—A condition in which a portion of a bridge's substructure or superstructure is subject to physical contact by any portion of a colliding vessel's bow, deck house, or mast.

Extreme—A maximum or a minimum.

Fender—Protection hardware attached to the structural component to be protected or used to delineate channels or to redirect aberrant vessels.

Frazil Ice—Ice resulting from turbulent water flow.

Global—Pertinent to the entire superstructure or to the whole bridge.

Influence Surface—A continuous or discretized function over a bridge deck whose value at a point, multiplied by a load acting normal to the deck at that point, yields the force effect being sought.

Knot—A velocity of 1.1508 mph.

Lane—The area of deck receiving one vehicle or one uniform load line.

Lever Rule—The statical summation of moments about one point to calculate the reaction at a second point.

Liquefaction—The loss of shear strength in a saturated soil due to excess hydrostatic pressure. In saturated, cohesionless soils, such a strength loss can result from loads that are applied instantaneously or cyclicly, particularly in loose fine to medium sands that are uniformly graded.

Load—The effect of acceleration, including that due to gravity, imposed deformation, or volumetric change.

Local—Pertinent to a component or subassembly of components.

Mode of Vibration—A shape of dynamic deformation associated with a frequency of vibration.

Navigable Waterway—A waterway, determined by the U.S. Coast Guard as being suitable for interstate or foreign commerce, as described in 33CFR205-25.

Nominal Load—An arbitrarily selected design load level.

Normally Consolidated Soil—A soil for which the current effective overburden pressure is the same as the maximum pressure that has been experienced.

Overconsolidated Soil—A soil that has been under greater overburden pressure than currently exists.

Overall Stability—Stability of the entire retaining wall or abutment structure and is determined by evaluating potential slip surfaces located outside of the whole structure.

Overconsolidation Ratio—Ratio of the maximum preconsolidation pressure to the overburden pressure.

Passive Earth Pressure—Lateral pressure resulting from the earth's resistance to the lateral movement of a structure or component into the soil mass.

Permanent Loads—Loads and forces that are, or are assumed to be, constant upon completion of construction.

Permit Vehicle—Any vehicle whose right to travel is administratively restricted in any way due to its weight or size.

Reliability Index—A quantitative assessment of safety expressed as the ratio of the difference between the mean resistance and mean force effect to the combined standard deviation of resistance and force effect.

Restrainers—A system of high-strength cables or rods that transfers forces between superstructure elements and/or superstructure and substructure elements under seismic or other dynamic loads after an initial slack is taken up, while permitting thermal movements.

Roadway Width—Clear space between barriers and/or curbs.

Setting Temperature—A structure's average temperature, which is used to determine the dimensions of a structure when a component is added or set in place.

Shallow Draft Waterways—A navigable waterway used primarily by barge vessels with loaded drafts of less than 9–10 ft.

Shock Transmission Unit (STU)—A device that provides a temporary rigid link between superstructure elements and/or superstructure and substructure elements under seismic, braking or other dynamic loads, while permitting thermal movements.

Structurally Continuous Barrier—A barrier, or any part thereof, that is interrupted only at deck joints.

Substructure—Structural parts of the bridge that support the horizontal span.

Superstructure—Structural parts of the bridge that provide the horizontal span.

Surcharge—A load used to model the weight of earth fill or other loads applied to the top of the retained material.

Tandem—Two closely spaced axles, usually connected to the same under-carriage, by which the equalization of load between the axles is enhanced.

Tonne—2.205 kip.

Wall Friction Angle—An angle whose arctangent represents the apparent friction between a wall and a soil mass.

Wheel—Single or dual tire at one end of an axle.

Wheel Line—A transverse or longitudinal grouping of wheels.

3.3 NOTATION

3.3.1 General

A = plan area of ice floe (ft.^{2}); seismic acceleration coefficient; depth of temperature gradient (in.) (C3.9.2.3) (3.10.2) (3.12.3)
AEP = apparent earth pressure for anchored walls (ksf) (3.4.1)
AF = annual frequency of bridge element collapse (number/yr.) (C3.14.4)
a = length of uniform deceleration at braking (ft.); truncated distance (ft.); average bow damage length (ft.) (C3.6.4) (C3.9.5) (C3.14.9)
a_B = bow damage length of standard hopper barge (ft.) (3.14.11)
a_s = bow damage length of ship (ft.) (3.14.9)
B' = equivalent footing width (ft.) (3.11.6.3)
B_e = width of excavation (ft.) (3.11.5.7.2b)
B_M = beam (width) for barge, barge tows, and ship vessels (ft.) (C3.14.5.1)
B_p = width of bridge pier (ft.) (3.14.5.3)
BR = vehicular braking force; base rate of vessel aberrancy (3.3.2) (3.14.5.2.3)
b = braking force coefficient; width of a discrete vertical wall element (ft.) (C3.6.4) (3.11.5.6)
b_f = width of applied load or footing (ft.) (3.11.6.3)
C = coefficient to compute centrifugal forces; constant for terrain conditions in relation to wind approach (3.6.3) (C3.8.1.1)
C_a = coefficient for force due to crushing of ice (3.9.2.2)
C_D = drag coefficient (sec.^{2} lbs./ft.^{4}) (3.7.3.1)
C_H = hydrodynamic mass coefficient (3.14.7)

C_L	= lateral drag coefficient (C3.7.3.1)
C_n	= coefficient for nose inclination to compute F_b (3.9.2.2)
C_{sm}	= elastic seismic response coefficient for the m^{th} mode of vibration (3.10.1)
c	= soil cohesion (ksf) (3.11.5.4)
c_f	= distance from back of a wall face to the front of an applied load or footing (ft.) (3.11.6.3)
D	= depth of embedment for a permanent nongravity cantilever wall with discrete vertical wall elements (ft.) (3.11.5.6)
D_B	= bow depth (ft.) (C3.14.5.1)
D_E	= minimum depth of earth cover (ft.) (3.6.2.2)
D_o	= calculated embedment depth to provide equilibrium for nongravity cantilevered with continuous vertical elements by the simplified method (ft.) (3.11.5.6)
DWT	= size of vessel based on deadweight tonnage (tonne) (C3.14.1)
D_1	= effective width of applied load at any depth (ft.) (3.11.6.3)
d	= depth of potential base failure surface below base of excavation (ft.); horizontal distance from the back of a wall face to the centerline of an applied load (ft.) (3.11.5.7.2b) (3.11.6.3)
E	= Young's modulus (ksf) (C3.9.5)
E_B	= deformation energy (kip-ft.) (C3.14.11)
e'	= eccentricity of load on footing (ft.) (3.11.6.3)
F	= longitudinal force on pier due to ice floe (kip); force required to fail an ice sheet (kip/ft.); force at base of nongravity cantilevered wall required to provide force equilibrium (kip/ft.) (3.9.2.2) (C3.9.5) (3.11.5.6)
F_b	= horizontal force due to failure of ice flow due to bending (kip) (3.9.2.2)
F_c	= horizontal force due to crushing of ice (kip) (3.9.2.2)
FS_{BH}	= factor of safety against basal heave (C3.11.5.6)
F_t	= transverse force on pier due to ice flow (kip) (3.9.2.4.1)
F_v	= vertical ice force due to adhesion (kip) (3.9.5)
F_1	= lateral force due to earth pressure (kip/ft.) (3.11.6.3)
F_2	= lateral force due to traffic surcharge (kip/ft.) (3.11.6.3)
f	= constant applied in calculating the coefficient C used to compute centrifugal forces, taken equal to 4/3 for load combinations other than fatigue and 1.0 for fatigue (3.6.3)
f'_c	= specified compressive strength of concrete for use in design (ksi) (3.5.1)
g	= gravitational acceleration (ft./sec.^{2}) (3.6.3)
H	= ultimate bridge element strength (kip); final height of retaining wall (ft.); total excavation depth (ft.); resistance of bridge component to a horizontal force (kip) (C3.11.1) (3.11.5.7.1) (3.14.5.4)
H_L	= depth of barge head-block on its bow (ft.) (3.14.14.1)
H_p	= ultimate bridge pier resistance (kip) (3.14.5.4)
H_s	= ultimate bridge superstructure resistance (kip) (3.14.5.4)
H_1	= distance from ground surface to uppermost ground anchor (ft.) (3.11.5.7.1)
H_{n+1}	= distance from base of excavation to lowermost ground anchor (ft.) (3.11.5.7.1)
h	= notional height of earth pressure diagram (ft.) (3.11.5.7)
h_{eq}	= equivalent height of soil for vehicular load (ft.) (3.11.6.4)
IM	= dynamic load allowance (C3.6.1.2.5)
KE	= design impact energy of vessel collision (kip-ft.) (3.14.7)
K_1	= ice force reduction factor for small streams (C3.9.2.3)
k	= coefficient of lateral earth pressure (3.11.6.2)
k_a	= coefficient of active lateral earth pressure (3.11.5.1)
k_o	= coefficient of at rest lateral earth pressure (3.11.5.1)
k_p	= coefficient of passive lateral earth pressure (3.11.5.1)
k_s	= coefficient of earth pressure due to surcharge (3.11.6.1)
L	= perimeter of pier (ft.); length of soil reinforcing elements in an MSE wall (ft.); length of footing (ft.); expansion length (in.) (3.9.5) (3.11.5.8) (3.11.6.3) (3.12.2.3)
ℓ	= characteristic length (ft.); center-to-center spacing of vertical wall elements (ft.) (C3.9.5) (3.11.5.6)
LOA	= length overall of ship or barge tow including the tug or tow boat (ft.) (3.14.5)
m	= multiple presence factor (3.6.1.1.2)
N	= number of one-way passages of vessels navigating through the bridge (number/yr.) (3.14.5)
N_s	= stability number (3.11.5.6)
OCR	= overconsolidation ratio (3.11.5.2)
P	= maximum vertical force for single ice wedge (kip); load resulting from vessel impact (kip); concentrated wheel load (kip); live load intensity; point load (kip) (C3.9.5) (3.14.5.4) (C3.6.1.2.5) (C3.11.6.2) (3.11.6.1)

PA = probability of vessel aberrancy (3.14.5)
P_a = force resultant per unit width of wall (kip/ft.) (3.11.5.8.1)
P_B = barge collision impact force for head-on collision between barge bow and a rigid object (kip); base wind pressure corresponding to a wind speed of 100 mph (ksf) (3.14.11) (3.8.1.2)
$\overline{P_B}$ = average equivalent static barge impact force resulting from Meir-Dornberg Study (kip) (C3.14.11)
P_{BH} = ship collision impact force between ship bow and a rigid superstructure (kip) (3.14.10.1)
PC = probability of bridge collapse (3.14.5)
P_D = design wind pressure (ksf) (3.8.1.2.1)
P_{DH} = ship collision impact force between ship deck house and a rigid superstructure (kip) (3.14.5.4)
PG = geometric probability of vessel collision with bridge pier/span (3.14.5)
P_H = lateral force due to superstructure or other concentrated lateral loads (kip/ft.) (3.11.6.3)
P_h = horizontal component of resultant earth pressure on wall (kip/ft.) (3.11.5.5)
P_{MT} = ship collision impact force between ship mast and a rigid superstructure (kip) (3.14.5.4)
P_p = passive earth pressure (kip/ft.) (3.11.5.4)
P_S = ship collision impact force for head-on collision between ship bow and a rigid object (kip) (3.14.5.4)
P_v = vertical component of resultant earth pressure on wall (kip/ft.); load per linear foot of strip footing (kip/ft.) (3.11.5.5) (3.11.6.3)
P'_v = load on isolated rectangular footing or point load (kip) (3.11.6.3)
p = effective ice crushing strength (ksf); stream pressure (ksf); basic earth pressure (psf); fraction of truck traffic in a single lane; load intensity (ksf) (3.9.2.2) (3.7.3.1) (3.11.5.1) (3.6.1.4.2) (3.11.6.1)
p_a = apparent earth pressure (ksf); maximum ordinate of pressure diagram (ksf) (3.11.5.3) (3.11.5.7.1)
p_p = passive earth pressure (ksf) (3.11.5.4)
Q = total factored load; load intensity for infinitely long line loading (kip/ft.) (3.4.1) (3.11.6.2)
Q_i = force effects (3.4.1)
q = surcharge pressure (ksf) (3.11.6.3)
q_s = uniform surcharge pressure (ksf) (3.11.6.1)
R = radius of curvature (ft.); radius of circular pier (ft.); seismic response modification factor; reduction factor of lateral passive earth pressure; radial distance from point of load application to a point on the wall (ft.); reaction force to be resisted by subgrade below base of excavation (kip/ft.) (3.6.3) (3.9.5) (3.10.7.1) (3.11.5.4) (3.11.6.1) (3.11.5.7.1)
R_B = PA correction factor for bridge location (3.14.5.2.3)
R_{BH} = ratio of exposed superstructure depth to the total ship bow depth (3.14.10.1)
R_C = PA correction factor for currents parallel to vessel transit path (3.14.5.2.3)
R_D = PA correction factor for vessel traffic density (3.14.5.2.3)
R_{DH} = reduction factor for ship deck house collision force (3.14.10.2)
R_{XC} = PA correction factor for cross-currents acting perpendicular to vessel transit path (3.14.5.2.3)
r = radius of pier nose (ft.) (C3.9.2.3)
S = coefficient related to site conditions for use in determining seismic loads (3.10.5.1)
S_f = freezing index (C3.9.2.2)
S_m = shear strength of rock mass (ksf) (3.11.5.6)
S_u = undrained shear strength of cohesive soil (ksf) (3.11.5.6)
S_{ub} = undrained strength of soil below excavation base (ksf) (3.11.5.7.2b)
S_v = vertical spacing of reinforcements (ft.) (3.11.5.8.1)
T = mean daily air temperature (°F) (C3.9.2.2)
T_{hi} = horizontal load in anchor i (kip/ft.) (3.11.5.7.1)
T_m = period of vibration for m^{th} mode (sec.) (3.10.6.1)
T_{max} = applied load to reinforcement in a mechanically stabilized earth wall (kip/ft.) (3.11.5.8.2)
$T_{MaxDesign}$ = maximum design temperature used for thermal movement effects (°F) (3.12.2.1) (3.12.2.2) (3.12.2.3)
$T_{MinDesign}$ = minimum design temperature used for thermal movement effects (°F) (3.12.2.1) (3.12.2.2) (3.12.2.3)
t = thickness of ice (ft.); thickness of deck (in.) (3.9.2.2) (3.12.3)
V = design velocity of water (ft./sec.); design impact speed of vessel (ft./sec.) (3.7.3.1) (3.14.6)
V_B = base wind velocity taken as 100 mph (3.8.1.1)
V_C = waterway current component acting parallel to the vessel transit path (knots) (3.14.5.2.3)
V_{DZ} = design wind velocity at design Elevation Z (mph) (3.8.1.1)
V_{MIN} = minimum design impact velocity taken not less than the yearly mean current velocity for the bridge location (ft./sec.) (3.14.6)
V_T = vessel transit speed in the navigable channel (ft./sec.) (3.14.6)
V_{XC} = waterway current component acting perpendicular to the vessel transit path (knots) (3.14.5.2.3)

V_0 = friction velocity, a meteorological wind characteristic for various upwind surface characteristics (mph) (3.8.1.1)
V_{30} = wind speed at 30.0 ft. above low ground or water level (mph) (3.8.1.1)
v = highway design speed (ft./sec.) (3.6.3)
W = displacement weight of vessel (tonne) (C3.14.5.1)
w = width of clear roadway (ft.); width of pier at level of ice action (ft.); specific weight of water (kcf) (3.6.1.1.1) (3.9.2.2) (C3.7.3.1)
X = horizontal distance from back of wall to point of load application (ft.); distance to bridge element from the centerline of vessel transit path (ft.) (3.11.6.2) (3.14.6)
X_c = distance to edge of channel from centerline of vessel transit path (ft.) (3.14.6)
X_L = distance from centerline of vessel transit path equal to $3 \times LOA$ (ft.) (3.14.6)
X_1 = distance from the back of the wall to the start of the line load (ft.) (3.11.6.2)
X_2 = length of the line load (ft.) (3.11.6.2)
Z = structure height above low ground or water level > 30.0 ft. (ft.); depth below surface of soil (ft.); depth from the ground surface to a point on the wall under consideration (ft.); vertical distance from point of load application to the elevation of a point on the wall under consideration (ft.) (3.8.1.1) (3.11.6.3) (3.11.6.2)
Z_0 = friction length of upstream fetch, a meteorological wind characteristic (ft.) (3.8.1.1)
Z_2 = depth where effective width intersects back of wall face (ft.) (3.11.6.3)
z = depth below surface of backfill (ft.) (3.11.5.1)
α = constant for terrain conditions in relation to wind approach; coefficient for local ice condition; inclination of pier nose with respect to a vertical axis (°); inclination of back of wall with respect to a vertical axis (°); angle between foundation wall and a line connecting the point on the wall under consideration and a point on the bottom corner of the footing nearest to the wall (rad.); coefficient of thermal expansion (in./in./°F) (C3.8.1.1) (C3.9.2.2) (3.9.2.2) (C3.11.5.3) (3.11.6.2) (3.12.2.3)
B = notional slope of backfill (°) (3.11.5.8.1)
β = safety index; nose angle in a horizontal plane used to calculate transverse ice forces (°); slope of backfill surface behind retaining wall; {+ for slope up from wall; − for slope down from wall} (°) (C3.4.1) (3.9.2.4.1) (3.11.5.3)
β' = slope of ground surface in front of wall {+ for slope up from wall; − for slope down from wall} (°) (3.11.5.6)
γ = load factors; unit weight of materials (kcf); unit weight of water (kcf); unit weight of soil (kcf) (C3.4.1) (3.5.1) (C3.9.5) (3.11.5.1)
γ_s = unit weight of soil (kcf) (3.11.5.1)
γ'_s = effective soil unit weight (kcf) (3.11.5.6)
γ_{EQ} = load factor for live load applied simultaneously with seismic loads (3.4.1)
γ_{eq} = equivalent-fluid unit weight of soil (kcf) (3.11.5.5)
γ_i = load factor (3.4.1)
γ_p = load factor for permanent loading (3.4.1)
γ_{SE} = load factor for settlement (3.4.1)
γ_{TG} = load factor for temperature gradient (3.4.1)
Δ = movement of top of wall required to reach minimum active or maximum passive pressure by tilting or lateral translation (ft.) (C3.11.1) (3.11.5.5)
Δ_p = constant horizontal earth pressure due to uniform surcharge (ksf) (3.11.6.1)
Δ_{ph} = constant horizontal pressure distribution on wall resulting from various types of surcharge loading (ksf) (3.11.6.2)
Δ_T = design thermal movement range (in.) (3.12.2.3)
$\Delta\sigma_H$ = horizontal stress due to surcharge load (ksf) (3.11.6.3)
$\Delta\sigma_v$ = vertical stress due to surcharge load (ksf) (3.11.6.3)
δ = angle of truncated ice wedge (°); friction angle between fill and wall (°); angle between foundation wall and a line connecting the point on the wall under consideration and a point on the bottom corner of the footing furthest from the wall (rad.) (C3.9.5) (3.11.5.3) (3.11.6.2)
η_i = load modifier specified in Article 1.3.2; wall face batter (3.4.1) (3.11.5.9)
θ = angle of back of wall to the horizontal (°); angle of channel turn or bend (°); angle between direction of stream flow and the longitudinal axis of pier (°) (3.11.5.3) (3.14.5.2.3) (3.7.3.2)
θ_f = friction angle between ice floe and pier (°) (3.9.2.4.1)
σ = standard deviation of normal distribution (3.14.5.3)
σ_T = tensile strength of ice (ksf) (C3.9.5)
ν = Poisson's Ratio (dim.) (3.11.6.2)
ϕ = resistance factors (C3.4.1)

ϕ_f = angle of internal friction (°) (3.11.5.4)
ϕ'_f = effective angle of internal friction (°) (3.11.5.2)
ϕ_r = internal friction angle of reinforced fill (°) (3.11.6.3)
ϕ'_s = angle of internal friction of retained soil (°) (3.11.5.6)

3.3.2 Load and Load Designation

The following permanent and transient loads and forces shall be considered:

- Permanent Loads

DD = downdrag
DC = dead load of structural components and nonstructural attachments
DW = dead load of wearing surfaces and utilities
EH = horizontal earth pressure load
EL = accumulated locked-in force effects resulting from the construction process, including the secondary forces from post-tensioning
ES = earth surcharge load
EV = vertical pressure from dead load of earth fill

- Transient Loads

BR = vehicular braking force
CE = vehicular centrifugal force
CR = creep
CT = vehicular collision force
CV = vessel collision force
EQ = earthquake
FR = friction
IC = ice load
IM = vehicular dynamic load allowance
LL = vehicular live load
LS = live load surcharge
PL = pedestrian live load
SE = settlement
SH = shrinkage
TG = temperature gradient
TU = uniform temperature
WA = water load and stream pressure
WL = wind on live load
WS = wind load on structure

3.4 LOAD FACTORS AND COMBINATIONS

3.4.1 Load Factors and Load Combinations

The total factored force effect shall be taken as:

$$Q = \sum \eta_i \gamma_i Q_i \qquad (3.4.1\text{-}1)$$

C3.4.1

The background for the load factors specified herein, and the resistance factors specified in other sections of these Specifications is developed in Nowak (*1992*).

where:

η_i = load modifier specified in Article 1.3.2

Q_i = force effects from loads specified herein

γ_i = load factors specified in Tables 1 and 2

Components and connections of a bridge shall satisfy Eq. 1.3.2.1-1 for the applicable combinations of factored extreme force effects as specified at each of the following limit states:

- STRENGTH I—Basic load combination relating to the normal vehicular use of the bridge without wind.

