TABLE OF CONTENTS

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SECTION 3

LOADS AND LOAD FACTORS

Commentary is opposite the text it annotates.

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 5.12.5.

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, superstructure and substructure elements, or both, 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 cyclically, 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 33 CFR 205–25.

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, either constant upon completion of construction or varying only over a long time interval.

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.

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

Roadway Width—Clear space between barriers, curbs, or both.

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, superstructure and substructure elements, or both, 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.

Transient Loads—Loads and forces that can vary over a short time interval relative to the lifetime of the structure.

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); depth of temperature gradient (in.) (C3.9.2.3) (3.12.3)
AEP	=	apparent earth pressure for anchored walls (ksf) (3.4.1)
AF	=	annual frequency of bridge element collapse (number/year) (C3.14.4)
AF_{BC}	=	the expected annual frequency of bridge collapse (C3.6.5.1)
A_{LL}	=	rectangular area at depth H (ft^2) (3.6.1.2.6b)
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)
A_S	=	peak seismic ground acceleration coefficient modified by short-period site factor (3.10.4.2)
AR	=	horizontal curve away from fixed object (C3.6.5.1)
B	=	notional slope of backfill (degrees) (3.11.5.8.1)
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)

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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 (s^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 (3.10.4.2)
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_i = inside diameter or clear span of the culvert (in.); size of the critical component of the pier in direction i where size is either the diameter of the critical circular column or the smallest cross-sectional dimension of a rectangular column (3.6.1.2.6b) (C3.6.5.1)
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)
d_c = total thickness of cohesive soil layers in the top 100 ft (3.10.3.1)
d_s = total thickness of cohesionless soil layers in the top 100 ft (3.10.3.1)
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_a = site factor for short-period range of acceleration response spectrum (3.10.3.2)
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)
F_{pga} = site factor at zero-period on acceleration response spectrum (3.10.3.2)
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); site factor for long-period range of acceleration response spectrum (3.9.5) (3.10.3.2)
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_{ACC} = major access modification factor (C3.6.5.1)
f'_c = specified compressive strength of concrete for use in design (ksi) (3.5.1)
f_G = grade modification factor (C3.6.5.1)
f_{HC} = horizontal curve radius modification factor (C3.6.5.1)
f_{LN} = lanes in one direction modification factor (C3.6.5.1)
f_{LW} = lane width modification factor (C3.6.5.1)
f_{PSL} = posted speed limit modification factor (C3.6.5.1)
G = roadway grade (percent) (C3.6.5.1)
g = gravitational acceleration (ft/s^2) (3.6.3)
H = depth of fill over culvert (ft); ultimate bridge element strength (kip); final height of retaining wall (ft); total excavation depth (ft); resistance of bridge component to a horizontal force (kip) (3.6.1.2.6b) (C3.11.1) (3.11.5.7.1) (3.14.5.4)
$H_{int\text{-}p}$ = axle interaction depth parallel to culvert span (ft) (3.6.1.2.6b)

H_{int-t} = wheel interaction depth transverse to culvert span (ft) (3.6.1.2.6b)
H_L = depth of barge head-block on its bow (ft) (3.14.14.1)
H_{n+1} = distance from base of excavation to lowermost ground anchor (ft) (3.11.5.7.1)
H_p = ultimate bridge pier resistance (kip) (3.14.5.4)
H_s = ultimate bridge superstructure resistance (kip) (3.14.5.4)
HVE_i = the heavy vehicle base encroachment frequency (C3.6.5.1)
H_1 = distance from ground surface to uppermost 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; number of cohesive soil layers in the top 100 ft (3.11.6.2) (3.10.3.1)
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)
$LLDF$ = live load distribution factor as specified in Table 3.6.1.2.6-1a (3.6.1.2.6b)
L_s = horizontal length of sloping ground behind back face of retaining wall (ft) (3.11.5.8.1)
l_t = tire patch length, 10 (in.) (3.6.1.2.6b)
l_w = live load patch length at depth H (ft) (3.6.1.2.6b)
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; number of cohesionless soil layers in the top 100 ft (3.6.1.1.2) (3.10.3.1)
N = minimum support length (in.); number of one-way passages of vessels navigating through the bridge (number/yr.) (C3.10.9.2) (3.14.5)
$\bar{N}$ = average Standard Penetration Test (SPT) blow count (blows/ft) (ASTM D1586) for the upper 100 ft of the soil profile (3.10.3.1)
$\bar{N}_{ch}$ = average Standard Penetration Test (SPT) blow count (blows/ft) (ASTM D1586) for cohesive soil layers in the upper 100 ft of the soil profile and $\bar{s}_u$ for cohesive soil layers ($PI > 20$) in the top 100 ft ($\bar{s}_u$ method) (3.10.3.1)
N_{chi} = blowcount for a cohesionless soil layer (not to exceed 100 blows/ft in the above expression) (3.10.3.1)
N_i = the site-specific adjustment factor, N_i; Standard Penetration Test blow count of a layer (not to exceed 100 blows/ft in the above expression). Note that when using Method B, $\bar{N}$ values are for cohesionless soils and cohesive soil and rock layers within the upper 100 ft where refusal is met for a rock layer, N_i should be taken as 100 blows/ft (C3.6.5.1) (3.10.3.1)
N_s = stability number (3.11.5.6)
OCR = overconsolidation ratio (3.11.5.2)
P = live load applied at surface on all interacting wheels (kip); maximum vertical force for single ice wedge (kip); load resulting from vessel impact (kip); concentrated wheel load (kip); live load intensity; point load (kip) (3.6.1.2.6b) (C3.9.5) (3.14.5.4) (C3.6.1.2.5) (C3.11.6.2) (3.11.6.1)
$P(C|HVE_i)$ = the probability of a collision given a heavy vehicle encroachment from (C3.6.5.1)
$P(Q_{CT}>R_{CPC}|C)$ = the probability of the worst-case collision force, Q_{CT}, exceeding the critical pier component capacity, R_{CPC} (C3.6.5.1)
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)