A reduced value of 0.50, applicable to all strength load combinations, specified for *TU*, *CR*, and *SH*, used when calculating force effects other than displacements at the strength limit state, represents an expected reduction of these force effects in conjunction with the inelastic response of the structure. The calculation of displacements for these loads utilizes a factor greater than 1.0 to avoid undersized joints and bearings. The effect and significance of the temperature gradient remains unclear at this writing. Consult Article C3.12.3 for further information.

- STRENGTH II—Load combination relating to the use of the bridge by Owner-specified special design vehicles, evaluation permit vehicles, or both without wind.

The permit vehicle should not be assumed to be the only vehicle on the bridge unless so assured by traffic control. See Article 4.6.2.2.4 regarding other traffic on the bridge simultaneously.

- STRENGTH III—Load combination relating to the bridge exposed to wind velocity exceeding 55 mph.

Vehicles become unstable at higher wind velocities. Therefore, high winds prevent the presence of significant live load on the bridge.

- STRENGTH IV—Load combination relating to very high dead load to live load force effect ratios.

The standard calibration process for the strength limit state consists of trying out various combinations of load and resistance factors on a number of bridges and their components. Combinations that yield a safety index close to the target value of $\beta = 3.5$ are retained for potential application. From these are selected constant load factors γ and corresponding resistance factors ϕ for each type of structural component reflecting its use.

This calibration process had been carried out for a large number of bridges with spans not exceeding 200 ft. These calculations were for completed bridges. For the primary components of large bridges, the ratio of dead and live load force effects is rather high, and could result in a set of resistance factors different from those found acceptable for small- and medium-span bridges. It is believed to be more practical to investigate one additional load case than to require the use of two sets of resistance factors with the load factors provided in Strength Load Combination I, depending on other permanent loads present. Spot checks had been made on a few bridges with up to 600-ft. spans, and it appears that Strength Load Combination IV will govern where the dead load to live load force effect ratio exceeds about 7.0. This load combination can control during investigation of construction stages.

- STRENGTH V—Load combination relating to normal vehicular use of the bridge with wind of 55 mph velocity.

- EXTREME EVENT I—Load combination including earthquake. **The load factor for live load γ_{EQ}, shall be determined on a project-specific basis.**

Past editions of the Standard Specifications used $\gamma_{EQ} = 0.0$. This issue is not resolved. The possibility of partial live load, i.e., $\gamma_{EQ} < 1.0$, with earthquakes should be considered. Application of Turkstra's rule for combining uncorrelated loads indicates that $\gamma_{EQ} = 0.50$ is reasonable for a wide range of values of average daily truck traffic (ADTT).

- EXTREME EVENT II—Load combination relating to ice load, collision by vessels and vehicles, **check floods,** and certain hydraulic events with a reduced live load other than that which is part of the vehicular collision load, *CT*. **The cases of check floods shall not be combined with *CV*, *CT*, or *IC*.**

The following applies to both Extreme Event I and II:

- **The recurrence interval of extreme events is thought to exceed the design life.**

- **Although these limit states include water loads, *WA*, the effects due to *WA* are considerably less significant than the effects on the structure stability due to scour. Therefore, unless specific site conditions dictate otherwise, local pier scour and contraction scour depths should not be combined with *EQ*, *IC*, *CV*, or *CT*. However, the effects due to degradation of the channel should be considered. Alternatively, one-half of the total scour may be considered in combination with *EQ*, *IC*, *CV*, or *CT*.**

- **The joint probability of these events is extremely low, and, therefore, the events are specified to be applied separately. Under these extreme conditions, the structure may undergo considerable inelastic deformation by which locked-in force effects due to *TU*, *TG*, *CR*, *SH*, and *SE* are expected to be relieved.**

The 0.50 live load factor signifies a low probability of the concurrence of the maximum vehicular live load (other than *CT*) and the extreme events.

- SERVICE I—Load combination relating to the normal operational use of the bridge with a 55 mph wind and all loads taken at their nominal values. Also related to deflection control in buried metal structures, tunnel liner plate, and thermoplastic pipe, to control crack width in reinforced concrete structures, and for transverse analysis relating to tension in concrete segmental girders. This load combination should also be used for the investigation of slope stability.

Compression in prestressed concrete components and tension in prestressed bent caps are investigated using this load combination. Service III is used to investigate tensile stresses in prestressed concrete components.

- SERVICE II—Load combination intended to control yielding of steel structures and slip of slip-critical connections due to vehicular live load.

This load combination corresponds to the overload provision for steel structures in past editions of the AASHTO Specifications, and it is applicable only to steel structures. From the point of view of load level, this combination is approximately halfway between that used for Service I and Strength I Limit States.

- SERVICE III—Load combination for longitudinal analysis relating to tension in prestressed concrete superstructures with the objective of crack control and to principal tension in the webs of segmental concrete girders.

The live load specified in these specifications reflects, among other things, current exclusion weight limits mandated by various jurisdictions. Vehicles permitted under these limits have been in service for many years prior to 1993. For longitudinal loading, there is no nationwide physical evidence that these vehicles have caused cracking in existing prestressed concrete components. The statistical significance of the 0.80 factor on live load is that the event is expected to occur about once a year for bridges with two traffic lanes, less often for bridges with more than two traffic lanes, and about once a day for bridges with a single traffic lane. Service I should be used for checking tension related to transverse analysis of concrete segmental girders.

The principal tensile stress check is introduced in order to verify the adequacy of webs of segmental concrete girder bridges for longitudinal shear and torsion.

- SERVICE IV—Load combination relating only to tension in prestressed concrete columns with the objective of crack control.

The 0.70 factor on wind represents an 84 mph wind. This should result in zero tension in prestressed concrete columns for ten-year mean reoccurrence winds. The prestressed concrete columns must still meet strength requirements as set forth in Load Combination Strength III in Article 3.4.1.

It is not recommended that thermal gradient be combined with high wind forces. Superstructure expansion forces are included.

- **FATIGUE I—Fatigue and fracture load combination related to infinite load-induced fatigue life.**

The load factor for the Fatigue I load combination, applied to a single design truck having the axle spacing specified in Article 3.6.1.4.1, reflects load levels found to be representative of the maximum stress range of the truck population for infinite fatigue life design. The factor was chosen on the assumption that the maximum stress range in the random variable spectrum is twice the effective stress range caused by Fatigue II load combination.

- **FATIGUE II—Fatigue and fracture load combination related to finite load-induced fatigue life.**

The load factor for the Fatigue II load combination, applied to a single design truck, reflects a load level found to be representative of the effective stress range of the truck population with respect to a small number of stress range cycles and to their cumulative effects in steel elements, components, and connections for finite fatigue life design.

The load factors for various loads comprising a design load combination shall be taken as specified in Table 1. All relevant subsets of the load combinations shall be investigated. For each load combination, every load that is indicated to be taken into account and that is germane to the component being designed, including all significant effects due to distortion, shall be multiplied by the appropriate load factor and multiple presence factor specified in Article 3.6.1.1.2, if applicable. The products shall be summed as specified in Eq. 1.3.2.1-1 and multiplied by the load modifiers specified in Article 1.3.2.

The factors shall be selected to produce the total extreme factored force effect. For each load combination, both positive and negative extremes shall be investigated.

In load combinations where one force effect decreases another effect, the minimum value shall be applied to the load reducing the force effect. For permanent force effects, the load factor that produces the more critical combination shall be selected from Table 2. Where the permanent load increases the stability or load-carrying capacity of a component or bridge, the minimum value of the load factor for that permanent load shall also be investigated.

This Article reinforces the traditional method of selecting load combinations to obtain realistic extreme effects and is intended to clarify the issue of the variability of permanent loads and their effects. As has always been the case, the Owner or Designer may determine that not all of the loads in a given load combination apply to the situation under investigation.

It is recognized herein that the actual magnitude of permanent loads may also be less than the nominal value. This becomes important where the permanent load reduces the effects of transient loads.

It has been observed that permanent loads are more likely to be greater than the nominal value than to be less than this value.

In the application of permanent loads, force effects for each of the specified six load types should be computed separately. It is unnecessary to assume that one type of load varies by span, length, or component within a bridge. For example, when investigating uplift at a bearing in a continuous beam, it would not be appropriate to use the maximum load factor for permanent loads in spans that produce a negative reaction and the minimum load factor in spans that produce a positive reaction. Consider the investigation of uplift. Uplift, which was treated as a separate load case in past editions of the AASHTO Standard Specifications, now becomes a strength load combination. Where a permanent load produces uplift, that load would be multiplied by the maximum load factor, regardless of the span in which it is located. If another permanent load reduces the uplift, it would be multiplied by the minimum load factor, regardless of the span in which it is located. For example, at Strength I Limit State where the permanent load reaction is positive and live load can cause a negative reaction, the load combination would be $0.9DC + 0.65DW + 1.75(LL + IM)$. If both reactions were negative, the load combination would be $1.25DC + 1.50DW + 1.75(LL + IM)$. For each force effect, both extreme combinations may need to be investigated by applying either the high or the low load factor as appropriate. The algebraic sums of these products are the total force effects for which the bridge and its components should be designed.

The larger of the two values provided for load factor of *TU* shall be used for deformations and the smaller values for all other effects. For simplified analysis of concrete substructures in the strength limit state, a value of 0.50 for γ_{TU} may be used when calculating force effects, but shall be taken in conjunction with the gross moment of inertia in the columns or piers. When a refined analysis is completed for concrete substructures in the strength limit state, a value of 1.0 for γ_{TU} shall be used in conjunction with a partially cracked moment of inertia determined by analysis. For concrete substructures in the strength limit state, the value of 0.50 for γ_{PS}, γ_{CR}, and γ_{SH} may similarly be used when calculating force effects in non-segmental structures, but shall be taken in conjunction with the gross moment of inertia in the columns or piers. For steel substructures, a value of 1.0 for γ_{TU}, γ_{PS}, γ_{CR}, and γ_{SH} shall be used.

The evaluation of overall stability of retained fills, as well as earth slopes with or without a shallow or deep foundation unit should be investigated at the service limit state based on the Service I Load Combination and an appropriate resistance factor as specified in Article 11.5.6 and Article 11.6.2.3.

For structural plate box structures complying with the provisions of Article 12.9, the live load factor for the vehicular live loads *LL* and *IM* shall be taken as 2.0.

PS, *CR*, *SH*, *TU*, and *TG* are superimposed deformations as defined in Article 3.12. Load factors for *TU*, and *TG* are as shown in Table 1. Load factors for *PS*, *CR*, and *SH* are as shown in Table 3. For prestressed members in typical bridge types, secondary prestressing, creep and shrinkage are generally designed for in the service limit state. In concrete segmental structures, *CR* and *SH* are factored by γ_P for *DC* because analysis for time-dependent effects in segmental bridges is nonlinear. Abutments, piers, columns, and bent caps are to be considered as substructure components.

The calculation of displacements for *TU* utilizes a factor greater than 1.0 to avoid undersizing joints, expansion devices, and bearings.

Applying these criteria for the evaluation of the sliding resistance of walls:

- The vertical earth load on the rear of a cantilevered retaining wall would be multiplied by γ_{pmin} (1.00) and the weight of the structure would be multiplied by γ_{pmin} (0.90) because these forces result in an increase in the contact stress (and shear strength) at the base of the wall and foundation.

- The horizontal earth load on a cantilevered retaining wall would be multiplied by γ_{pmax} (1.50) for an active earth pressure distribution because the force results in a more critical sliding force at the base of the wall.

Similarly, the values of γ_{pmax} for structure weight (1.25), vertical earth load (1.35) and horizontal active earth pressure (1.50) would represent the critical load combination for an evaluation of foundation bearing resistance.

Water load and friction are included in all strength load combinations at their respective nominal values.

For creep and shrinkage, the specified nominal values should be used. For friction, settlement, and water loads, both minimum and maximum values need to be investigated to produce extreme load combinations.

The load factor for temperature gradient, γ_{TG}, should be considered on a project-specific basis. In lieu of project-specific information to the contrary, γ_{TG} may be taken as:

- 0.0 at the strength and extreme event limit states,
- 1.0 at the service limit state when live load is not considered, and
- 0.50 at the service limit state when live load is considered.

The load factor for temperature gradient should be determined on the basis of the:

- Type of structure, and
- Limit state being investigated.

Open girder construction and multiple steel box girders have traditionally, but perhaps not necessarily correctly, been designed without consideration of temperature gradient, i.e., $\gamma_{TG} = 0.0$.

The load factor for settlement, γ_{SE}, should be considered on a project-specific basis. In lieu of project-specific information to the contrary, γ_{SE}, may be taken as 1.0. Load combinations which include settlement shall also be applied without settlement.

For segmentally constructed bridges, the following combination shall be investigated at the service limit state:

$$DC + DW + EH + EV + ES + WA + CR + SH + TG + EL + PS \quad (3.4.1\text{-}2)$$

Table 3.4.1-1 Load Combinations and Load Factors.

Load Combination Limit State	*DC* *DD* *DW* *EH* *EV* *ES* *EL* *PS* *CR* *SH*	*LL* *IM* *CE* *BR* *PL* *LS*	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*	Use One of These at a Time			
										EQ	*IC*	*CT*	*CV*
STRENGTH I (unless noted)	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
STRENGTH II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
STRENGTH III	γ_p	—	1.00	1.40	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
STRENGTH IV	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—
STRENGTH V	γ_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
EXTREME EVENT I	γ_p	γEQ	1.00	—	—	1.00	—	—	—	1.00	—	—	—
EXTREME EVENT II	γ_p	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00
SERVICE I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—
SERVICE II	1.00	1.30	1.00	—	—	1.00	1.00/1.20	—	—	—	—	—	—
SERVICE III	1.00	0.80	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—
SERVICE IV	1.00	—	1.00	0.70	—	1.00	1.00/1.20	—	1.0	—	—	—	—
FATIGUE I—*LL*, *IM* & *CE* ONLY	—	**1.50**	—	—	—	—	—	—	—	—	—	—	—
FATIGUE II—*LL*, *IM* & *CE* ONLY	—	**0.75**	—	—	—	—	—	—	—	—	—	—	—

Table 3.4.1-2 Load Factors for Permanent Loads, γ_p.

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor	
		Maximum	Minimum
DC: Component and Attachments		1.25	0.90
DC: Strength IV only		1.50	0.90
DD: Downdrag	Piles, α Tomlinson Method	1.4	0.25
	Piles, λ Method	1.05	0.30
	Drilled shafts, O'Neill and Reese (*1999*) Method	1.25	0.35
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure			
• Active		1.50	0.90
• At-Rest		1.35	0.90
• *AEP* for anchored walls		1.35	N/A
EL: Locked-in Construction Stresses		1.00	1.00
EV: Vertical Earth Pressure			
• Overall Stability		1.00	N/A
• Retaining Walls and Abutments		1.35	1.00
• Rigid Buried Structure		1.30	0.90
• Rigid Frames		1.35	0.90
• Flexible Buried Structures other than Metal Box Culverts		1.95	0.90
• Flexible Metal Box Culverts **and Structural Plate Culverts with Deep Corrugations**		1.50	0.90
ES: Earth Surcharge		1.50	0.75

Table 3.4.1-3 Load Factors for Permanent Loads Due to Superimposed Deformations, γ_p.

Bridge Component	*PS*	*CR, SH*
Superstructures—Segmental Concrete Substructures supporting Segmental Superstructures (see 3.12.4, 3.12.5)	1.0	See γ_P for *DC*, Table 2
Concrete Superstructures—non-segmental	1.0	1.0
Substructures supporting non-segmental Superstructures		
• using I_g	0.5	0.5
• using $I_{effective}$	1.0	1.0
Steel Substructures	1.0	1.0

Where prestressed components are used in conjunction with steel girders, the force effects from the following sources shall be considered as construction loads, *EL*:

- In conjunction with longitudinal prestressing of a precast deck prior to making the deck sections composite with the girders, the friction between the precast deck sections and the steel girders.
- When longitudinal post-tensioning is performed after the deck becomes composite with the girders, the additional forces induced in the steel girders and shear connectors.
- The effects of differential creep and shrinkage of the concrete.
- The Poisson effect.

The most common applications of prestressed concrete in steel girder bridges are transverse post-tensioning of the deck and integral pier caps in which the tendons penetrate the girder webs. When a composite deck is prestressed longitudinally, the shear connectors transfer force to the steel. The effect of shrinkage and long-term creep around the shear connectors should be evaluated to ensure that the composite girder is able to recognize the prestressing over the life of the bridge. The contribution of long-term deformations in closure pours between precast deck panels which have been aged to reduce shrinkage and creep may need evaluation.

The Poisson effect recognizes the bulging of concrete when subjected to prestressing. When used in pier caps, post-tensioning causes a transverse Poisson tensile stress resulting in a longitudinal stress in the steel girders.

The load factor for live load in Extreme Event Load Combination I, γ_{EQ}, shall be determined on a project-specific basis.

A load factor for passive lateral earth pressure is not given in Table 2 because, strictly speaking, passive lateral earth pressure is a resistance and not a load. For discussion of the selection of a passive lateral earth pressure resistance factor see Article 10.5.5.2.2.

Engineering judgment shall be exercised when applying blast loadings and when combining them with other loads.

Blast loads are considered an Extreme Event case of loading. However, not enough information exists at the time of this writing to determine what other loads should be combined with blast loads and the appropriate load factors.

3.4.2 Load Factors for Construction Loads

3.4.2.1 Evaluation at the Strength Limit State

All appropriate strength load combinations in Table 3.4.1-1, modified as specified herein, shall be investigated.

When investigating Strength Load Combinations I, III, and V during construction, load factors for the weight of the structure and appurtenances, *DC* and *DW*, shall not be taken to be less than 1.25.

Unless otherwise specified by the Owner, the load factor for construction loads and for any associated dynamic effects shall not be less than 1.5 in Strength Load Combination I. The load factor for wind in Strength Load Combination III shall not be less than 1.25.

C3.4.2.1

The load factors presented here should not relieve the contractor of responsibility for safety and damage control during construction.

Construction loads are permanent loads and other loads that act on the structure only during construction. Construction loads include the weight of equipment such as deck finishing machines or loads applied to the structure through falsework or other temporary supports. Often the construction loads are not accurately known at design time; however, the magnitude and location of these loads considered in the design should be noted on the contract documents.

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3.4.2.2 Evaluation of Deflection at the Service Limit State

In the absence of special provisions to the contrary, where evaluation of construction deflections are required by the contract documents, Load Combination Service I shall apply. Construction dead loads shall be considered as part of the permanent load and construction transient loads considered part of the live load. The associated permitted deflections shall be included in the contract documents.

3.4.3 Load Factors for Jacking and Post-Tensioning Forces

3.4.3.1 Jacking Forces

Unless otherwise specified by the Owner, the design forces for jacking in service shall not be less than 1.3 times the permanent load reaction at the bearing, adjacent to the point of jacking.

Where the bridge will not be closed to traffic during the jacking operation, the jacking load shall also contain a live load reaction consistent with the maintenance of traffic plan, multiplied by the load factor for live load.

3.4.3.2 Force for Post-Tensioning Anchorage Zones

The design force for post-tensioning anchorage zones shall be taken as 1.2 times the maximum jacking force.

3.5 PERMANENT LOADS

3.5.1 Dead Loads: *DC*, *DW*, and *EV*

Dead load shall include the weight of all components of the structure, appurtenances and utilities attached thereto, earth cover, wearing surface, future overlays, and planned widenings.

In the absence of more precise information, the unit weights, specified in Table 1, may be used for dead loads.

C3.5.1

Table 1 provides traditional unit weights. The unit weight of granular materials depends upon the degree of compaction and water content. The unit weight of concrete is primarily affected by the unit weight of the aggregate, which varies by geographical location and increases with concrete compressive strength. The unit weight of reinforced concrete is generally taken as 0.005 kcf greater than the unit weight of plain concrete. The values provided for wood include the weight of mandatory preservatives. The weight of transit rails, etc., is to be used only for preliminary design.

Table 3.5.1-1 Unit Weights.