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PGA = peak seismic ground acceleration coefficient on rock (Site Class B) (3.10.2.1) (3.10.4.2)
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)
PI = plasticity index (ASTM D4318) (3.10.3.1)
P_i = offset to critical pier component in direction i in ft where the distance is from the face of the critical pier component to the closest edge of travel lane i (C3.6.5.1)
P_L = live load vertical crown pressure (ksf) (3.6.1.2.6b)
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_{CT} = worst-case collision force (kip) (C3.6.5.1)
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_{CPC} = critical pier component capacity (C3.6.5.1)
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_{DS} = horizontal response spectral acceleration coefficient at 0.2-s period modified by short-period site factor (3.10.4.2)
S_{D1} = horizontal response spectral acceleration coefficient at 1.0-s period modified by long-period site factor (3.10.4.2)
S_f = freezing index (C3.9.2.2)
S_m = shear strength of rock mass (ksf) (3.11.5.6)
S_S = horizontal response spectral acceleration coefficient at 0.2-s period on rock (Site Class B) (3.10.2.1) (3.10.4.2)
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)
$\overline{s}_u$ = average undrained shear strength in ksf (ASTM D2166 or ASTM D2850) for the upper 100 ft of the soil profile (3.10.3.1)
s_a = axle spacing (ft) (3.6.1.2.6b)
s_{ui} = undrained shear strength for a cohesive soil layer (not to exceed 5.0 ksf in the above expression) (3.10.3.1)
s_w = wheel spacing, 6.0 ft (3.6.1.2.6b)
S_1 = horizontal response spectral acceleration coefficient at 1.0-s period on rock (Site Class B) (3.10.2.1) (3.10.4.2)
T = mean daily air temperature (°F) (C3.9.2.2)
T_F = period of fundamental mode of vibration of bridge (s) (3.10.2.2)
T_{hi} = horizontal load in anchor i (kip/ft) (3.11.5.7.1)
T_m = period of vibration for mth mode (s) (3.10.4.2)
T_{max} = applied load to reinforcement in a mechanically stabilized earth wall (kip/ft) (3.11.5.8.2)

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$T_{MaxDesign}$ = 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)
TR = horizontal curve toward fixed object (C3.6.5.1)
T_S = corner period at which acceleration response spectrum changes from being independent of period to being inversely proportional to period (s) (3.10.4.2)
T_0 = reference period used to define shape of acceleration response spectrum (s) (3.10.4.2)
t = thickness of ice (ft); thickness of deck (in.) (3.9.2.2) (3.12.3)
V = design velocity of water (ft/s); design impact speed of vessel (ft/s) (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/s) (3.14.6)
V_T = vessel transit speed in the navigable channel (ft/s) (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 ft above low ground or water level (mph) (3.8.1.1)
v = highway design speed (ft/s) (3.6.3)
$\overline{v}_s$ = average shear wave velocity for the upper 100 ft of the soil profile (3.10.3.1)
W = displacement weight of vessel (tonne) (C3.14.5.1)
w = width of clear roadway (ft); width of clear pedestrian and/or bicycle bridge (ft); width of pier at level of ice action (ft); specific weight of water (kcf); moisture content (ASTM D2216) (3.6.1.1.1) (3.6.1.6) (3.9.2.2) (C3.7.3.1) (3.10.3.1)
w_t = tire patch width, 20 (in.) (3.6.1.2.6b)
w_w = live load patch width at depth H (ft) (3.6.1.2.6b)
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 × 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 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 (degrees); inclination of back of wall with respect to a vertical axis (degrees); 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)
β = safety index; nose angle in a horizontal plane used to calculate transverse ice forces (degrees); slope of backfill surface behind retaining wall; {+ for slope up from wall; − for slope down from wall} (degrees) (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} (degrees) (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)

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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 (degrees); friction angle between fill and wall (degrees); angle between the far and near corners of a footing measured from the point on the wall under consideration (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 (degrees); angle of channel turn or bend (degrees); angle between direction of stream flow and the longitudinal axis of pier (degrees) (3.11.5.3) (3.14.5.2.3) (3.7.3.2)
θ_f = friction angle between ice floe and pier (degrees) (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 (degrees) (3.11.5.4)
ϕ'_f = effective angle of internal friction (degrees) (3.11.5.2)
ϕ_r = internal friction angle of reinforced fill (degrees) (3.11.6.3)
ϕ'_s = angle of internal friction of retained soil (degrees) (3.11.5.6)

3.3.2—Load and Load Designation

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

- Permanent Loads

CR = force effects due to creep
DD = downdrag force
DC = dead load of structural components and nonstructural attachments
DW = dead load of wearing surfaces and utilities
EH = horizontal earth pressure load
EL = miscellaneous locked-in force effects resulting from the construction process, including jacking apart of cantilevers in segmental construction
ES = earth surcharge load
EV = vertical pressure from dead load of earth fill
PS = secondary forces from post-tensioning for strength limit states; total prestress forces for service limit states
SH = force effects due to shrinkage

- Transient Loads

BL = blast loading
BR = vehicular braking force
CE = vehicular centrifugal force
CT = vehicular collision force
CV = vessel collision force
EQ = earthquake load
FR = friction load

IC	=	ice load
IM	=	vehicular dynamic load allowance
LL	=	vehicular live load
LS	=	live load surcharge
PL	=	pedestrian live load
SE	=	force effect due to settlement
TG	=	force effect due to temperature gradient
TU	=	force effect due to 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:

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

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 3.4.1-1 to 3.4.1-5

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 load combinations specified in Table 3.4.1-1 at the following limit states:

- Strength I—Basic load combination relating to the normal vehicular use of the bridge without wind.
- Strength II—Load combination relating to the use of the bridge by Owner-specified special design vehicles, evaluation permit vehicles, or both without wind.
- Strength III—Load combination relating to the bridge exposed to the design wind speed at the location of the bridge.
- Strength IV—Load combination emphasizing dead load force effects in bridge superstructures.

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).

The permit vehicle should not be assumed to be the only vehicle on the bridge unless so assured by traffic control. See Article 4.6.2.2.5 regarding other traffic on the bridge simultaneously.

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

Wind load provisions in earlier editions of the specifications were based on fastest-mile wind speed measurements. The current wind load provisions are based on 3-second wind gust speed with 7 percent probability of exceedance in 50 years (mean return period of 700 years).

The Strength IV load combination shown in these specifications was not fully statistically calibrated. It does not include live load; it controls over Strength I for components with dead load to live load ratio exceeding 7.0. These are typically long span bridges. The reliability indices tend to increase with the increase in the dead load to live load ratio, albeit at slow rate for bridges with high ratios.

A study was performed by Modjeski and Masters, Inc. (2013) using the same process to calibrate Strength IV as was used to statistically calibrate the Strength I load combination. Some load combinations that still emphasized dead load force effects, but produced a more uniform reliability across the possible practical range of dead load to live load ratios, were proposed. However, except for steel trusses, the relative effect on the controlling factored design loads was small and did not warrant changing the current load combination. Trusses and other structures with high DL/LL ratios can come closer to the targeted reliability of 3.5 by using the equation $1.4DC + 1.5DW + 1.45LL$. Trusses see the largest increase in the reliability index. However, the true reliability of steel trusses, steel box girders, and concrete box girder structures may be higher than reported in this study due to:

- not having been included in the live load distribution factor study that refined the design load for more common bridge types.
- the HL93 loading being conservative for long-span bridges as discussed in Article C3.6.1.3.

This load combination is not applicable to investigation of construction stages, substructures, earth retaining systems, and bearing design.

- Strength V—Load combination relating to normal vehicular use of the bridge with wind of 80 mph velocity.

When applied with the load factor specified in Table 3.4.1-1 (i.e. 1.0), the 80 mph 3-second gust wind speed is approximately equivalent to the 100 mph fastest-mile wind used in earlier specifications applied with a load factor of 0.4. The latter was meant to be equivalent to a 55 mph fastest-mile wind applied with a load factor of 1.4.