Material		Unit Weight (kcf)
Aluminum Alloys		0.175
Bituminous Wearing Surfaces		0.140
Cast Iron		0.450
Cinder Filling		0.060
Compacted Sand, Silt, or Clay		0.120
Concrete	Lightweight	0.110
	Sand-Lightweight	0.120
	Normal Weight with $f'_c \leq 5.0$ ksi	0.145
	Normal Weight with $5.0 < f'_c \leq 15.0$ ksi	$0.140 + 0.001 f'_c$
Loose Sand, Silt, or Gravel		0.100
Soft Clay		0.100
Rolled Gravel, Macadam, or Ballast		0.140
Steel		0.490
Stone Masonry		0.170
Wood	Hard	0.060
	Soft	0.050
Water	Fresh	0.0624
	Salt	0.0640
Item		Weight per Unit Length (klf)
Transit Rails, Ties, and Fastening per Track		0.200

3.5.2 Earth Loads: *EH*, *ES*, and *DD*

Earth pressure, earth surcharge, and downdrag loads shall be as specified in Article 3.11.

3.6 LIVE LOADS

3.6.1 Gravity Loads: *LL* and *PL*

3.6.1.1 Vehicular Live Load

3.6.1.1.1 Number of Design Lanes

Generally, the number of design lanes should be determined by taking the integer part of the ratio $w/12.0$, where w is the clear roadway width in ft. between curbs and/or barriers. Possible future changes in the physical or functional clear roadway width of the bridge should be considered.

In cases where the traffic lanes are less than 12.0 ft. wide, the number of design lanes shall be equal to the number of traffic lanes, and the width of the design lane shall be taken as the width of the traffic lane.

Roadway widths from 20.0 to 24.0 ft. shall have two design lanes, each equal to one-half the roadway width.

C3.6.1.1.1

It is not the intention of this Article to promote bridges with narrow traffic lanes. Wherever possible, bridges should be built to accommodate the standard design lane and appropriate shoulders.

3.6.1.1.2 Multiple Presence of Live Load

The provisions of this Article shall not be applied to the fatigue limit state for which one design truck is used, regardless of the number of design lanes. Where the single-lane approximate distribution factors in Articles 4.6.2.2 and 4.6.2.3 are used, other than the lever rule and statical method, the force effects shall be divided by 1.20.

Unless specified otherwise herein, the extreme live load force effect shall be determined by considering each possible combination of number of loaded lanes multiplied by a corresponding multiple presence factor to account for the probability of simultaneous lane occupation by the full HL93 design live load. In lieu of site specific data, the values in Table 1:

- Shall be used when investigating the effect of one lane loaded,
- May be used when investigating the effect of three or more lanes loaded.

For the purpose of determining the number of lanes when the loading condition includes the pedestrian loads specified in Article 3.6.1.6 combined with one or more lanes of the vehicular live load, the pedestrian loads may be taken to be one loaded lane.

The factors specified in Table 1 shall not be applied in conjunction with approximate load distribution factors specified in Articles 4.6.2.2 and 4.6.2.3, except where the lever rule is used or where special requirements for exterior beams in beam-slab bridges, specified in Article 4.6.2.2.2d, are used.

Table 3.6.1.1.2-1 Multiple Presence Factors *m*.

Number of Loaded Lanes	Multiple Presence Factors m
1	1.20
2	1.00
3	0.85
>3	0.65

C3.6.1.1.2

The multiple presence factors have been included in the approximate equations for distribution factors in Articles 4.6.2.2 and 4.6.2.3, both for single and multiple lanes loaded. The equations are based on evaluation of several combinations of loaded lanes with their appropriate multiple presence factors and are intended to account for the worst case scenario. Where use of the lever rule is specified in Article 4.6.2.2 and 4.6.2.3, the Engineer must determine the number and location of vehicles and lanes, and, therefore, must include the multiple presence. Stated another way, if a sketch is required to determine load distribution, the Engineer is responsible for including multiple presence factors and selecting the worst design case. The factor 1.20 from Table 1 has already been included in the approximate equations and should be removed for the purpose of fatigue investigations.

The entry greater than 1.0 in Table 1 results from statistical calibration of these Specifications on the basis of pairs of vehicles instead of a single vehicle. Therefore, when a single vehicle is on the bridge, it can be heavier than each one of a pair of vehicles and still have the same probability of occurrence.

The consideration of pedestrian loads counting as a "loaded lane" for the purpose of determining a multiple presence factor (m) is based on the assumption that simultaneous occupancy by a dense loading of people combined with a 75-year design live load is remote. For the purpose of this provision, it has been assumed that if a bridge is used as a viewing stand for eight hours each year for a total time of about one month, the appropriate live load to combine with it would have a one-month recurrence interval. This is reasonably approximated by use of the multiple presence factors, even though they are originally developed for vehicular live load.

Thus, if a component supported a sidewalk and one lane, it would be investigated for the vehicular live load alone with $m = 1.20$, and for the pedestrian loads combined with the vehicular live load with $m = 1.0$. If a component supported a sidewalk and two lanes of vehicular live load, it would be investigated for:

- One lane of vehicular live load, $m = 1.20$;
- The greater of the more significant lanes of vehicular live load and the pedestrian loads or two lanes of vehicular live load, $m = 1.0$, applied to the governing case; and
- Two lanes of vehicular live load and the pedestrian loads, $m = 0.85$.

The multiple presence factor of 1.20 for a single lane does not apply to the pedestrian loads. Therefore, the case of the pedestrian loads without the vehicular live load is a subset of the second bulleted item.

The multiple presence factors in Table 1 were developed on the basis of an ADTT of 5,000 trucks in one direction. The force effect resulting from the appropriate number of lanes may be reduced for sites with lower ADTT as follows:

- If $100 \leq ADTT \leq 1{,}000$, 95 percent of the specified force effect may be used; and
- If $ADTT < 100$, 90 percent of the specified force effect may be used.

This adjustment is based on the reduced probability of attaining the design event during a 75-year design life with reduced truck volume.

3.6.1.2 Design Vehicular Live Load

3.6.1.2.1 General

Vehicular live loading on the roadways of bridges or incidental structures, designated HL-93, shall consist of a combination of the:

- Design truck or design tandem, and
- Design lane load.

C3.6.1.2.1

Consideration should be given to site-specific modifications to the design truck, design tandem, and/or the design lane load under the following conditions:

- The legal load of a given jurisdiction is significantly greater than typical;
- The roadway is expected to carry unusually high percentages of truck traffic;
- Flow control, such as a stop sign, traffic signal, or toll booth, causes trucks to collect on certain areas of a bridge or to not be interrupted by light traffic; or
- Special industrial loads are common due to the location of the bridge.

See also discussion in Article C3.6.1.3.1.

The live load model, consisting of either a truck or tandem coincident with a uniformly distributed load, was developed as a notional representation of shear and moment produced by a group of vehicles routinely permitted on highways of various states under "grandfather" exclusions to weight laws. The vehicles considered to be representative of these exclusions were based on a study conducted by the Transportation Research Board (*Cohen 1990*). The load model is called "notional" because it is not intended to represent any particular truck.

Except as modified in Article 3.6.1.3.1, each design lane under consideration shall be occupied by either the design truck or tandem, coincident with the lane load, where applicable. The loads shall be assumed to occupy 10.0 ft. transversely within a design lane.

In the initial development of the notional live load model, no attempt was made to relate to escorted permit loads, illegal overloads, or short duration special permits. The moment and shear effects were subsequently compared to the results of truck weight studies (*Csagoly and Knobel 1981; Nowak 1992*), selected WIM data, and the 1991 OHBDC live load model. These subsequent comparisons showed that the notional load could be scaled by appropriate load factors to be representative of these other load spectra.

The following nomenclature applies to Figures C1 through C6, which show results of live load studies involving two equal continuous spans or simple spans:

M POS 0.4L	=	positive moment at 4/10 point in either span
M NEG 0.4L	=	negative moment at 4/10 point in either span
M SUPPORT	=	moment at interior support
Vab	=	shear adjacent to either exterior support
Vba	=	shear adjacent to interior support
Mss	=	midspan moment in a simply supported span

The "span" is the length of the simple-span or of one of each of the two continuous spans. The comparison is in the form of ratios of the load effects produced in either simple-span or two-span continuous girders. A ratio greater than 1.0 indicates that one or more of the exclusion vehicles produces a larger load effect than the HS20 loading. The figures indicate the degree by which the exclusion loads deviate from the HS loading of designation, e.g., HS25.

Figures C1 and C2 show moment and shear comparisons between the envelope of effects caused by 22 truck configurations chosen to be representative of the exclusion vehicles and the HS20 loading, either the HS20 truck or the lane load, or the interstate load consisting of two 24.0-kip axles 4.0 ft. apart, as used in previous editions of the AASHTO Standard Specifications. The largest and smallest of the 22 configurations can be found in Kulicki and Mertz (*1991*). In the case of negative moment at an interior support, the results presented are based on two identical exclusion vehicles in tandem and separated by at least 50.0 ft.

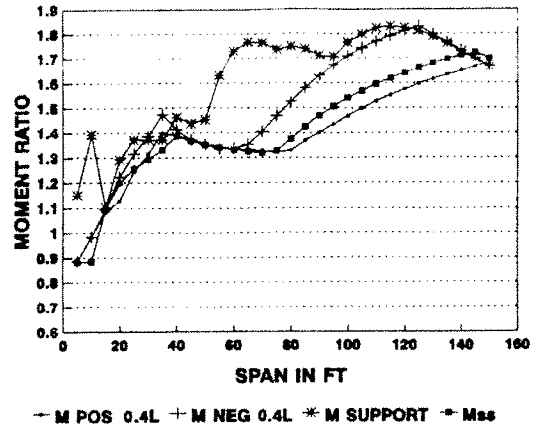


Figure C3.6.1.2.1-1 Moment Ratios: Exclusion Vehicles to HS20 (truck or lane) or Two 24.0-kip Axles at 4.0 ft.

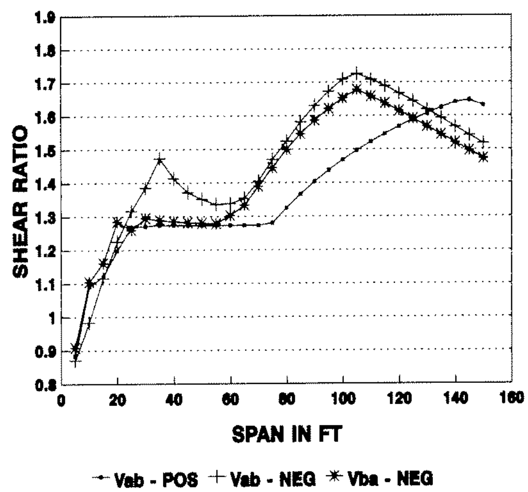


Figure C3.6.1.2.1-2 Shear Ratios: Exclusion Vehicles to HS20 (truck or lane) or Two 24.0-kip Axles at 4.0 ft.

Figures C3 and C4 show comparisons between the force effects produced by a single exclusion truck per lane and the notional load model, except for negative moment, where the tandem exclusion vehicles were used. In the case of negative moment at a support, the provisions of Article 3.6.1.3.1 requiring investigation of 90 percent of the effect of two design trucks, plus 90 percent of the design lane load, has been included in Figures C3 and C5. Compared with Figures C1 and C2, the range of ratios can be seen as more closely grouped:

- Over the span range,
- Both for shear and moment, and
- Both for simple-span and continuous spans.

The implication of close grouping is that the notional load model with a single-load factor has general applicability.

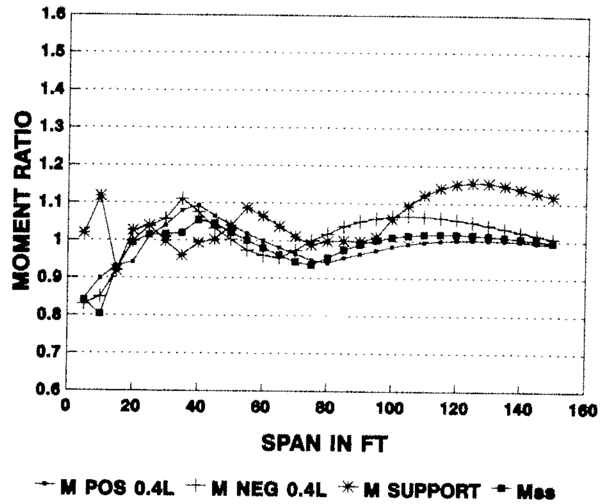


Figure C3.6.1.2.1-3 Moment Ratios: Exclusion Vehicles to Notional Model.

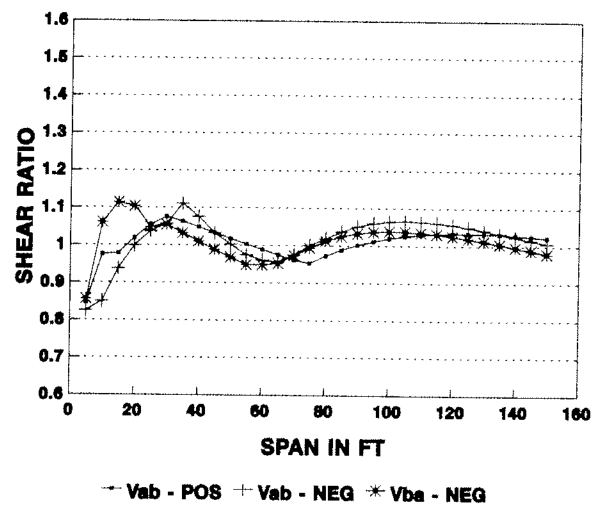


Figure C3.6.1.2.1-4 Shear Ratios: Exclusion Vehicles to Notional Model.

Figures C5 and C6 show the ratios of force effects produced by the notional load model and the greatest of the HS20 truck or lane loading, or Alternate Military Loading.

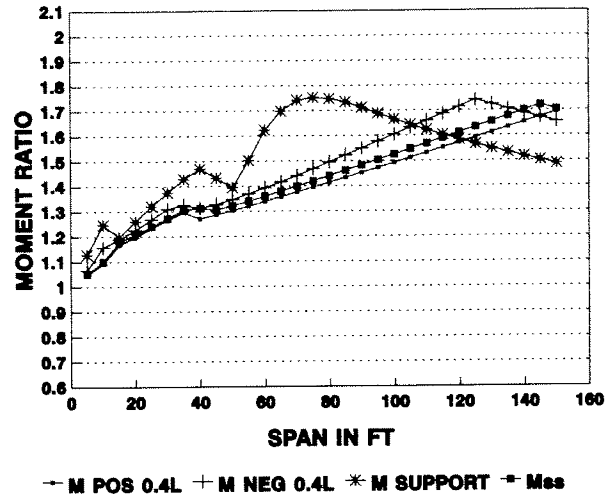


Figure C3.6.1.2.1-5 Moment Ratios: Notional Model to HS20 (truck or lane) or Two 24.0-kip Axles at 4.0 ft.

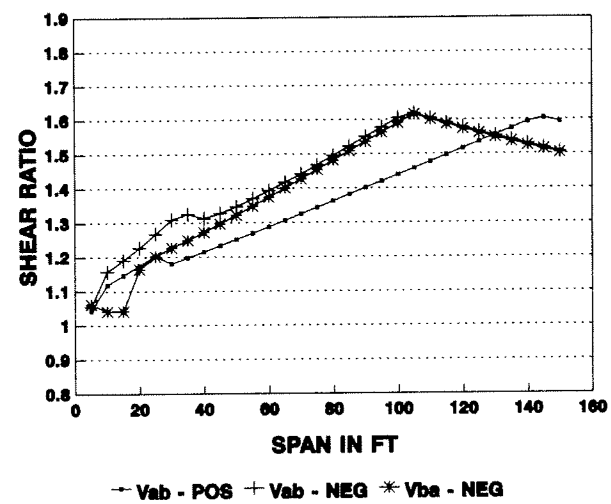


Figure C3.6.1.2.1-6 Shear Ratios: Notional Model to HS20 (truck and lane) or Two 24.0-kip Axles at 4.0 ft.

In reviewing Figures C5 and C6, it should be noted that the total design force effect is also a function of load factor, load modifier, load distribution, and dynamic load allowance.

3.6.1.2.2 Design Truck

The weights and spacings of axles and wheels for the design truck shall be as specified in Figure 1. A dynamic load allowance shall be considered as specified in Article 3.6.2.

Except as specified in Articles 3.6.1.3.1 and 3.6.1.4.1, the spacing between the two 32.0-kip axles shall be varied between 14.0 ft. and 30.0 ft. to produce extreme force effects.

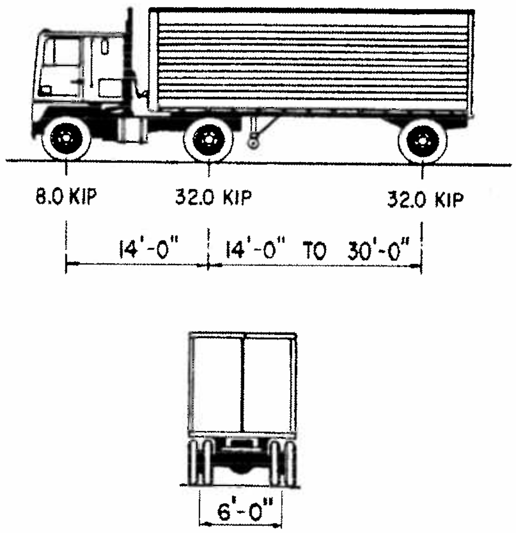


Figure 3.6.1.2.2-1 Characteristics of the Design Truck.

3.6.1.2.3 Design Tandem

The design tandem shall consist of a pair of 25.0-kip axles spaced 4.0 ft. apart. The transverse spacing of wheels shall be taken as 6.0 ft. A dynamic load allowance shall be considered as specified in Article 3.6.2.

3.6.1.2.4 Design Lane Load

The design lane load shall consist of a load of 0.64 klf uniformly distributed in the longitudinal direction. Transversely, the design lane load shall be assumed to be uniformly distributed over a 10.0-ft. width. The force effects from the design lane load shall not be subject to a dynamic load allowance.

3.6.1.2.5 Tire Contact Area

The tire contact area of a wheel consisting of one or two tires shall be assumed to be a single rectangle, whose width is 20.0 in. and whose length is 10.0 in.

The tire pressure shall be assumed to be uniformly distributed over the contact area. The tire pressure shall be assumed to be distributed as follows:

- On continuous surfaces, uniformly over the specified contact area, and
- On interrupted surfaces, uniformly over the actual contact area within the footprint with the pressure increased in the ratio of the specified to actual contact areas.

C3.6.1.2.5

The area load applies only to the design truck and tandem. For other design vehicles, the tire contact area should be determined by the engineer.

As a guideline for other truck loads, the tire area in in.^{2} may be calculated from the following dimensions:

Tire width = $P/0.8$

Tire length = $6.4\gamma(1 + IM/100)$

where:

γ = load factor

IM = dynamic load allowance percent

P = design wheel load (kip)

3.6.1.2.6 Distribution of Wheel Loads Through Earth Fills

Where the depth of fill is less than 2.0 ft., live loads shall be distributed to the top slabs of culverts as specified in Article 4.6.2.10.

In lieu of a more precise analysis, or the use of other acceptable approximate methods of load distribution permitted in Section 12, where the depth of fill is 2.0 ft. or greater, wheel loads may be considered to be uniformly distributed over a rectangular area with sides equal to the dimension of the tire contact area, as specified in Article 3.6.1.2.5, and increased by either 1.15 times the depth of the fill in select granular backfill, or the depth of the fill in all other cases. The provisions of Articles 3.6.1.1.2 and 3.6.1.3 shall apply.

Where such areas from several wheels overlap, the total load shall be uniformly distributed over the area.

For single-span culverts, the effects of live load may be neglected where the depth of fill is more than 8.0 ft. and exceeds the span length; for multiple span culverts, the effects may be neglected where the depth of fill exceeds the distance between faces of end walls.

Where the live load and impact moment in concrete slabs, based on the distribution of the wheel load through earth fills, exceeds the live load and impact moment calculated according to Article 4.6.2.10, the latter moment shall be used.

C3.6.1.2.6

Elastic solutions for pressures produced within an infinite half-space by loads on the ground surface can be found in Poulos and Davis (*1974*), NAVFAC DM-7.1 (*1982*), and soil mechanics textbooks.

This approximation is similar to the 60° rule found in many texts on soil mechanics. The dimensions of the tire contact area are determined at the surface based on the dynamic load allowance of 33 percent at depth = 0. They are projected through the soil as specified. The pressure intensity on the surface is based on the wheel load without dynamic load allowance. A dynamic load allowance is added to the pressure on the projected area. The dynamic load allowance also varies with depth as specified in Article 3.6.2.2. The design lane load is applied where appropriate and multiple presence factors apply.

This provision applies to relieving slabs below grade and to top slabs of box culverts.

Traditionally, the effect of fills less than 2.0 ft. deep on live load has been ignored. Research (*McGrath, et al. 2004*) has shown that in design of box sections allowing distribution of live load through fill in the direction parallel to the span provides a more accurate design model to predict moment, thrust, and shear forces. Provisions in Article 4.6.2.10 provide a means to address the effect of shallow fills.

3.6.1.3 Application of Design Vehicular Live Loads

3.6.1.3.1 General

Unless otherwise specified, the extreme force effect shall be taken as the larger of the following:

- The effect of the design tandem combined with the effect of the design lane load, or
- The effect of one design truck with the variable axle spacing specified in Article 3.6.1.2.2, combined with the effect of the design lane load, and

C3.6.1.3.1

The effects of an axle sequence and the lane load are superposed in order to obtain extreme values. This is a deviation from the traditional AASHTO approach, in which either the truck or the lane load, with an additional concentrated load, provided for extreme effects.

The lane load is not interrupted to provide space for the axle sequences of the design tandem or the design truck; interruption is needed only for patch loading patterns to produce extreme force effects.

- For both negative moment between points of contraflexure under a uniform load on all spans, and reaction at interior piers only, 90 percent of the effect of two design trucks spaced a minimum of 50.0 ft. between the lead axle of one truck and the rear axle of the other truck, combined with 90 percent of the effect of the design lane load. The distance between the 32.0-kip axles of each truck shall be taken as 14.0 ft.

Axles that do not contribute to the extreme force effect under consideration shall be neglected.

Both the design lanes and the 10.0-ft. loaded width in each lane shall be positioned to produce extreme force effects. The design truck or tandem shall be positioned transversely such that the center of any wheel load is not closer than:

- For the design of the deck overhang—1.0 ft. from the face of the curb or railing, and
- For the design of all other components—2.0 ft. from the edge of the design lane.

Unless otherwise specified, the lengths of design lanes, or parts thereof, that contribute to the extreme force effect under consideration, shall be loaded with the design lane load.

The notional design loads were based on the information described in Article C3.6.1.2.1, which contained data on "low boy" type vehicles weighing up to about 110 kip. Where multiple lanes of heavier versions of this type of vehicle are considered probable, consideration should be given to investigating negative moment and reactions at interior supports for pairs of the design tandem spaced from 26.0 ft. to 40.0 ft. apart, combined with the design lane load specified in Article 3.6.1.2.4. One hundred percent of the combined effect of the design tandems and the design lane load should be used. This is consistent with Article 3.6.1.2.1 and should not be considered a replacement for the Strength II Load Combination.

Only those areas or parts of areas that contribute to the same extreme being sought should be loaded. The loaded length should be determined by the points where the influence surface meets the centerline of the design lane.

Where a sidewalk is not separated from the roadway by a crashworthy traffic barrier, consideration should be given to the possibility that vehicles can mount the sidewalk.

3.6.1.3.2 Loading for Optional Live Load Deflection Evaluation

If the Owner invokes the optional live load deflection criteria specified in Article 2.5.2.6.2, the deflection should be taken as the larger of:

- That resulting from the design truck alone, or
- That resulting from 25 percent of the design truck taken together with the design lane load.

C3.6.1.3.2

As indicated in C2.5.2.6.1, live load deflection is a service issue, not a strength issue. Experience with bridges designed under previous editions of the AASHTO Standard Specifications indicated no adverse effects of live load deflection per se. Therefore, there appears to be little reason to require that the past criteria be compared to a deflection based upon the heavier live load required by these Specifications.

The provisions of this Article are intended to produce apparent live load deflections similar to those used in the past. The current design truck is identical to the HS20 truck of past Standard Specifications. For the span lengths where the design lane load controls, the design lane load together with 25 percent of the design truck, i.e., three concentrated loads totaling 18.0 kip, is similar to the past lane load with its single concentrated load of 18.0 kip.

3.6.1.3.3 Design Loads for Decks, Deck Systems, and the Top Slabs of Box Culverts

The provisions of this Article shall not apply to decks designed under the provisions of Article 9.7.2, "Empirical Design."

Where the approximate strip method is used to analyze decks and top slabs of culverts, force effects shall be determined on the following basis:

- Where the slab spans primarily in the transverse direction, only the axles of the design truck of Article 3.6.1.2.2 or design tandem of Article 3.6.1.2.3 shall be applied to the deck slab or the top slab of box culverts.

- Where the slab spans primarily in the longitudinal direction:

  - For top slabs of box culverts of all spans and for all other cases, including slab-type bridges where the span does not exceed 15.0 ft., only the axle loads of the design truck or design tandem of Articles 3.6.1.2.2 and 3.6.1.2.3, respectively, shall be applied.

  - For all other cases, including slab-type bridges (excluding top slabs of box culverts) where the span exceeds 15.0 ft., all of the load specified in Article 3.6.1.2 shall be applied.

Where the refined methods are used to analyze decks, force effects shall be determined on the following basis:

- Where the slab spans primarily in the transverse direction, only the axles of the design truck of Article 3.6.1.2.2 or design tandem of Article 3.6.1.2.3 shall be applied to the deck slab.

- Where the slab spans primarily in the longitudinal direction (including slab-type bridges), all of the loads specified in Article 3.6.1.2 shall be applied.

Wheel loads shall be assumed to be equal within an axle unit, and amplification of the wheel loads due to centrifugal and braking forces need not be considered for the design of decks.

C3.6.1.3.3

This Article clarifies the selection of wheel loads to be used in the design of bridge decks, slab bridges, and top slabs of box culverts.

The design load is always an axle load; single wheel loads should not be considered.

The design truck and tandem without the lane load and with a multiple presence factor of 1.2 results in factored force effects that are similar to the factored force effects using earlier specifications for typical span ranges of box culverts.

Individual owners may choose to develop other axle weights and configurations to capture the load effects of the actual loads in their jurisdiction based upon local legal-load and permitting policies. Triple axle configurations of single unit vehicles have been observed to have load effects in excess of the HL-93 tandem axle load.

It is theoretically possible that an extreme force effect could result from a 32.0-kip axle in one lane and a 50.0-kip tandem in a second lane, but such sophistication is not warranted in practical design.

3.6.1.3.4 Deck Overhang Load

For the design of deck overhangs with a cantilever, not exceeding 6.0 ft. from the centerline of the exterior girder to the face of a structurally continuous concrete railing, the outside row of wheel loads may be replaced with a uniformly distributed line load of 1.0 klf intensity, located 1.0 ft. from the face of the railing.

Horizontal loads on the overhang resulting from vehicle collision with barriers shall be in accordance with the provisions of Section 13.

C3.6.1.3.4

Structurally continuous barriers have been observed to be effective in distributing wheel loads in the overhang. Implicit in this provision is the assumption that the 25.0-kip half weight of a design tandem is distributed over a longitudinal length of 25.0 ft., and that there is a cross beam or other appropriate component at the end of the bridge supporting the barrier which is designed for the half tandem weight. This provision does not apply if the barrier is not structurally continuous.

3.6.1.4 Fatigue Load

3.6.1.4.1 Magnitude and Configuration

The fatigue load shall be one design truck or axles thereof specified in Article 3.6.1.2.2, but with a constant spacing of 30.0 ft. between the 32.0-kip axles.

The dynamic load allowance specified in Article 3.6.2 shall be applied to the fatigue load.

3.6.1.4.2 Frequency

The frequency of the fatigue load shall be taken as the single-lane average daily truck traffic ($ADTT_{SL}$). This frequency shall be applied to all components of the bridge, even to those located under lanes that carry a lesser number of trucks.

In the absence of better information, the single-lane average daily truck traffic shall be taken as:

$$ADTT_{SL} = p \times ADTT \qquad (3.6.1.4.2\text{-}1)$$

where:

$ADTT$ = the number of trucks per day in one direction averaged over the design life

$ADTT_{SL}$ = the number of trucks per day in a single-lane averaged over the design life

p = **fraction of traffic in a single lane,** taken as specified in Table 1

C3.6.1.4.2

Since the fatigue and fracture limit state is defined in terms of accumulated stress-range cycles, specification of load alone is not adequate. Load should be specified along with the frequency of load occurrence.

For the purposes of this Article, a truck is defined as any vehicle with more than either two axles or four wheels.

The single-lane $ADTT$ is that for the traffic lane in which the majority of the truck traffic crosses the bridge. On a typical bridge with no nearby entrance/exit ramps, the shoulder lane carries most of the truck traffic. **The frequency of the fatigue load for a single lane is assumed to apply to all lanes since future traffic patterns on the bridge are uncertain.**

Consultation with traffic engineers regarding any directionality of truck traffic may lead to the conclusion that one direction carries more than one-half of the bidirectional *ADTT*. If such data is not available from traffic engineers, designing for 55 percent of the bidirectional *ADTT* is suggested.

The value of $ADTT_{SL}$ is best determined in consultation with traffic engineers. However, traffic growth data is usually not predicted for the design life of the bridge, taken as 75 years in these specifications unless specified otherwise by the owner. Techniques exist to extrapolate available data such as curve fitting growth rate vs. time and using extreme value distributions, but some judgment is required. Research has shown that the average daily traffic (ADT), including all vehicles, i.e., cars and trucks, is physically limited to about 20,000 vehicles per lane per day under normal conditions. This limiting value of traffic should be considered when estimating the $ADTT$. The $ADTT$ can be determined by multiplying the ADT by the fraction of trucks in the traffic. In lieu of site-specific fraction of truck traffic data, the values of Table C1 may be applied for routine bridges.

Table 3.6.1.4.2-1 Fraction of Truck Traffic in a Single Lane, *p*.

Number of Lanes Available to Trucks	*p*
1	1.00
2	0.85
3 or more	0.80

Table C3.6.1.4.2-1 Fraction of Trucks in Traffic.

Class of Highway	Fraction of Trucks in Traffic
Rural Interstate	0.20
Urban Interstate	0.15
Other Rural	0.15
Other Urban	0.10

3.6.1.4.3 Load Distribution for Fatigue

3.6.1.4.3a Refined Methods

Where the bridge is analyzed by any refined method, as specified in Article 4.6.3, a single design truck shall be positioned transversely and longitudinally to maximize stress range at the detail under consideration, regardless of the position of traffic or design lanes on the deck.

C3.6.1.4.3a

If it were assured that the traffic lanes would remain as they are indicated at the opening of the bridge throughout its entire service life, it would be more appropriate to place the truck at the center of the traffic lane that produces maximum stress range in the detail under consideration. But because future traffic patterns on the bridge are uncertain and in the interest of minimizing the number of calculations required of the Designer, the position of the truck is made independent of the location of both the traffic lanes and the design lanes.

3.6.1.4.3b Approximate Methods

Where the bridge is analyzed by approximate load distribution, as specified in Article 4.6.2, the distribution factor for one traffic lane shall be used.

3.6.1.5 Rail Transit Load

Where a bridge also carries rail-transit vehicles, the Owner shall specify the transit load characteristics and the expected interaction between transit and highway traffic.

C3.6.1.5

If rail transit is designed to occupy an exclusive lane, transit loads should be included in the design, but the bridge should not have less strength than if it had been designed as a highway bridge of the same width.

If the rail transit is supposed to mix with regular highway traffic, the Owner should specify or approve an appropriate combination of transit and highway loads for the design.

Transit load characteristics may include:

- Loads,
- Load distribution,
- Load frequency,
- Dynamic allowance, and
- Dimensional requirements.

3.6.1.6 Pedestrian Loads

A pedestrian load of 0.075 ksf shall be applied to all sidewalks wider than 2.0 ft. and considered simultaneously with the vehicular design live load in the vehicle lane. Where vehicles can mount the sidewalk, sidewalk pedestrian load shall not be considered concurrently. If a sidewalk may be removed in the future, the vehicular live loads shall be applied at 1 ft. from edge-of-deck for design of the overhang, and 2 ft. from edge-of-deck for design of all other components.

C3.6.1.6

See the provisions of Article C3.6.1.1.2 for applying the pedestrian loads in combination with the vehicular live load.

Where sidewalks, pedestrian, and/or bicycle bridges are intended to be used by maintenance and/or other incidental vehicles, these loads shall be considered in the design. The dynamic load allowance need not be considered for these vehicles.

Where vehicles can mount the sidewalk, sidewalk pedestrian load shall not be considered concurrently.

Snow removal and other maintenance vehicles sometimes have access to pedestrian bridges. The slow speed of such vehicles justifies the omission of dynamic effects.

3.6.1.7 Loads on Railings

Loads on railings shall be taken as specified in Section 13.

3.6.2 Dynamic Load Allowance: *IM*

3.6.2.1 General

Unless otherwise permitted in Articles 3.6.2.2 and 3.6.2.3, the static effects of the design truck or tandem, other than centrifugal and braking forces, shall be increased by the percentage specified in Table 1 for dynamic load allowance.

The factor to be applied to the static load shall be taken as: $(1 + IM/100)$.

The dynamic load allowance shall not be applied to pedestrian loads or to the design lane load.

Table 3.6.2.1-1 Dynamic Load Allowance, *IM*.

Component	IM
Deck Joints—All Limit States	75%
All Other Components	
• Fatigue and Fracture Limit State	15%
• All Other Limit States	33%

The application of dynamic load allowance for buried components, covered in Section 12, shall be as specified in Article 3.6.2.2.

Dynamic load allowance need not be applied to:

- Retaining walls not subject to vertical reactions from the superstructure, and
- Foundation components that are entirely below ground level.

C3.6.2.1

Page (*1976*) contains the basis for some of these provisions.

The dynamic load allowance (*IM*) in Table 1 is an increment to be applied to the static wheel load to account for wheel load impact from moving vehicles.

Dynamic effects due to moving vehicles may be attributed to two sources:

- Hammering effect is the dynamic response of the wheel assembly to riding surface discontinuities, such as deck joints, cracks, potholes, and delaminations, and
- Dynamic response of the bridge as a whole to passing vehicles, which may be due to long undulations in the roadway pavement, such as those caused by settlement of fill, or to resonant excitation as a result of similar frequencies of vibration between bridge and vehicle.

Field tests indicate that in the majority of highway bridges, the dynamic component of the response does not exceed 25 percent of the static response to vehicles. This is the basis for dynamic load allowance with the exception of deck joints. However, the specified live load combination of the design truck and lane load, represents a group of exclusion vehicles that are at least 4/3 of those caused by the design truck alone on short- and medium-span bridges. The specified value of 33 percent in Table 1 is the product of 4/3 and the basic 25 percent.

The dynamic load allowance may be reduced for components, other than joints, if justified by sufficient evidence, in accordance with the provisions of Article 4.7.2.1.

Generally speaking, the dynamic amplification of trucks follows the following general trends:

- As the weight of the vehicle goes up, the apparent amplification goes down.
- Multiple vehicles produce a lower dynamic amplification than a single vehicle.
- More axles result in a lower dynamic amplification.

For heavy permit vehicles which have many axles compared to the design truck, a reduction in the dynamic load allowance may be warranted. A study of dynamic effects presented in a report by the Calibration Task Group (*Nowak 1992*) contains details regarding the relationship between dynamic load allowance and vehicle configuration.

This Article recognizes the damping effect of soil when in contact with some buried structural components, such as footings. To qualify for relief from impact, the entire component must be buried. For the purpose of this Article, a retaining type component is considered to be buried to the top of the fill.

3.6.2.2 Buried Components

The dynamic load allowance for culverts and other buried structures covered by Section 12, in percent, shall be taken as:

$$IM = 33(1.0 - 0.125D_E) \geq 0\% \qquad (3.6.2.2\text{-}1)$$

where:

D_E = the minimum depth of earth cover above the structure (ft.)

3.6.2.3 Wood Components

Dynamic load allowance need not be applied to wood components.

C3.6.2.3

Wood structures are known to experience reduced dynamic wheel load effects due to internal friction between the components and the damping characteristics of wood. Additionally, wood is stronger for short duration loads, as compared to longer duration loads. This increase in strength is greater than the increase in force effects resulting from the dynamic load allowance.

3.6.3 Centrifugal Forces: *CE*

For the purpose of computing the radial force or the overturning effect on wheel loads, the centrifugal effect on live load shall be taken as the product of the axle weights of the design truck or tandem and the factor C, taken as:

$$C = f\frac{v^2}{gR} \tag{3.6.3-1}$$

where:

v = highway design speed (ft./sec.)

f = 4/3 for load combinations other than fatigue and 1.0 for fatigue

g = gravitational acceleration: 32.2 (ft./sec.^{2})

R = radius of curvature of traffic lane (ft.)

Highway design speed shall not be taken to be less than the value specified in the current edition of the AASHTO publication, *A Policy of Geometric Design of Highways and Streets*.

The multiple presence factors specified in Article 3.6.1.1.2 shall apply.

Centrifugal forces shall be applied horizontally at a distance 6.0 ft. above the roadway surface. A load path to carry the radial force to the substructure shall be provided.

The effect of superelevation in reducing the overturning effect of centrifugal force on vertical wheel loads may be considered.

C3.6.3

Centrifugal force is not required to be applied to the design lane load, as the spacing of vehicles at high speed is assumed to be large, resulting in a low density of vehicles following and/or preceding the design truck. For all other consideration of live load other than for fatigue, the design lane load is still considered even though the centrifugal effect is not applied to it.

The specified live load combination of the design truck and lane load, however, represents a group of exclusion vehicles that produce force effects of at least 4/3 of those caused by the design truck alone on short- and medium-span bridges. This ratio is indicated in Eq. 1 for the service and strength limit states. For the fatigue and fracture limit state, the factor 1.0 is consistent with cumulative damage analysis. The provision is not technically perfect, yet it reasonably models the representative exclusion vehicle traveling at design speed with large headways to other vehicles. The approximation attributed to this convenient representation is acceptable in the framework of the uncertainty of centrifugal force from random traffic patterns.

1.0 ft./sec. = 0.682 mph.

Centrifugal force also causes an overturning effect on the wheel loads because the radial force is applied 6.0 ft. above the top of the deck. Thus, centrifugal force tends to cause an increase in the vertical wheel loads toward the outside of the bridge and an unloading of the wheel loads toward the inside of the bridge. Superelevation helps to balance the overturning effect due to the centrifugal force and this beneficial effect may be considered. The effects due to vehicle cases with centrifugal force effects included should be compared to the effects due to vehicle cases with no centrifugal force, and the worst case selected.

3.6.4 Braking Force: *BR*

The braking force shall be taken as the greater of:

- 25 percent of the axle weights of the design truck or design tandem or,
- 5 percent of the design truck plus lane load or 5 percent of the design tandem plus lane load

C3.6.4

Based on energy principles, and assuming uniform deceleration, the braking force determined as a fraction of vehicle weight is:

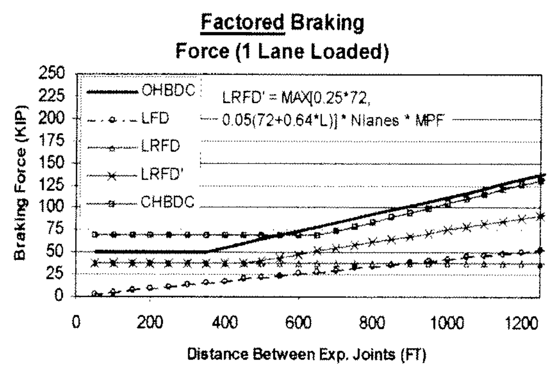
$$b = \frac{v^2}{2ga} \tag{C3.6.4-1}$$

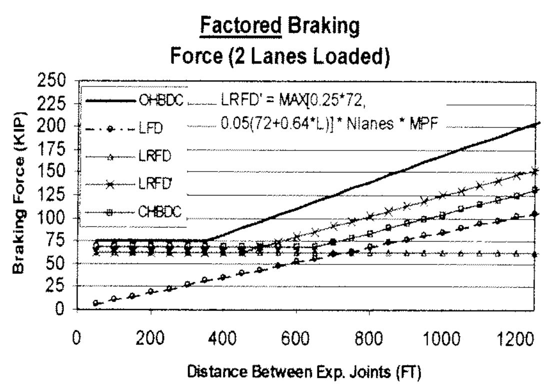
This braking force shall be placed in all design lanes which are considered to be loaded in accordance with Article 3.6.1.1.1 and which are carrying traffic headed in the same direction. These forces shall be assumed to act horizontally at a distance of 6.0 ft. above the roadway surface in either longitudinal direction to cause extreme force effects. All design lanes shall be simultaneously loaded for bridges likely to become one-directional in the future.

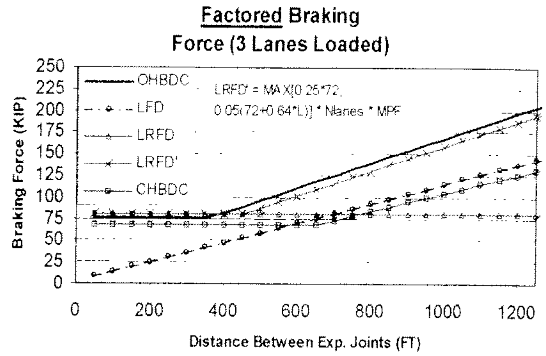
The multiple presence factors specified in Article 3.6.1.1.2 shall apply.

where a is the length of uniform deceleration and b is the fraction. Calculations using a braking length of 400 ft. and a speed of 55 mph yield $b = 0.25$ for a horizontal force that will act for a period of about 10 seconds. The factor b applies to all lanes in one direction because all vehicles may have reacted within this time frame.

For short- and medium-span bridges, the specified braking force can be significantly larger than was required in the Standard Specifications. The braking force specified in the Standard Specifications dates back to at least the early 1940's without any significant changes to address the improved braking capacity of modern trucks. A review of other bridge design codes in Canada and Europe showed that the braking force required by the Standard Specification is much lower than that specified in other design codes for most typical bridges. One such comparison is shown in Figure C1.







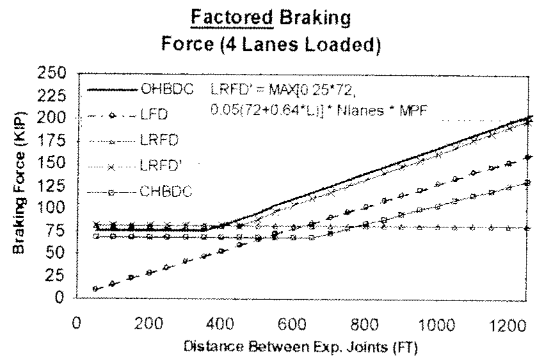


Figure C3.6.4-1 Comparison of Braking Force Models.

where:

OHBDC = factored braking force as specified in the 3rd edition of the *Ontario Highway Bridge Design Code*

LFD = factored braking force as specified in the AASHTO Standard Specifications (Load Factor)

LRFD = factored braking force as specified in previous versions of the LRFD Specifications (up to 2001 Interim edition)

LRFD′ = factored braking force as specified in Article 3.6.4

CHBDC = factored braking force as specified in the *Canadian Highway Bridge Design Code*

The sloping portion of the curves represents the braking force that includes a portion of the lane load. This represents the possibility of having multiple lanes of vehicles contributing to the same braking event on a long bridge. Although the probability of such an event is likely to be small, the inclusion of a portion of the lane load gives such an event consideration for bridges with heavy truck traffic and is consistent with other design codes.

Because the LRFD braking force is significantly higher than that required in the Standard Specifications, this issue becomes important in rehabilitation projects designed under previous versions of the design code. In cases where substructures are found to be inadequate to resist the increased longitudinal forces, consideration should be given to design and detailing strategies which distribute the braking force to additional substructure units during a braking event.

3.6.5 Vehicular Collision Force: *CT*

3.6.5.1 Protection of Structures

The provisions of Article 3.6.5.2 need not be considered for structures which are protected by:

- An embankment;
- A structurally independent, crashworthy ground-mounted 54.0-in. high barrier, located within 10.0 ft. from the component being protected; or
- A 42.0-in. high barrier located at more than 10.0 ft. from the component being protected.

In order to qualify for this exemption, such barrier shall be structurally and geometrically capable of surviving the crash test for Test Level 5, as specified in Section 13.

C3.6.5.1

For the purpose of this Article, a barrier may be considered structurally independent if it does not transmit loads to the bridge.

Full-scale crash tests have shown that some vehicles have a greater tendency to lean over or partially cross over a 42.0-in. high barrier than a 54.0-in. high barrier. This behavior would allow a significant collision of the vehicle with the component being protected if the component is located within a few ft. of the barrier. If the component is more than about 10.0 ft. behind the barrier, the difference between the two barrier heights is no longer important.

3.6.5.2 Vehicle and Railway Collision with Structures

Unless protected as specified in Article 3.6.5.1, abutments and piers located within a distance of 30.0 ft. to the edge of roadway, or within a distance of 50.0 ft. to the centerline of arailway track, shall be designed for an equivalent static force of 400 kip, which is assumed to act in any direction in a horizontal plane, at a distance of 4.0 ft. above ground.

The provisions of Article 2.3.2.2.1 shall apply.

C3.6.5.2

It is not the intent of this provision to encourage unprotected piers and abutments within the setbacks indicated, but rather to supply some guidance for structural design when it is deemed totally impractical to meet the requirements of Article 3.6.5.1.

The equivalent static force of 400 kip is based on the information from full-scale crash tests of barriers for redirecting 80.0-kip tractor trailers and from analysis of other truck collisions. The 400-kip train collision load is based on recent, physically unverified, analytical work (*Hirsch 1989*). For individual column shafts, the 400-kip load should be considered a point load. For wall piers, the load may be considered to be a point load or may be distributed over an area deemed suitable for the size of the structure and the anticipated impacting vehicle, but not greater than 5.0 ft. wide by 2.0 ft. high. These dimensions were determined by considering the size of a truck frame.

3.6.5.3 Vehicle Collision with Barriers

The provisions of Section 13 shall apply.

3.7 WATER LOADS: *WA*

3.7.1 Static Pressure

Static pressure of water shall be assumed to act perpendicular to the surface that is retaining the water. Pressure shall be calculated as the product of height of water above the point of consideration and the specific weight of water.

Design water levels for various limit states shall be as specified and/or approved by the Owner.

3.7.2 Buoyancy

Buoyancy shall be considered to be an uplift force, taken as the sum of the vertical components of static pressures, as specified in Article 3.7.1, acting on all components below design water level.

C3.7.2

For substructures with cavities in which the presence or absence of water cannot be ascertained, the condition producing the least favorable force effect should be chosen.

3.7.3 Stream Pressure

3.7.3.1 Longitudinal

The pressure of flowing water acting in the longitudinal direction of substructures shall be taken as:

$$p = \frac{C_D V^2}{1{,}000} \qquad (3.7.3.1\text{-}1)$$

where:

p = pressure of flowing water (ksf)

C_D = drag coefficient for piers as specified in Table 1

V = design velocity of water for the design flood in strength and service limit states and for the check flood in the extreme event limit state (ft./sec.)

Table 3.7.3.1-1 Drag Coefficient.

Type	C_D
semicircular-nosed pier	0.7
square-ended pier	1.4
debris lodged against the pier	1.4
wedged-nosed pier with nose angle 90° or less	0.8

C3.7.3.1

For the purpose of this Article, the longitudinal direction refers to the major axis of a substructure unit.

The theoretically correct expression for Eq. 1 is:

$$p = C_D \frac{w}{2g} V^2 \qquad (C3.7.3.1\text{-}1)$$

where:

w = specific weight of water (kcf)

V = velocity of water (ft./sec.)

g = gravitational acceleration constant—32.2 (ft./sec.^{2})

As a convenience, Eq. 1 recognizes that $w/2g \sim 1/1{,}000$, but the dimensional consistency is lost in the simplification.

The longitudinal drag force shall be taken as the product of longitudinal stream pressure and the projected surface exposed thereto.

The drag coefficient, C_D, and the lateral drag coefficient, C_L, given in Tables 1 and 3.7.3.2-1, were adopted from the Ontario Highway Bridge Design Code (*1991*). The more favorable drag coefficients measured by some researchers for wedge-type pier nose angles of less than 90° are not given here because such pier noses are more prone to catching debris.

Floating logs, roots, and other debris may accumulate at piers and, by blocking parts of the waterway, increase stream pressure load on the pier. Such accumulation is a function of the availability of such debris and level of maintenance efforts by which it is removed. It may be accounted for by the judicious increase in both the exposed surface and the velocity of water.

The draft New Zealand Highway Bridge Design Specification contains the following provision, which may be used as guidance in the absence of site-specific criteria:

> Where a significant amount of driftwood is carried, water pressure shall also be allowed for on a driftwood raft lodged against the pier. The size of the raft is a matter of judgment, but as a guide, Dimension A in Figure C1 should be half the water depth, but not greater than 10.0 ft. Dimension B should be half the sum of adjacent span lengths, but no greater than 45.0 ft. Pressure shall be calculated using Eq. 1, with C_D = 0.5 . (Distances have been changed from SI.)

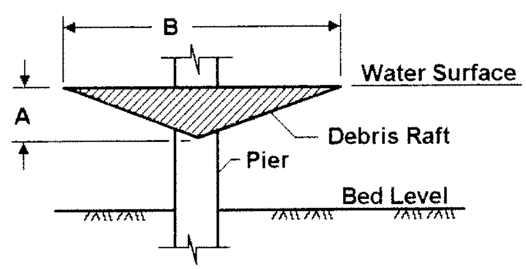


Figure C3.7.3.1-1 Debris Raft for Pier Design.

3.7.3.2 Lateral

The lateral, uniformly distributed pressure on a substructure due to water flowing at an angle, θ, to the longitudinal axis of the pier shall be taken as:

$$p = \frac{C_L V^2}{1000} \tag{3.7.3.2-1}$$

where:

p = lateral pressure (ksf)

C_L = lateral drag coefficient specified in Table 1

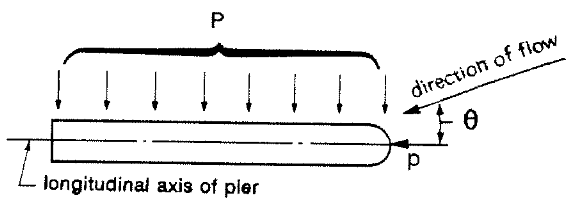


Figure 3.7.3.2-1 Plan View of Pier Showing Stream Flow Pressure.

Table 3.7.3.2-1 Lateral Drag Coefficient.

Angle, θ, between direction of flow and longitudinal axis of the pier	C_L
0°	0.0
5°	0.5
10°	0.7
20°	0.9
≥30°	1.0

The lateral drag force shall be taken as the product of the lateral stream pressure and the surface exposed thereto.

C3.7.3.2

The discussion of Eq. 3.7.3.1-1 also applies to Eq. 1.

3.7.4 Wave Load

Wave action on bridge structures shall be considered for exposed structures where the development of significant wave forces may occur.

C3.7.4

Loads due to wave action on bridge structures shall be determined using accepted engineering practice methods. Site-specific conditions should be considered. The latest edition of the *Shore Protection Manual*, published by the Coastal Engineering Research Center, Department of the Army, is recommended for the computation of wave forces.

3.7.5 Change in Foundations Due to Limit State for Scour

The provisions of Article 2.6.4.4 shall apply.

The consequences of changes in foundation conditions resulting from the design flood for scour shall be considered at strength and service limit states. The consequences of changes in foundation conditions due to scour resulting from the check flood for bridge scour and from hurricanes shall be considered at the extreme event limit states.

C3.7.5

Statistically speaking, scour is the most common reason for the failure of highway bridges in the United States.

Provisions concerning the effects of scour are given in Section 2. Scour per se is not a force effect, but by changing the conditions of the substructure it may significantly alter the consequences of force effects acting on structures.

3.8 WIND LOAD: *WL* AND *WS*

3.8.1 Horizontal Wind Pressure

3.8.1.1 General

Pressures specified herein shall be assumed to be caused by a base design wind velocity, V_B, of 100 mph.

Wind load shall be assumed to be uniformly distributed on the area exposed to the wind. The exposed area shall be the sum of areas of all components, including floor system and railing, as seen in elevation taken perpendicular to the assumed wind direction. This direction shall be varied to determine the extreme force effect in the structure or in its components. Areas that do not contribute to the extreme force effect under consideration may be neglected in the analysis.

For bridges or parts of bridges more than 30.0 ft. above low ground or water level, the design wind velocity, V_{DZ}, should be adjusted according to:

$$V_{DZ} = 2.5V_0\left(\frac{V_{30}}{V_B}\right)\ln\left(\frac{Z}{Z_0}\right) \quad (3.8.1.1\text{-}1)$$

where:

V_{DZ} = design wind velocity at design elevation, Z (mph)

V_{30} = wind velocity at 30.0 ft. above low ground or above design water level (mph)

V_B = base wind velocity of 100 mph at 30.0 ft. height, yielding design pressures specified in Articles 3.8.1.2 and 3.8.2

Z = height of structure at which wind loads are being calculated as measured from low ground, or from water level, > 30.0 ft.

V_0 = friction velocity, a meteorological wind characteristic taken, as specified in Table 1, for various upwind surface characteristics (mph)

C3.8.1.1

Base design wind velocity varies significantly due to local conditions. For small and/or low structures, wind usually does not govern. For large and/or tall bridges, however, the local conditions should be investigated.

Pressures on windward and leeward sides are to be taken simultaneously in the assumed direction of wind.

Typically, a bridge structure should be examined separately under wind pressures from two or more different directions in order to ascertain those windward, leeward, and side pressures producing the most critical loads on the structure.

Eq. 1 is based on boundary layer theory combined with empirical observations and represents the most recent approach to defining wind speeds for various conditions as used in meteorology. In the past, an exponential equation was sometimes used to relate wind speed to heights above 30.0 ft. This formulation was based solely on empirical observations and had no theoretical basis.

$$V_{DZ} = CV_{30}\left(\frac{Z}{30}\right)^{\alpha} \quad (C3.8.1.1\text{-}1)$$

The purpose of the term C and exponent α was to adjust the equation for various upstream surface conditions, similar to the use of Table 1. Further information can be found in Liu (*1991*) and Simiu (*1973, 1976*).

The following descriptions for the terms "open country," "suburban," and "city" in Table 1 are paraphrased from ASCE-7-93:

- Open Country—Open terrain with scattered obstructions having heights generally less than 30.0 ft. This category includes flat open country and grasslands.

Z_0 = friction length of upstream fetch, a meteorological wind characteristic taken as specified in Table 1 (ft.)

- Suburban—Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single-family or larger dwellings. Use of this category shall be limited to those areas for which representative terrain prevails in the upwind direction at least 1,500 ft.

- City—Large city centers with at least 50 percent of the buildings having a height in excess of 70.0 ft. Use of this category shall be limited to those areas for which representative terrain prevails in the upwind direction at least one-half mile. Possible channeling effects of increased velocity pressures due to the bridge or structure's location in the wake of adjacent structures shall be taken into account.

Table 3.8.1.1-1 Values of V_0 and Z_0 for Various Upstream Surface Conditions.

CONDITION	OPEN COUNTRY	SUBURBAN	CITY
V_0 (mph)	8.20	10.90	12.00
Z_0 (ft.)	0.23	3.28	8.20

V_{30} may be established from:

- Fastest-mile-of-wind charts available in ASCE 7-88 for various recurrence intervals,
- Site-specific wind surveys, and
- In the absence of better criterion, the assumption that $V_{30} = V_B = 100$ mph.

3.8.1.2 Wind Pressure on Structures: *WS*

3.8.1.2.1 General

If justified by local conditions, a different base design wind velocity may be selected for load combinations not involving wind on live load. The direction of the design wind shall be assumed to be horizontal, unless otherwise specified in Article 3.8.3. In the absence of more precise data, design wind pressure, in ksf, may be determined as:

$$P_D = P_B \left(\frac{V_{DZ}}{V_B} \right)^2 = P_B \frac{V_{DZ}^2}{10{,}000} \qquad (3.8.1.2.1\text{-}1)$$

P_B = base wind pressure specified in Table 1 (ksf)

C3.8.1.2.1

The stagnation pressure associated with a wind velocity of 100 mph is 0.0256 ksf, which is significantly less than the values specified in Table 1. The difference reflects the effect of gusting combined with some tradition of long-time usage.

The pressures specified in klf or ksf should be chosen to produce the greater net wind load on the structure.

Wind tunnel tests may be used to provide more precise estimates of wind pressures. Such testing should be considered where wind is a major design load.

Table 3.8.1.2.1-1 Base Pressures, P_B Corresponding to V_B = 100 mph.

Superstructure Compontent	Windward Load, ksf	Leeward Load, ksf
Trusses, Columns, and Arches	0.050	0.025
Beams	0.050	NA
Large Flat Surfaces	0.040	NA

The term "columns" in Table 1 refers to columns in superstructures such as spandrel columns in arches.

The total wind loading shall not be taken less than 0.30 klf in the plane of a windward chord and 0.15 klf in the plane of a leeward chord on truss and arch components, and not less than 0.30 klf on beam or girder spans.

3.8.1.2.2 Loads from Superstructures

Except where specified herein, where the wind is not taken as normal to the structure, the base wind pressures, P_B, for various angles of wind direction may be taken as specified in Table 1 and shall be applied to **the centroid of** a single **plane** of exposed area. The skew angle shall be taken as measured from a perpendicular to the longitudinal axis. The wind direction for design shall be that which produces the extreme force effect on the component under investigation. The transverse and longitudinal pressures shall be applied simultaneously.

C3.8.1.2.2

For trusses, columns, and arches, the base wind pressures specified in Table 1 are the sum of the pressures applied to both the windward and leeward areas.

Table 3.8.1.2.2-1 Base Wind Pressures, P_B, for Various Angles of Attack and V_B = 100 mph.

Skew Angle of Wind	Trusses, Columns and Arches		Girders	
	Lateral Load	Longitudinal Load	Lateral Load	Longitudinal Load
Degrees	ksf	ksf	ksf	ksf
0	0.075	0.000	0.050	0.000
15	0.070	0.012	0.044	0.006
30	0.065	0.028	0.041	0.012
45	0.047	0.041	0.033	0.016
60	0.024	0.050	0.017	0.019

For the usual girder and slab bridges having an individual span length of not more than 125 ft. and a maximum height of 30.0 ft. above low ground or water level the following wind loading may be used:

- 0.05 ksf, transverse
- 0.012 ksf, longitudinal

Both forces shall be applied simultaneously.

3.8.1.3 Wind Pressure on Vehicles: *WL*

When vehicles are present, the design wind pressure shall be applied to both structure and vehicles. Wind pressure on vehicles shall be represented by an interruptible, moving force of 0.10 klf acting normal to, and 6.0 ft. above, the roadway and shall be transmitted to the structure.

When wind on vehicles is not taken as normal to the structure, the components of normal and parallel force applied to the live load may be taken as specified in Table 1 with the skew angle taken as referenced normal to the surface.

C3.8.1.3

Based on practical experience, maximum live loads are not expected to be present on the bridge when the wind velocity exceeds 55 mph. The load factor corresponding to the treatment of wind on structure only in Load Combination Strength III would be $(55/100)^2 (1.4) = 0.42$, which has been rounded to 0.40 in the Strength V Load Combination. This load factor corresponds to 0.3 in Service I.

The 0.10 klf wind load is based on a long row of randomly sequenced passenger cars, commercial vans, and trucks exposed to the 55 mph design wind. This horizontal live load, similar to the design lane load, should be applied only to the tributary areas producing a force effect of the same kind.

Table 3.8.1.3-1 Wind Components on Live Load.

Skew Angle	Normal Component	Parallel Component
Degrees	klf	klf
0	0.100	0.000
15	0.088	0.012
30	0.082	0.024
45	0.066	0.032
60	0.034	0.038

3.8.2 Vertical Wind Pressure

Unless otherwise determined in Article 3.8.3, a vertical upward wind force of 0.020 ksf times the width of the deck, including parapets and sidewalks, shall be considered to be a longitudinal line load. This force shall be applied only for the Strength III and Service IV limit states which do not involve wind on live load, and only when the direction of wind is taken to be perpendicular to the longitudinal axis of the bridge. This lineal force shall be applied at the windward quarter-point of the deck width in conjunction with the horizontal wind loads specified in Article 3.8.1.

C3.8.2

The intent of this Article is to account for the effect resulting from interruption of the horizontal flow of air by the superstructure. This load is to be applied even to discontinuous bridge decks, such as grid decks. This load may govern where overturning of the bridge is investigated.

3.8.3 Aeroelastic Instability

3.8.3.1 General

Aeroelastic force effects shall be taken into account in the design of bridges and structural components apt to be wind-sensitive. For the purpose of this Article, all bridges, and structural components thereof with a span length to width or depth ratio exceeding 30.0 shall be deemed to be wind-sensitive.

The vibration of cables due to the interaction of wind and rain shall also be considered.

C3.8.3.1

Because of the complexity of analyses often necessary for an in-depth evaluation of structural aeroelasticity, this Article is intentionally kept to a simple statement. Many bridges, decks, or individual structural components have been shown to be aeroelastically insensitive if their length-to-width or length-to-depth ratios are under about 30.0, a somewhat arbitrary value helpful only in identifying likely wind-sensitive cases.

Flexible bridges, such as cable-supported or very long spans of any type, may require special studies based on wind tunnel information. In general, appropriate wind tunnel tests involve simulation of the wind environment local to the bridge site. Details of this are part of the existing wind tunnel state of the art and are beyond the scope of this commentary.

3.8.3.2 Aeroelastic Phenomena

The aeroelastic phenomena of vortex excitation, galloping, flutter, and divergence shall be considered where applicable.

C3.8.3.2

Excitation due to vortex shedding is the escape of wind-induced vortices behind the member, which tend to excite the component at its fundamental natural frequency in harmonic motion. It is important to keep stresses due to vortex-induced oscillations below the "infinite life" fatigue stress. Methods exist for estimating such stress amplitudes, but they are outside the scope of this commentary.

Tubular components can be protected against vortex-induced oscillation by adding bracing, strakes, or tuned mass dampers or by attaching horizontal flat plates parallel to the tube axis above and/or below the central third of their span. Such aerodynamic damper plates should lie about one-third tube diameter above or below the tube to allow free passage of wind. The width of the plates may be the diameter of the tube.

Galloping is a high-amplitude oscillation associated with ice-laden cables or long, flexible members having aerodynamically unsymmetrical cross-sections. Cable-stays, having circular sections, will not gallop unless their circumferences are deformed by ice, dropping water, or accumulated debris.

Flexible bridge decks, as in very long spans and some pedestrian bridges, may be prone to wind-induced flutter, a wind-excited oscillation of destructive amplitudes, or, on some occasions, divergence, an irreversible twist under high wind. Analysis methods, including wind tunnel studies leading to adjustments of the deck form, are available for prevention of both flutter and divergence.

3.8.3.3 Control of Dynamic Responses

Bridges and structural components thereof, including cables, shall be designed to be free of fatigue damage due to vortex-induced or galloping oscillations. Bridges shall be designed to be free of divergence and catastrophic flutter up to 1.2 times the design wind velocity applicable at bridge deck height.

C3.8.3.3

Cables in stayed-girder bridges have been successfully stabilized against excessive dynamic responses by attaching automotive dampers to the bridge at deck level or by cross-tying multiple cable-stays.

3.8.3.4 Wind Tunnel Tests

Representative wind tunnel tests may be used to satisfy the requirements of Articles 3.8.3.2 and 3.8.3.3.

C3.8.3.4

Wind tunnel testing of bridges and other civil engineering structures is a highly developed technology, which may be used to study the wind response characteristics of a structural model or to verify the results of analysis (*Simiu 1976*).

3.9 ICE LOADS: *IC*

3.9.1 General

This Article refers only to freshwater ice in rivers and lakes; ice loads in seawater should be determined by suitable specialists using site-specific information.

Ice forces on piers shall be determined with regard to site conditions and expected modes of ice action as follows:

- Dynamic pressure due to moving sheets or floes of ice being carried by stream flow, wind, or currents;
- Static pressure due to thermal movements of ice sheets;
- Pressure resulting from hanging dams or jams of ice; and
- Static uplift or vertical load resulting from adhering ice in waters of fluctuating level.

The expected thickness of ice, the direction of its movement, and the height of its action shall be determined by field investigations, review of public records, aerial surveys, or other suitable means.

C3.9.1

Most of the information for ice loads was taken from Montgomery et al. (*1984*), which provided background for the clauses on ice loads for Canadian Standards Association (*1988*). A useful additional source has been Neill (*1981*).

It is convenient to classify ice forces on piers as dynamic forces and static forces.

Dynamic forces occur when a moving ice floe strikes a bridge pier. The forces imposed by the ice floe on a pier are dependent on the size of the floe, the strength and thickness of the ice, and the geometry of the pier.

The following types of ice failure have been observed (*Montgomery et al. 1984*):

- Crushing, where the ice fails by local crushing across the width of a pier. The crushed ice is continually cleared from a zone around the pier as the floe moves past.
- Bending, where a vertical reaction component acts on the ice floe impinging on a pier with an inclined nose. This reaction causes the floe to rise up the pier nose, as flexural cracks form.
- Splitting, where a comparatively small floe strikes a pier and is split into smaller parts by stress cracks propagating from the pier.
- Impact, where a small floe is brought to a halt by impinging on the nose of the pier before it has crushed over the full width of the pier, bent or split.
- Buckling, where compressive forces cause a large floe to fail by buckling in front of the nose of a very wide pier.

For bridge piers of usual proportions on larger bodies of water, crushing and bending failures usually control the magnitude of the design dynamic ice force. On smaller streams, which cannot carry large ice floes, impact failure can be the controlling mode.

In all three cases, it is essential to recognize the effects of resonance between the pier and the ice forces. Montgomery et al. (*1980*) have shown that for a massive pier with a damping coefficient of 20 percent of critical, the maximum dynamic effect is approximately equal to the greatest force, but for lesser damping values there is a considerable amplification.

Montgomery and Lipsett (*1980*) measured damping of a massive pier at 19 percent of critical, but it is expected that slender piers and individual piles may have damping values of 5 percent or less.

In the discussion of impact-type ice failure above, the indication is that the floe is "small." Small is extremely difficult to define and is site-specific. Floes up to 75.0 ft. long have been observed to fail by splitting when driven by water velocities of 10.0 ft./sec. (*Haynes 1996*).

Static forces may be caused by the thermal expansion of ice in which a pier is embedded or by irregular growth of the ice field. This has typically been observed downstream of a dam, or hydroelectric plant or other channel where ice predominantly forms only on one side of the river or pier.

Ice jams can arch between bridge piers. The break-up ice jam is a more or less cohesionless accumulation of ice fragments (*Montgomery et al. 1984*).

Hanging dams are created when frazil ice passes under the surface layer of ice and accumulates under the surface ice at the bridge site. The frazil ice comes typically from rapids or waterfalls upstream. The hanging dam can cause a backup of water, which exerts pressure on the pier and can cause scour around or under piers as water flows at an increased velocity.

3.9.2 Dynamic Ice Forces on Piers

3.9.2.1 Effective Ice Strength

In the absence of more precise information, the following values may be used for effective ice crushing strength:

- 8.0 ksf, where breakup occurs at melting temperatures and the ice structure is substantially disintegrated;
- 16.0 ksf, where breakup occurs at melting temperatures and the ice structure is somewhat disintegrated;
- 24.0 ksf, where breakup or major ice movement occurs at melting temperatures, but the ice moves in large pieces and is internally sound; and
- 32.0 ksf, where breakup or major ice movement occurs when the ice temperature, averaged over its depth, is measurably below the melting point.

C3.9.2.1

It should be noted that the effective ice strengths given herein are for the purpose of entering into a formula to arrive at forces on piers. Different formulas might require different effective ice strengths to arrive at the same result.

As a guide, the 8.0 ksf strength is appropriate for piers where long experience indicates that ice forces are minimal, but some allowance is required for ice effects; the 32.0 ksf strength is considered to be a reasonable upper limit based on the observed history of bridges that have survived ice conditions (*Neill 1981*). Effective ice strengths of up to 57.6 ksf have been used in the design of some bridges in Alaska (*Haynes 1996*).

The effective ice strength depends mostly on the temperature and grain size of the ice (*Montgomery et al. 1984*). For example, laboratory measured compressive strengths at 32°F vary from about 60.0 ksf for grain sizes of 0.04 in. to 27.0 ksf for grain sizes of 0.2 in., and at 23°F ice strengths are approximately double the values given. Thus, the effective ice strengths given herein are not necessarily representative of laboratory tests or actual ice strengths, and, in fact, are on the order of one-half of observed values (*Neill 1981*).

The compressive strength of the ice depends upon temperature, but the tensile strength is not sensitive to temperature. Because much ice failure is the result of splitting or tensile failure in bending, and because grain sizes, cracks, and other imperfections vary in the field, only crude approximations of ice strengths can be made. Thus, temperature is not a consideration for setting effective ice strengths in these Specifications.

Some of the most severe ice runs in the United States occur during a rapid January thaw, when the air temperature is about 50°F, but the average ice temperature can still be below 32°F because of an insulating snow cover (*Haynes 1996*).

3.9.2.2 Crushing and Flexing

The horizontal force, F, resulting from the pressure of moving ice shall be taken as:

- If $\frac{w}{t} \leq 6.0$, then:

F = lesser of either F_c or, when ice failure by flexure is considered applicable as described herein, F_b, and

- If $\frac{w}{t} > 6.0$, then:

$F = F_c$

in which:

$$F_c = C_a ptw \quad (3.9.2.2\text{-}1)$$

$$F_b = C_n pt^2 \quad (3.9.2.2\text{-}2)$$

$$C_a = (5t/w+1)^{0.5} \quad (3.9.2.2\text{-}3)$$

$$C_n = \frac{0.5}{\tan(\alpha - 15)} \quad (3.9.2.2\text{-}4)$$

where:

t = thickness of ice (ft.)

α = inclination of the nose to the vertical (°)

p = effective ice crushing strength as specified in Article 3.9.2.1 (ksf)

w = pier width at level of ice action (ft.)

F_c = horizontal ice force caused by ice floes that fail by crushing over the full width of the pier (kip)

F_b = horizontal ice force caused by ice floes that fail by flexure as they ride up the inclined pier nose (kip)

C_a = coefficient accounting for the effect of the pier width/ice thickness ratio where the floe fails by crushing

C3.9.2.2

The expression of F_c is based on field measurements of forces on two bridge piers in Alberta (*Lipsett and Gerard 1980*). See also Huiskamp (*1983*), with a C_a proposed by Afanas'ev et al. (*1971*), and verified by Neill (*1976*).

The expression for F_b is taken from Lipsett and Gerard (*1980*).

$w/t = 6.0$ is a rough estimate of the upper limit of w/t at which ice that has failed by bending will be washed around the pier.

It is assumed that the force on the pier is governed by the crushing or bending strength of the ice, and thus there is not a term in Eqs. 1 or 2 relating to velocity of the ice. The interaction between an ice floe and a pier depends on the size and strength of the floe and how squarely it strikes the pier. It has been reported that an ice floe 200 ft. in size will usually fail by crushing if it hits a pier squarely. If a floe 100 ft. in size does not hit the pier squarely, it will usually impact the pier and rotate around the pier and pass downstream with only little local crushing.

Although no account is taken of the shape of the nose of the pier, laboratory tests at the U.S. Army Corps of Engineers' Cold Regions Research and Engineering Laboratory (CRREL) have shown the bullet-shaped pier nose can reduce ice forces the most compared to other types of geometry. Pointed angular noses, as shown in Figure C3.9.2.4.1-1, have been found to cause lateral vibrations of the pier without reducing the streamwise force. CRREL has measured lateral or torsional vibrations on the pointed nose Yukon River Bridge piers. The long-term ramifications of these vibrations are not known at this time (*Haynes 1996*).

Ice thickness is the greatest unknown in the determination of ice forces on piers. Equations can be used for estimating ice thickness. The design should be based on the extreme, not average, ice thickness. The elevation on the pier where the design force shall be applied is important for calculating the overturning moments. Because ice stage increases during an ice run, relying on local knowledge of the maximum stage is vital to proper design (*Haynes 1995*). For the purpose of design, the preferred method to establish the thickness of ice, t, is to base it on measurements of maximum thicknesses, taken over a period of several years, at the potential bridge sites.

Where observations over a long period of time are not available, an empirical method based on Neill (*1981*) is suggested as follows:

$$t = 0.083\alpha\sqrt{S_f} \quad (C3.9.2.2\text{-}1)$$

C_n = coefficient accounting for the inclination of the pier nose with respect to a vertical

where $\alpha \leq 15°$, ice failure by flexure shall not be considered to be a possible ice failure mode for the purpose of calculating the horizontal force, F, in which case F shall be taken as F_c.

where:

α = coefficient for local conditions, normally less than 1.0

S_f = freezing index, being the algebraic sum, $\Sigma(32 - T)$, summed from the date of freeze-up to the date of interest, in degree days

T = mean daily air temperature (°F)

Assuming that temperature records are available, the maximum recorded value of S_f can be determined.

One possible method of determining α is by simple calibration in which, through the course of a single winter, the ice thickness can be measured at various times and plotted against $\sqrt{S_f}$.

As a guide, Neill (*1981*) indicates the following values for α:

windy lakes without snow............................0.8
average lake with snow..........................0.5–0.7
average river with snow..........................0.4–0.5
sheltered small river with snow.................0.2–0.4

Due to its good insulating characteristics, snow has a significant effect on ice growth. Williams (*1963*) has shown that a snow cover greater than 6.0 in. in thickness has the effect of reducing α by as much as 50 percent.

Neill does not define "average," and it has been noted by Gerard and Stanely (*1992*) that deep snow can produce snow-ice, thus offsetting the benefits of snow insulation.

Large lakes take longer to cool down, which leads to a later freeze-up date. This results in fewer degree-days of freezing and, hence, smaller ice thicknesses.

The remaining decision is to establish the appropriate elevation of the ice force to be applied to the pier. The elevation required is that at break-up, not at the mean winter level. Neill (*1981*) suggests several methods of determining ice elevations, but the most common method in general use is probably to rely on local knowledge and examination of the river banks to determine the extent of damage by ice, such as the marking or removal of trees.

3.9.2.3 Small Streams

On small streams not conducive to the formation of large ice floes, consideration may be given to reducing the forces F_b and F_c, determined in accordance with Article 3.9.2.2, but under no circumstances shall the forces be reduced by more than 50 percent.

C3.9.2.3

CAN/CSA-S6-88 has an expression for ice forces in small streams, for which a theory is given by Montgomery et al. (*1984*). It is considered insufficiently verified to be included herein.

On small streams, with a width of less than 300 ft. at the mean water level, dynamic ice forces, as determined in Article 3.9.2.2, may be reduced in accordance with Table C1. Another important factor that determines the ice floe size are the type of features in the river upstream of the site. Islands, dams, and bridge piers can break ice into small floes.

where:

A = plan area of the largest ice floe in (ft.^{2})

r = radius of pier nose (ft.)

Table C3.9.2.3-1 Reduction Factor K_1 for Small Streams.

A/r^2	Reduction Factor, K_1
1000	1.0
500	0.9
200	0.7
100	0.6
50	0.5

The rationale for the reduction factor, K_1, is that the bridge may be struck only by small ice floes with insufficient momentum to cause failure of the floe.

3.9.2.4 Combination of Longitudinal and Transverse Forces

3.9.2.4.1 Piers Parallel to Flow

The force F, determined as specified in Articles 3.9.2.2 and 3.9.2.3, shall be taken to act along the longitudinal axis of the pier if the ice movement has only one direction and the pier is approximately aligned with that direction. In this case, two design cases shall be investigated as follows:

- A longitudinal force equal to F shall be combined with a transverse force of $0.15F$, or
- A longitudinal force of $0.5F$ shall be combined with a transverse force of F_t.

The transverse force, F_t, shall be taken as:

$$F_t = \frac{F}{2\tan(\beta/2 + \theta_f)} \qquad (3.9.2.4.1\text{-}1)$$

where:

β = nose angle in a horizontal plane for a round nose taken as 100 (°)

θ_f = friction angle between ice and pier nose (°)

Both the longitudinal and transverse forces shall be assumed to act at the pier nose.

C3.9.2.4.1

It would be unrealistic to expect the ice force to be exactly parallel to the pier, so a minimum lateral component of 15 percent of the longitudinal force is specified.

The expression for F_t comes from Montgomery et al. (*1984*), and is explained in Figure C1 taken from the same source.

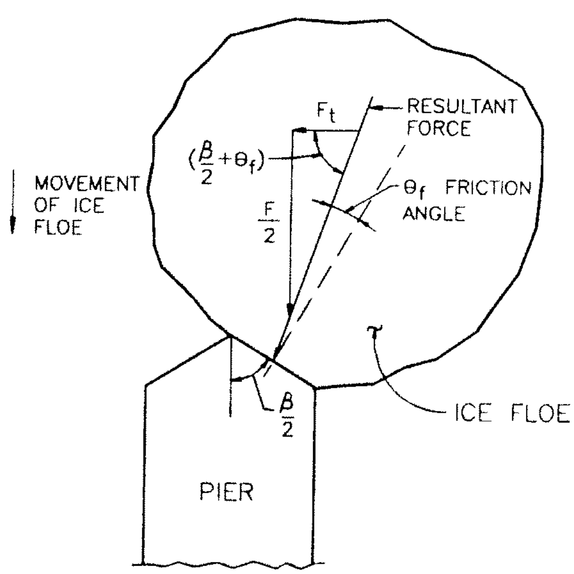


Figure C3.9.2.4.1-1 Transverse Ice Force Where a Floe Fails Over a Portion of a Pier.

3.9.2.4.2 Piers Skewed to Flow

Where the longitudinal axis of a pier is not parallel to the principal direction of ice action, or where the direction of ice action may shift, the total force on the pier shall be determined on the basis of the projected pier width and resolved into components. Under such conditions, forces transverse to the longitudinal axis of the pier shall be taken to be at least 20 percent of the total force.

C3.9.2.4.2

The provisions for piers skewed to flow are taken from CAN/CSA-S6-88 (*1988*).

3.9.2.5 Slender and Flexible Piers

Slender and flexible piers shall not be used in regions where ice forces are significant, unless advice on ice/structure interaction has been obtained from an ice specialist. This provision also applies to slender and flexible components of piers, including piles that come into contact with water-borne ice.

C3.9.2.5

It has been shown by Montgomery et al. (*1980*) and others that flexible piers and pier components may experience considerable amplification of the ice forces as a result of resonant ice/structure interaction at low levels of structural damping. In this case, the provisions of Article 3.9.5 may be inadequate for vertical forces on piers.

3.9.3 Static Ice Loads on Piers

Ice pressures on piers frozen into ice sheets shall be investigated where the ice sheets are subject to significant thermal movements relative to the pier where the growth of shore ice is on one side only or in other situations that may produce substantial unbalanced forces on the pier.

C3.9.3

Little guidance is available for predicting static ice loads on piers. Under normal circumstances, the effects of static ice forces on piers may be strain-limited, but expert advice should be sought if there is reason for concern. Static ice forces due to thermal expansion of ice are discussed in Haynes (*1995*). Ice force can be reduced by several mitigating factors that usually apply. For example, ice does not act simultaneously over the full length of the pier. Thermal stresses relax in time and prevent high stresses over the full ice thickness. A snow cover on the ice insulates the ice and reduces the thermal stresses, and ice usually acts simultaneously on both sides of the pier surrounded by the ice so that the resultant force is considerably less than the larger directional force, i.e., force on one side of the pier. Article C3.9.1 contains additional discussion.

3.9.4 Hanging Dams and Ice Jams

The frazil accumulation in a hanging dam may be taken to exert a pressure of 0.2 to 2.0 ksf as it moves by the pier. An ice jam may be taken to exert a pressure of 0.02 to 0.20 ksf.

C3.9.4

The theory behind the ice pressures given for hanging dams can be found in Montgomery et al. (*1984*). The wide spread of pressures quoted reflects both the variability of the ice and the lack of firm information on the subject.

3.9.5 Vertical Forces Due to Ice Adhesion

The vertical force, in kips, on a bridge pier due to rapid water level fluctuation shall be taken as:

For a circular pier:

$$F_v = 80.0t^2\left(0.35 + \frac{0.03R}{t^{0.75}}\right) \qquad (3.9.5\text{-}1)$$

C3.9.5

Eq. 1 was derived by considering the failure of a semi-infinite, wedge-shaped ice sheet on an elastic foundation under vertical load applied at its apex. For a single ice wedge, the maximum vertical force, P, can be evaluated from the expression (*Nevel 1972*).

$$P = \frac{\tan\left(\frac{\delta}{2}\right)\sigma_T t^2}{3}\left[1.05 + 2\left(\frac{a}{\ell}\right) + 0.5\left(\frac{a}{\ell}\right)^3\right] \qquad (C3.9.5\text{-}1)$$

For an oblong pier:

$$F_v = 0.2t^{1.25}L + 80.0t^2\left(0.35 + \frac{0.03R}{t^{0.75}}\right) \qquad (3.9.5\text{-}2)$$

where:

t = ice thickness (ft.)

R = radius of circular pier (ft.); or radius of half circles at ends of an oblong pier (ft.); or radius of a circle that circumscribes each end of an oblong pier of which the ends are not circular in plan at water level (ft.)

L = perimeter of pier, excluding half circles at ends of oblong pier (ft.)

in which:

$$\ell = \left(\frac{Et^3}{12\gamma}\right)^{0.25} = 21.0t^{0.75} \qquad (C3.9.5\text{-}2)$$

where:

σ_T = tensile strength of ice (ksf)

t = maximum thickness of ice (ft.)

δ = angle of the truncated wedge (°)

a = truncated distance, which is assumed to be equal to the radius of a circular pier (ft.)

ℓ = characteristic length calculated from the expression (ft.)

E = Young's modulus for ice (ksf)

γ = unit weight of water (kcf)

To obtain Eq. 1, the vertical force is summed for four wedges, each with a truncated angle of 90°. It is assumed that the tensile strength of ice is 0.84 times an effective crushing strength of 23 ksf and that the ratio of the truncated distance to the characteristic length, a/ℓ, is less than 0.6.

Eq. 2 is the sum of two expressions:

- Eq. 1, which accounts for the vertical ice forces acting on the half circles at the ends of an oblong pier, and

- An expression that calculates the vertical ice forces on the straight walls of the pier.

The expression for calculating the vertical ice forces on the long straight walls of the pier was derived by considering a semi-infinite, rectangular ice sheet on an elastic foundation under a uniformly distributed edge load. The force required to fail the ice sheet, F, can be expressed as $F = 0.236\,\sigma_T\, t^2/\ell$ (*Montgomery et al. 1984*).

Eqs. 1 and 2 are based on the conservative assumption that ice adheres around the full perimeter of the pier cross-section. They neglect creep and are, therefore, conservative for water level fluctuations occurring over more than a few minutes. However, they are also based on the nonconservative assumption that failure occurs on the formation of the first crack.

Some issues surrounding ice forces have been reported in Zabilansky (*1996*).

3.9.6 Ice Accretion and Snow Loads on Superstructures

Generally snow loads, other than those caused by an avalanche, need not be considered. However, Owners in areas where unique accumulations of snow and/or ice are possible should specify appropriate loads for that condition.

Loads due to icing of the superstructure by freezing rain shall be specified if local conditions so warrant.

C3.9.6

The following discussion of snow loads is taken from Ritter (*1990*).

Snow loads should be considered where a bridge is located in an area of potentially heavy snowfall. This can occur at high elevations in mountainous areas with large seasonal accumulations. Snow loads are normally negligible in areas of the United States that are below 2,000 ft. elevation and east of longitude 105°W, or below 1,000 ft. elevation and west of longitude 105°W. In other areas of the country, snow loads as large as 0.7 ksf may be encountered in mountainous locations.

The effects of snow are assumed to be offset by an accompanying decrease in vehicle live load. This assumption is valid for most structures, but is not realistic in areas where snowfall is significant. When prolonged winter closure of a road makes snow removal impossible, the magnitude of snow loads may exceed those from vehicular live loads. Loads also may be notable where plowed snow is stockpiled or otherwise allowed to accumulate. The applicability and magnitude of snow loads are left to the Designer's judgment.

Snow loads vary from year to year and depend on the depth and density of snow pack. The depth used for design should be based on a mean recurrence interval or the maximum recorded depth. Density is based on the degree of compaction. The lightest accumulation is produced by fresh snow falling at cold temperatures. Density increases when the snow pack is subjected to freeze-thaw cycles or rain. Probable densities for several snow pack conditions are indicated in Table C1, ASCE (*1980*).

Table C3.9.6-1 Snow Density.

CONDITION OF SNOW PACK	PROBABLE DENSITY (kcf)
Freshly Fallen	0.006
Accumulated	0.019
Compacted	0.031
Rain or Snow	0.031

Estimated snow load can be determined from historical records or other reliable data. General information on ground snow loads is available from the National Weather Service, from state and local agencies, and ASCE (*1988*). Snow loads in mountain areas are subject to extreme variations. The extent of these loads should be determined on the basis of local experience or records, instead of on generalized information.

The effect of snow loads on a bridge structure is influenced by the pattern of snow accumulation. Windblown snow drifts may produce unbalanced loads considerably greater than those produced from uniformly distributed loads. Drifting is influenced by the terrain, structure shape, and other features that cause changes in the general wind flow. Bridge components, such as railings, can serve to contain drifting snow and cause large accumulations to develop.

3.10 EARTHQUAKE EFFECTS: *EQ*

3.10.1 General

Bridges shall be designed to have a low probability of collapse but may suffer significant damage and disruption to service when subject to earthquake ground motions that have a 7 percent probability of exceedance in 75 years. Partial or complete replacement may be required. Higher levels of performance may be used with the authorization of the bridge owner.

Earthquake loads shall be taken to be horizontal force effects determined in accordance with the provisions of Article 4.7.4 on the basis of the elastic response coefficient, C_{sm}, specified in Article 3.10.4, and the equivalent weight of the superstructure, and adjusted by the response modification factor, R, specified in Article 3.10.7.1.

The provisions herein shall apply to bridges of conventional **construction. The** Owner shall specify and/or approve appropriate provisions **for nonconventional construction**. Unless otherwise specified by the Owner, these provisions need not be applied to completely buried structures.

Seismic effects for box culverts and buried structures need not be considered, except where they cross active faults.

The potential for soil liquefaction and slope movements shall be considered.

C3.10.1

The design earthquake motions and forces specified in these provisions are based on a low probability of their being exceeded during the normal life expectancy of a bridge. Bridges that are designed and detailed in accordance with these provisions may suffer damage, but should have low probability of collapse due to seismically induced ground shaking.

The principles used for the development of these Specifications are:

- Small to moderate earthquakes should be resisted within the elastic range of the structural components without significant damage;
- Realistic seismic ground motion intensities and forces should be used in the design procedures; and
- Exposure to shaking from large earthquakes should not cause collapse of all or part of the bridge. Where possible, damage that does occur should be readily detectable and accessible for inspection and repair.

Bridge Owners may choose to mandate higher levels of performance for special bridges.

Earthquake loads are given by the product of the elastic seismic response coefficient C_{sm} and the equivalent weight of the superstructure. The equivalent weight is a function of the actual weight and bridge configuration and is automatically included in both the single-mode and multimode methods of analysis specified in Article 4.7.4. Design and detailing provisions for bridges to minimize their susceptibility to damage from earthquakes are contained in Sections 3, 4, 5, 6, 7, 10, and 11. A flow chart summarizing these provisions is presented in Appendix A3.

Conventional bridges include those with slab, beam, box girder, or truss superstructures, and single- or multiple-column piers, wall-type piers, or pile-bent substructures. In addition, conventional bridges are founded on shallow or piled footings, or shafts. Substructures for conventional bridges are also listed in Table 3.10.7.1-1. Nonconventional bridges include bridges with cable-stayed/cable-suspended superstructures, bridges with truss towers or hollow piers for substructures, and arch bridges.

These Specifications are considered to be "force-based" wherein a bridge is designed to have adequate strength (capacity) to resist earthquake forces (demands). In recent years there has been a trend away from "force-based" procedures to those that are "displacement-based," wherein a bridge is designed to have adequate displacement capacity to accommodate earthquake demands. Displacement-based procedures are believed to more reliably identify the limit states that cause damage leading to collapse, and in some cases produce more efficient designs against collapse. It is recommended that the displacement capacity of bridges designed in accordance with these Specifications, be checked using a displacement-based procedure, particularly those bridges in high seismic zones. The *AASHTO Guide Specifications for LRFD Seismic Design* (***AASHTO, 2008***), are "displacement-based."

3.10.2 Seismic Hazard

The seismic hazard at a bridge site shall be characterized by the acceleration response spectrum for the site and the site factors for the relevant site class.

The acceleration spectrum shall be determined using either the General Procedure specified in Article 3.10.2.1 or the Site Specific Procedure specified in Article 3.10.2.2.

A Site-Specific Procedure shall be used if any one of the following conditions exist:

- The site is located within 6 mi. of an active fault,
- The site is classified as Site Class F (Article 3.10.3.1),
- Long-duration earthquakes are expected in the region,
- The importance of the bridge is such that a lower probability of exceedance (and therefore a longer return period) should be considered.

If time histories of ground acceleration are used to characterize the seismic hazard for the site, they shall be determined in accordance with Article 4.7.4.3.4b.

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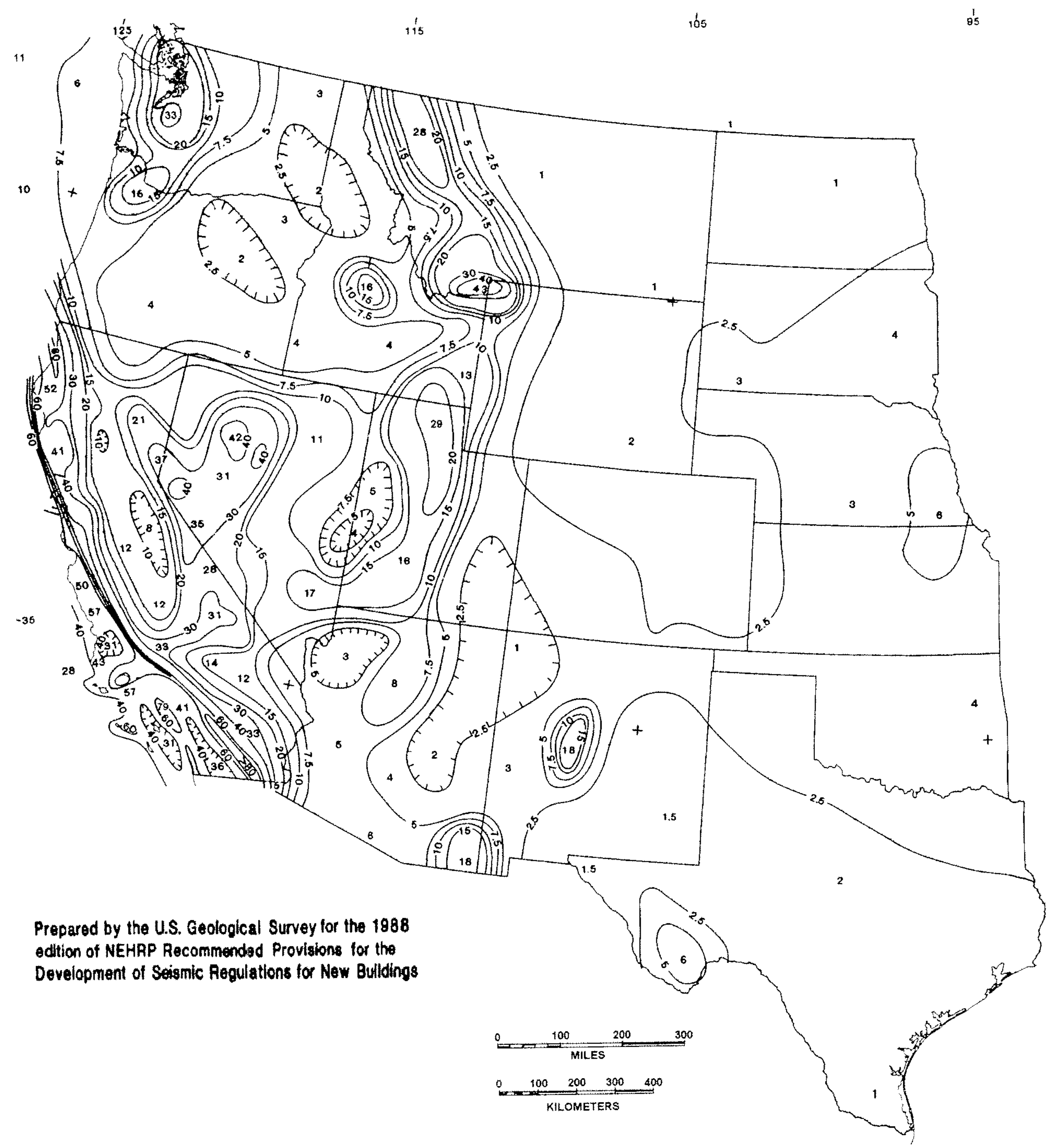

Figure 3.10.2-1 Acceleration Coefficient for Contiguous States Generally West of the 95th Longitude.

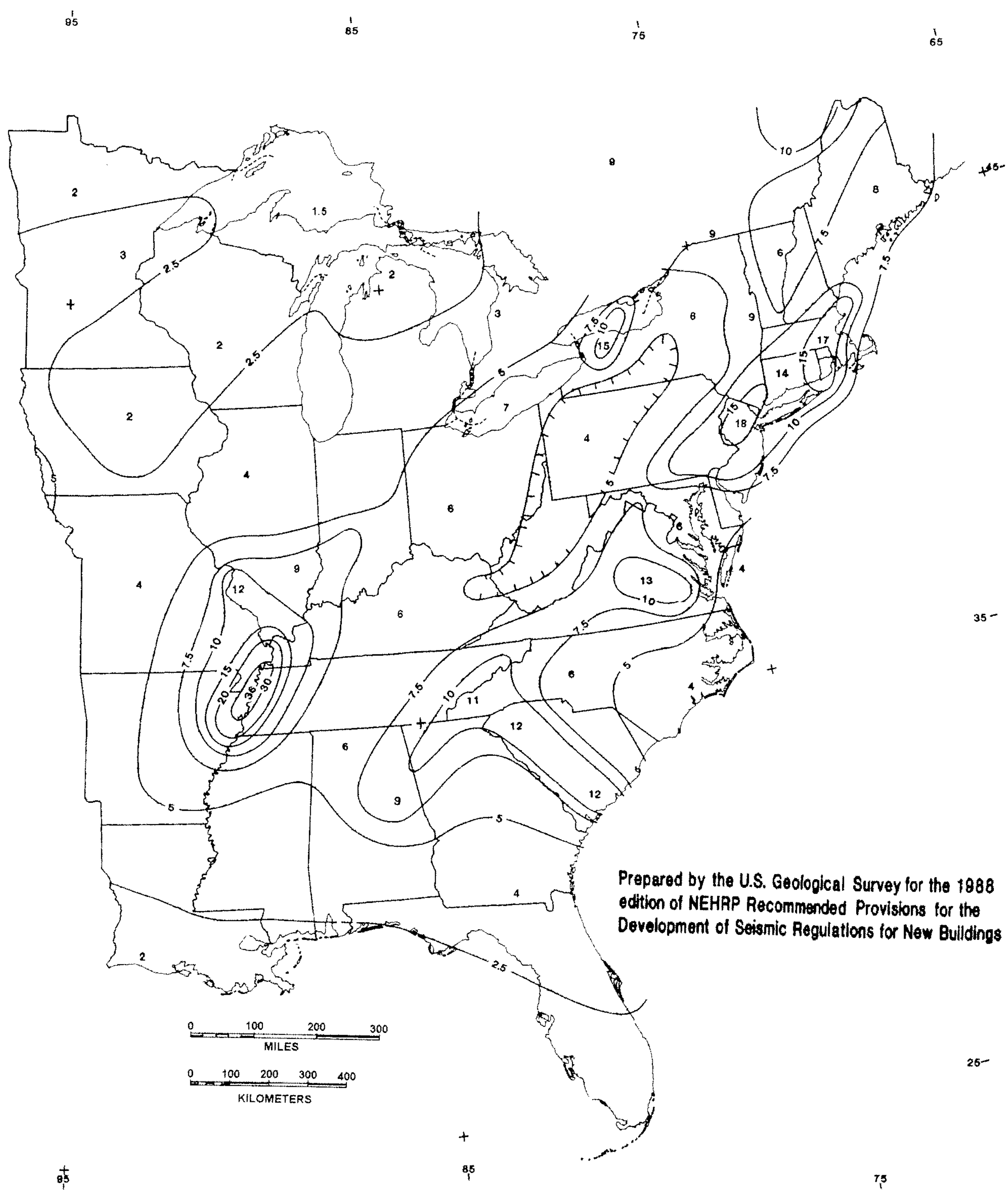

Figure 3.10.2-2 Acceleration Coefficient for Contiguous States Generally East of the 95th Longitude.

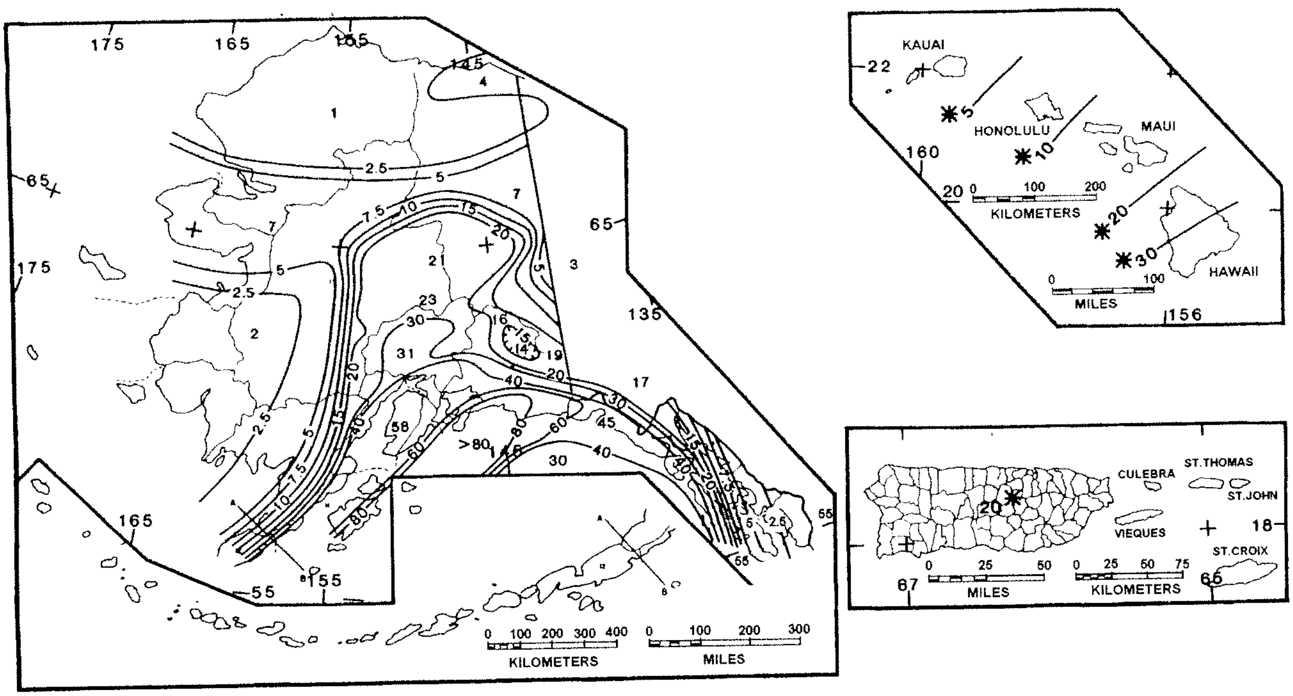

Figure 3.10.2-3 Acceleration Coefficient for Alaska, Hawaii, and Puerto Rico.

3.10.3 Importance Categories

For the purpose of Article 3.10, the Owner or those having jurisdiction shall classify the bridge into one of three importance categories as follows:

- Critical bridges,
- Essential bridges, or
- Other bridges.

The basis of classification shall include social/survival and security/defense requirements. In classifying a bridge, consideration should be given to possible future changes in conditions and requirements.

C3.10.3

Essential bridges are generally those that should, as a minimum, be open to emergency vehicles and for security/defense purposes immediately after the design earthquake, i.e., a 475-year return period event. However, some bridges must remain open to all traffic after the design earthquake and be usable by emergency vehicles and for security/defense purposes immediately after a large earthquake, e.g., a 2,500-year return period event. These bridges should be regarded as critical structures.

3.10.4 Seismic Performance Zones

Each bridge shall be assigned to one of the four seismic zones in accordance with Table 1.

Table 3.10.4-1 Seismic Zones.

Acceleration Coefficient	Seismic Zone
$A \leq 0.09$	1
$0.09 < A \leq 0.19$	2
$0.19 < A \leq 0.29$	3
$0.29 < A$	4

C3.10.4

These seismic zones reflect the variation in seismic risk across the country and are used to permit different requirements for methods of analysis, minimum support lengths, column design details, and foundation and abutment design procedures.

3.10.5 Site Effects

3.10.5.1 General

Site effects shall be included in the determination of seismic loads for bridges.

The site coefficient, *S*, specified in Table 1, shall be based upon soil profile types defined in Articles 3.10.5.2 through 3.10.5.5.

C3.10.5.1

Site effects on structural response are due to the soil conditions. Four soil profiles are used in these Specifications to define a site coefficient used to modify the acceleration coefficient. These soil profiles are representative of different subsurface conditions, which were selected on the basis of a statistical study of spectral shapes developed on such soils close to seismic source zones in past earthquakes.

The site coefficient, *S*, is used to include the effect of site conditions on the elastic seismic response coefficient as specified in Article 3.10.6.

Table 3.10.5.1-1 Site Coefficients.

Site Coefficient	Soil Profile Type			
	I	II	III	IV
S	1.0	1.2	1.5	2.0

In locations where the soil properties are not known in sufficient detail to determine the soil profile type, or where the profile does not fit any of the four types, the site coefficient for Soil Profile Type II shall be used.

The decision to specify Type II as a default site coefficient was a committee decision based on judgment during the development of the parent provisions under Project ATC-6.

3.10.5.2 Soil Profile Type I

A profile shall be taken as Type I if composed of:

- Rock of any description, either shale-like or crystalline in nature, or
- Stiff soils where the soil depth is less than 200 ft., and the soil types overlying the rock are stable deposits of sands, gravels, or stiff clays.

C3.10.5.2

These materials may be characterized by a shear wave velocity greater than 2,500 ft./sec.

3.10.5.3 Soil Profile Type II

A profile with stiff cohesive or deep cohesionless soils where the soil depth exceeds 200 ft. and the soil types overlying the rock are stable deposits of sands, gravels, or stiff clays shall be taken as Type II.

3.10.5.4 Soil Profile Type III

A profile with soft to medium-stiff clays and sands, characterized by 30.0 ft. or more of soft to medium-stiff clays with or without intervening layers of sand or other cohesionless soils shall be taken as Type III.

3.10.5.5 Soil Profile Type IV

A profile with soft clays or silts greater than 40.0 ft. in depth shall be taken as Type IV.

C3.10.5.5

These materials may be characterized by a shear wave velocity of less than 500 ft./sec. and might include loose natural deposits or manmade, nonengineered fill.

3.10.6 Elastic Seismic Response Coefficient

3.10.6.1 General

Unless specified otherwise in Article 3.10.6.2, the elastic seismic response coefficient, C_{sm}, for the m^{th} mode of vibration shall be taken as:

$$C_{sm} = \frac{1.2AS}{T_m^{2/3}} \leq 2.5A \tag{3.10.6.1-1}$$

where:

T_m = period of vibration of the m^{th} mode (sec.)

A = acceleration coefficient specified in Article 3.10.2

S = site coefficient specified in Article 3.10.5

The determination of the period of vibration, T_m, should be based on the nominal, unfactored mass of the component or structure.

C3.10.6.1

The elastic seismic response coefficient may be normalized using the input ground acceleration A and the result plotted against the period of vibration. Such a plot is given in Figure C1 for different soil profiles, based on 5 percent damping.

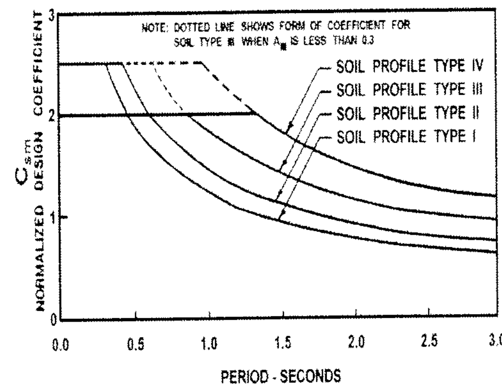

Figure C3.10.6.1-1 Seismic Response Coefficients for Various Soil Profiles, Normalized with Respect to Acceleration Coefficient *A*.

An earthquake may excite several modes of vibration in a bridge and, therefore, the elastic response coefficient should be found for each relevant mode.

The discussion of the single-mode method in the commentary to Article 4.7.4.3.2 is used to illustrate the relation between period, C_{sm}, and quasi-static seismic forces, $P_e(x)$. The structure is analyzed for these seismic forces in the single-mode method. In the multimode method, the structure is analyzed for several seismic forces, each corresponding to the period and mode shape of one of the fundamental modes of vibration, and the results are combined using acceptable methods, such as the root-mean-square method.

3.10.6.2 Exceptions

For bridges on soil profiles III or IV and in areas where the coefficient A is not less than 0.30, C_{sm} need not exceed $2.0A$.

For soil profiles III and IV, and for modes other than the fundamental mode that have periods less than 0.3 sec., C_{sm} shall be taken as:

$$C_{sm} = A(0.8 + 4.0T_m) \tag{3.10.6.2-1}$$

If the period of vibration for any mode exceeds 4.0 sec., the value of C_{sm} for that mode shall be taken as:

$$C_{sm} = \frac{3AS}{T_m^{4/3}} \quad (3.10.6.2\text{-}2)$$

3.10.7 Response Modification Factors

3.10.7.1 General

To apply the response modification factors specified herein, the structural details shall satisfy the provisions of Articles 5.10.2.2, 5.10.11, and 5.13.4.6.

Except as noted herein, seismic design force effects for substructures and the connections between parts of structures, listed in Table 2, shall be determined by dividing the force effects resulting from elastic analysis by the appropriate response modification factor, R, as specified in Tables 1 and 2, respectively.

As an alternative to the use of the R-factors, specified in Table 2 for connections, monolithic joints between structural members and/or structures, such as a column-to-footing connection, may be designed to transmit the maximum force effects that can be developed by the inelastic hinging of the column or multicolumn bent they connect as specified in Article 3.10.9.4.3.

If an inelastic time history method of analysis is used, the response modification factor, R, shall be taken as 1.0 for all substructure and connections.

C3.10.7.1

These Specifications recognize that it is uneconomical to design a bridge to resist large earthquakes elastically. Columns are assumed to deform inelastically where seismic forces exceed their design level, which is established by dividing the elastically computed force effects by the appropriate R-factor.

R-factors for connections are smaller than those for substructure members in order to preserve the integrity of the bridge under these extreme loads. For expansion joints within the superstructure and connections between the superstructure and abutment, the application of the R-factor results in force effect magnification. Connections that transfer forces from one part of a structure to another include, but are not limited to, fixed bearings, expansion bearings with either restrainers, STUs, or dampers, and shear keys. For one-directional bearings, these R-factors are used in the restrained direction only. In general, forces determined on the basis of plastic hinging will be less than those given by using Table 2, resulting in a more economical design.

Table 3.10.7.1-1 Response Modification Factors—Substructures.

Substructure	Importance Category		
	Critical	Essential	Other
Wall-type piers—larger dimension	1.5	1.5	2.0
Reinforced concrete pile bents			
• Vertical piles only	1.5	2.0	3.0
• With batter piles	1.5	1.5	2.0
Single columns	1.5	2.0	3.0
Steel or composite steel and concrete pile bents			
• Vertical pile only	1.5	3.5	5.0
• With batter piles	1.5	2.0	3.0
Multiple column bents	1.5	3.5	5.0

Table 3.10.7.1-2 Response Modification Factors—Connections.

Connection	All Importance Categories
Superstructure to abutment	0.8
Expansion joints within a span of the superstructure	0.8
Columns, piers, or pile bents to cap beam or superstructure	1.0
Columns or piers to foundations	1.0

3.10.7.2 Application

Seismic loads shall be assumed to act in any lateral direction.

The appropriate R-factor shall be used for both orthogonal axes of the substructure.

A wall-type concrete pier may be analyzed as a single column in the weak direction if all the provisions for columns, as specified in Section 5, are satisfied.

C3.10.7.2

Usually the orthogonal axes will be the longitudinal and transverse axes of the bridge. In the case of a curved bridge, the longitudinal axis may be the chord joining the two abutments.

Wall-type piers may be treated as wide columns in the strong direction, provided the appropriate R-factor in this direction is used.

3.10.8 Combination of Seismic Force Effects

The elastic seismic force effects on each of the principal axes of a component resulting from analyses in the two perpendicular directions shall be combined to form two load cases as follows:

- 100 percent of the absolute value of the force effects in one of the perpendicular directions combined with 30 percent of the absolute value of the force effects in the second perpendicular direction, and

- 100 percent of the absolute value of the force effects in the second perpendicular direction combined with 30 percent of the absolute value of the force effects in the first perpendicular direction.

Where foundation and/or column connection forces are determined from plastic hinging of the columns specified in Article 3.10.9.4.3, the resulting force effects may be determined without consideration of combined load cases specified herein. For the purpose of this provision, "column connection forces" shall be taken as the shear and moment, computed on the basis of plastic hinging. The axial load shall be taken as that resulting from the appropriate load combination with the axial load, if any, associated with plastic hinging taken as *EQ*. If a pier is designed as a column as specified in Article 3.10.7.2, this exception shall be taken to apply for the weak direction of the pier where force effects resulting from plastic hinging are used; the combination load cases specified must be used for the strong direction of the pier.

C3.10.8

The exception to these load combinations indicated at the end of this section should also apply to bridges in Zone 2 where foundation forces are determined from plastic hinging of the columns.

3.10.9 Calculation of Design Forces

3.10.9.1 General

For single-span bridges, regardless of seismic zone, the minimum design connection force effect in the restrained direction between the superstructure and the substructure shall not be less than the product of the site coefficient, the acceleration coefficient, and the tributary permanent load.

Seat widths at expansion bearings of multispan bridges shall either comply with Article 4.7.4.4 or STUs, and dampers shall be provided.

C3.10.9.1

This Article refers to superstructure effects carried into substructure. Abutments on multispan bridges, but not single-span bridges, and retaining walls are subject to acceleration-augmented soil pressures as specified in Articles 3.11.4 and 11.6.5. Wingwalls on single-span structures are not fully covered at this time, and the Engineer should use judgment in this area.

3.10.9.2 Seismic Zone 1

For bridges on sites in Zone 1 where the acceleration coefficient is less than 0.025 and the soil profile is either Type I or Type II, the horizontal design connection force in the restrained directions shall not be taken to be less than 0.1 times the vertical reaction due to the tributary permanent load and the tributary live loads assumed to exist during an earthquake.

For all other sites in Zone 1, the horizontal design connection force in the restrained directions shall not be taken to be less than 0.2 times the vertical reaction due to the tributary permanent load and the tributary live loads assumed to exist during an earthquake.

For each uninterrupted segment of a superstructure, the tributary permanent load at the line of fixed bearings, used to determine the longitudinal connection design force, shall be the total permanent load of the segment.

If each bearing supporting an uninterrupted segment or simply supported span is restrained in the transverse direction, the tributary permanent load used to determine the connection design force shall be the permanent load reaction at that bearing.

Each elastomeric bearing and its connection to the masonry and sole plates shall be designed to resist the horizontal seismic design forces transmitted through the bearing. For all bridges in Seismic Zone 1 and all single-span bridges, these seismic shear forces shall not be less than the connection force specified herein.

C3.10.9.2

These provisions arise because, as specified in Article 4.7.4, seismic analysis for bridges in Zone 1 is not generally required. These default values are used as minimum design forces in lieu of rigorous analysis. The division of Zone 1 at an acceleration coefficient 0.025 for sites with favorable soil condition is an arbitrary expedience intended to provide some relief to parts of the country with very low seismicity.

If each bearing supporting a continuous segment or simply supported span is an elastomeric bearing, there are no restrained directions due to the flexibility of the bearings.

The magnitude of live load assumed to exist at the time of the earthquake should be consistent with the value of γ_{eq} used in conjunction with Table 3.4.1-1.

3.10.9.3 Seismic Zone 2

Structures in Seismic Zone 2 shall be analyzed according to the minimum requirements specified in Articles 4.7.4.1 and 4.7.4.3.

Except for foundations, seismic design forces for all components, including pile bents and retaining walls, shall be determined by dividing the elastic seismic forces, obtained from Article 3.10.8, by the appropriate response modification factor, R, specified in Table 3.10.7.1-1.

Seismic design forces for foundations, other than pile bents and retaining walls, shall be determined by dividing elastic seismic forces, obtained from Article 3.10.8, by half of the response modification factor, R, from Table 3.10.7.1-1, for the substructure component to which it is attached. The value of $R/2$ shall not be taken as less than 1.0.

Where a group load other than EXTREME EVENT I, specified in Table 3.4.1-1, governs the design of columns, the possibility that seismic forces transferred to the foundations may be larger than those calculated using the procedure specified above, due to possible overstrength of the columns, shall be considered.

C3.10.9.3

This Article specifies the design forces for foundations which include the footings, pile caps and piles. The design forces are essentially twice the seismic design forces of the columns. This will generally be conservative and was adopted to simplify the design procedure for bridges in Zone 2. However, if seismic forces do not govern the design of columns and piers there is a possibility that during an earthquake the foundations will be subjected to forces larger than the design forces. For example, this may occur due to unintended column overstrengths which may exceed the capacity of the foundations. An estimate of this effect may be found by using a resistance factor, ϕ, of 1.3 for reinforced concrete columns and 1.25 for structural steel columns. It is also possible that even in cases when seismic loads govern the column design, the columns may have insufficient shear strength to enable a ductile flexural mechanism to develop, but instead allow a brittle shear failure to occur. Again, this situation is due to potential overstrength in the flexural capacity of columns and could possibly be prevented by arbitrarily increasing the column design shear by the overstrength factor cited above.

Conservatism in the design, and in some cases underdesign, of foundations and columns in Zone 2 based on the simplified procedure of this Article has been widely debated (*Gajer and Wagh 1994*). In light of the above discussion, it is recommended that for critical or essential bridges in Zone 2 consideration should be given to the use of the forces specified in Article 3.10.9.4.3f for foundations in Zone 3 and Zone 4. Ultimate soil and pile strengths are to be used with the specified foundation seismic design forces.

3.10.9.4 Seismic Zones 3 and 4

3.10.9.4.1 General

Structures in Seismic Zones 3 and 4 shall be analyzed according to the minimum requirements specified in Articles 4.7.4.1 and 4.7.4.3.

The design forces of each component shall be taken as the lesser of those determined using:

- the provisions of Article 3.10.9.4.2; or
- the provisions of Article 3.10.9.4.3,

for all components of a column, column bent and its foundation and connections.

C3.10.9.4.1

In general, the design forces resulting from an R-factor and inelastic hinging analysis will be less than those from an elastic analysis. However, in the case of architecturally oversized column(s), the forces from an inelastic hinging analysis may exceed the elastic forces in which case the elastic forces may be used for that column, column bent and its connections and foundations.

3.10.9.4.2 Modified Design Forces

Modified design forces shall be determined as specified in Article 3.10.9.3, except that for foundations the R-factor shall be taken as 1.0.

C3.10.9.4.2

Acceptable damage is restricted to inelastic hinges in the columns. The foundations should, therefore, remain in their elastic range. Hence the value for the R-factor is taken as 1.0.

3.10.9.4.3 Inelastic Hinging Forces

3.10.9.4.3a General

Where inelastic hinging is invoked as a basis for seismic design, the force effects resulting from plastic hinging at the top and/or bottom of the column shall be calculated after the preliminary design of the columns has been completed utilizing the modified design forces specified in Article 3.10.9.4.2 as the seismic loads. The consequential forces resulting from plastic hinging shall then be used for determining design forces for most components as identified herein. The procedures for calculating these consequential forces for single column and pier supports and bents with two or more columns shall be taken as specified in the following Articles.

Inelastic hinges shall be ascertained to form before any other failure due to overstress or instability in the structure and/or in the foundation. Inelastic hinges shall only be permitted at locations in columns where they can be readily inspected and/or repaired. Inelastic flexural resistance of substructure components shall be determined in accordance with the provisions of Sections 5 and 6.

Superstructure and substructure components and their connections to columns shall also be designed to resist a lateral shear force from the column determined from the factored inelastic flexural resistance of the column using the resistance factors specified herein.

These consequential shear forces, calculated on the basis of inelastic hinging, may be taken as the extreme seismic forces that the bridge is capable of developing.

C3.10.9.4.3a

By virtue of Article 3.10.9.4.2, alternative conservative design forces are specified if plastic hinging is not invoked as a basis for seismic design.

In most cases, the maximum force effects on the foundation will be limited by the extreme horizontal force that a column is capable of developing. In these circumstances, the use of a lower force, lower than that specified in Article 3.10.9.4.2, is justified and should result in a more economic foundation design.

See also Appendix B3.

3.10.9.4.3b Single Columns and Piers

Force effects shall be determined for the two principal axes of a column and in the weak direction of a pier or bent as follows:

- Step 1—Determine the column overstrength moment resistance. Use a resistance factor, ϕ of 1.3 for reinforced concrete columns and 1.25 for structural steel columns. For both materials, the applied axial load in the column shall be determined using Extreme Event Load Combination I, with the maximum elastic column axial load from the seismic forces determined in accordance with Article 3.10.8 taken as *EQ*.

C3.10.9.4.3b

The use of the factors 1.3 and 1.25 corresponds to the normal use of a resistance factor for reinforced concrete. In this case, it provides an increase in resistance, i.e., overstrength. Thus, the term "overstrength moment resistance" denotes a factor resistance in the parlance of these Specifications.

- Step 2—Using the column overstrength moment resistance, calculate the corresponding column shear force. For flared columns, this calculation shall be performed using the overstrength resistances at both the top and bottom of the flare in conjunction with the appropriate column height. If the foundation of a column is significantly below ground level, consideration should be given to the possibility of the plastic hinge forming above the foundation. If this can occur, the column length between plastic hinges shall be used to calculate the column shear force.

Force effects corresponding to a single column hinging shall be taken as:

- Axial Forces—Those determined using Extreme Event Load Combination I, with the unreduced maximum and minimum seismic axial load of Article 3.10.8 taken as EQ.

- Moments—Those calculated in Step 1.

- Shear Force—That calculated in Step 2.

3.10.9.4.3c Piers with Two or More Columns

Force effects for bents with two or more columns shall be determined both in the plane of the bent and perpendicular to the plane of the bent. Perpendicular to the plane of the bent, the forces shall be determined as for single columns in Article 3.10.9.4.3b. In the plane of the bent, the forces shall be calculated as follows:

- Step 1—Determine the column overstrength moment resistances. Use a resistance factor, ϕ of 1.3 for reinforced concrete columns and 1.25 for structural steel columns. For both materials the initial axial load should be determined using the Extreme Event Load Combination I with $EQ = 0$.

- Step 2—Using the column overstrength moment resistance, calculate the corresponding column shear forces. Sum the column shears of the bent to determine the maximum shear force for the pier. If a partial-height wall exists between the columns, the effective column height should be taken from the top of the wall. For flared columns and foundations below ground level, the provisions of Article 3.10.9.4.3b shall apply. For pile bents, the length of pile above the mud line shall be used to calculate the shear force.

C3.10.9.4.3c

See Article C3.10.9.4.3b.

- Step 3—Apply the bent shear force to the center of mass of the superstructure above the pier and determine the axial forces in the columns due to overturning when the column overstrength moment resistances are developed.

- Step 4—Using these column axial forces as *EQ* in the Extreme Event Load Combination I, determine revised column overstrength moment resistance. With the revised overstrength moment resistances, calculate the column shear forces and the maximum shear force for the bent. If the maximum shear force for the bent is not within 10 percent of the value previously determined, use this maximum bent shear force and return to Step 3.

The forces in the individual columns in the plane of a bent corresponding to column hinging shall be taken as:

- Axial Forces—The maximum and minimum axial loads determined using Extreme Event Load Combination I, with the axial load determined from the final iteration of Step 3 taken as *EQ* and treated as plus and minus.

- Moments—The column overstrength moment resistances corresponding to the maximum compressive axial load specified above.

- Shear Force—The shear force corresponding to the column overstrength moment resistances specified above, noting the provisions in Step 2 above.

3.10.9.4.3d Column and Pile Bent Design Forces

Design forces for columns and pile bents shall be taken as a consistent set of the lesser of the forces determined as specified in Article 3.10.9.4.1, applied as follows:

- Axial Forces—The maximum and minimum design forces determined using Extreme Event Load Combination I with either the elastic design values determined in Article 3.10.8 taken as *EQ*, or the values corresponding to plastic hinging of the column taken as *EQ*.

- Moments—The modified design moments determined for Extreme Event Limit State Load Combination I.

C3.10.9.4.3d

The design axial forces which control both the flexural design of the column and the shear design requirements are either the maximum or minimum of the unreduced design forces or the values corresponding to plastic hinging of the columns. In most cases, the values of axial load and shear corresponding to plastic hinging of the columns will be lower than the unreduced design forces. The design shear forces are specified so that the possibility of a shear failure in the column is minimized.

When an inelastic hinging analysis is performed, these moments and shear forces are the maximum forces that can develop and, therefore, the directional load combinations of Article 3.10.8 do not apply.

- Shear Force—The lesser of either the elastic design value determined for Extreme Event Limit State Load Combination I with the seismic loads combined as specified in Article 3.10.8 and using an R-factor of 1 for the column, or the value corresponding to plastic hinging of the column.

3.10.9.4.3e Pier Design Forces

The design forces shall be those determined for Extreme Event Limit State Load Combination I, except where the pier is designed as a column in its weak direction. If the pier is designed as a column, the design forces in the weak direction shall be as specified in Article 3.10.9.4.3d and all the design requirements for columns, as specified in Section 5, shall apply. When the forces due to plastic hinging are used in the weak direction, the combination of forces, specified in Article 3.10.8, shall be applied to determine the elastic moment which is then reduced by the appropriate R-factor.

C3.10.9.4.3e

The design forces for piers specified in Article 3.10.9.4.3e are based on the assumption that a pier has low ductility capacity and no redundancy. As a result, a low R-factor of 2 is used in determining the reduced design forces, and it is expected that only a small amount of inelastic deformation will occur in the response of a pier when subjected to the forces of the design earthquake. If a pier is designed as a column in its weak direction, then both the design forces and, more importantly, the design requirements of Articles 3.10.9.4.3d and Section 5 are applicable.

3.10.9.4.3f Foundation Design Forces

The design forces for foundations including footings, pile caps and piles may be taken as either those forces determined for the Extreme Event Load Combination I, with the seismic loads combined as specified in Article 3.10.8, or the forces at the bottom of the columns corresponding to column plastic hinging as determined in Article 3.10.8.

When the columns of a bent have a common footing, the final force distribution at the base of the columns in Step 4 of Article 3.10.9.4.3c may be used for the design of the footing in the plane of the bent. This force distribution produces lower shear forces and moments on the footing because one exterior column may be in tension and the other in compression due to the seismic overturning moment. This effectively increases the ultimate moments and shear forces on one column and reduces them on the other.

C3.10.9.4.3f

The foundation design forces specified are consistent with the design philosophy of minimizing damage that would not be readily detectable. The recommended design forces are the maximum forces that can be transmitted to the footing by plastic hinging of the column. The alternate design forces are the elastic design forces. It should be noted that these may be considerably greater than the recommended design forces, although where architectural considerations govern the design of a column, the alternate elastic design forces may be less than the forces resulting from column plastic hinging.

See also the second paragraph of C3.10.9.4.3d.

3.10.9.5 Longitudinal Restrainers

Friction shall not be considered to be an effective restrainer.

Restrainers shall be designed for a force calculated as the acceleration coefficient times the permanent load of the lighter of the two adjoining spans or parts of the structure.

If the restrainer is at a point where relative displacement of the sections of superstructure is designed to occur during seismic motions, sufficient slack shall be allowed in the restrainer so that the restrainer does not start to act until the design displacement is exceeded.

Where a restrainer is to be provided at columns or piers, the restrainer of each span may be attached to the column or pier rather than to interconnecting adjacent spans.