- 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 *BL*, *CV*, *CT*, or *IC*.

The following applies to both Extreme Event I and II:

- The design objective is life safety, i.e., noncollapse of the structure. Inelastic behavior such as spalling of concrete and bending of steel members is expected. In most cases the risk does not warrant the expense of designing for elastic behavior so long as vertical-load-carrying capacity is maintained for service-level loads.
- Prior to 2015, these Specifications used a value for γ_p greater than 1.0. This practice went against the intended philosophy behind the Extreme Event Limit State. A more conservative design is attained by increasing the hazard and using ductile detailing, rather than increasing γ_p, i.e., force effects due to permanent loads.
- 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 *BL*, *EQ*, *CT*, *CV*, or *IC*. 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 *BL*, *EQ*, *CT*, *CV*, or *IC*.

- 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 70 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.

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.

When applied with the load factor specified in Table 3.4.1-1 (i.e. 1.0), the 70 mph 3-second gust wind speed is equivalent to the 100 mph fastest-mile wind used in earlier specifications applied with a load factor of 0.3. The latter was meant to be equivalent to a 55 mph fastest-mile wind applied with a load factor of 1.0.

- Service II—Load combination intended to control yielding of steel structures and slip of slip-critical connections due to vehicular live load. For structures with unique truck loading conditions, such as access roads to ports or industrial sites which might lead to a disproportionate number of permit loads, a site-specific increase in the load factor should be considered.

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. An evaluation of WIM data from 31 sites around the country (Kulicki et al., 2015) indicated that the probability of exceeding the load level specified in Table 3.4.1-1 for this limit state could be less than once every six months.

- 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.

Prior to 2014, the longitudinal analysis relating to tension in prestressed concrete superstructures was investigated using a load factor for live load of 0.8. This load factor reflects, among other things, current exclusion weight limits mandated by various jurisdictions at the time of the development of the specifications in 1993. Vehicles permitted under these limits have been in service for many years prior to 1993. It was concluded at that time that, for longitudinal loading, there is no nationwide physical evidence that these vehicles have caused cracking in existing prestressed concrete components. The 0.8 load factor was applied regardless of the method used for determining the loss of prestressing.

The calibration of the service limit states for concrete components (Wassef et al., 2014) concluded that typical components designed using the Refined Estimates of Time-dependent Losses method incorporated in the specifications in 2005, which includes the use of transformed sections and elastic gains, have a lower reliability index against flexural cracking in prestressed components than components designed using the prestress

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loss calculation method specified prior to 2005 based on gross sections and do not include elastic gains. For components designed using the currently-specified methods for instantaneous prestressing losses and the currently-specified Refined Estimates of Time-dependent Losses method, an increase in the load factor for live load from 0.8 to 1.0 was required to maintain the level of reliability against cracking of prestressed concrete components inherent in the system.

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.

Wind load for Service IV load combination in earlier specifications was based on fastest-mile wind of 100 mph applied with a load factor of 0.7. This load represents an 84 mph fastest-mile wind applied with a load factor of 1.0. This wind load was meant to result in zero tension in prestressed concrete columns for ten-year mean reoccurrence winds. The wind speed specified in Table 3.8.1.1.2-1 for Service IV limit state is a product of the following:

- The wind speed used for the Strength III load combination taken as the 3-second gust wind speed with 7 percent probability of exceedance in 50 years.
- The ratio of the 3-second gust wind speed with 7 percent probability of exceedance in 10 years and 50 years, approximately 87 percent,
- A reduction factor equal to the square root of $^{1}/_{1.4}$ or 0.845. This reduction factor is meant to reduce the resulting wind pressure by $^{1}/_{1.4}$, the traditional ratio between wind pressures used for the strength limit states and service limit states for the same wind speed.

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. In previous editions of these specifications, and in their predecessor *AASHTO LRFD Bridge Design Specifications*, the load factor for this load combination was chosen on the assumption that the maximum stress range in the random variable spectrum is twice the effective stress range caused by the Fatigue II load combination. A reassessment of fatigue live load reported in Kulicki et al. (2014) indicated that the load factors for Fatigue I and Fatigue II should be upgraded to the values now shown in Table 3.4.1-1 to reflect current truck traffic. The resulting ratio between the load factor for the two fatigue load combinations is 2.2.

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- 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 3.4.1-1. 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.

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.

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.

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 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 3.4.1-2. 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.

The earth load factor for thermoplastic culverts is set to 1.3; however, to preserve the overall safety at the same levels as historical specifications, an earth-load-installation factor is introduced later in these Specifications as part of the implementation of NCHRP Report 631. This factor may be adjusted based on field control of construction practices.

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 for Highway Bridges*, 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

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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 nonsegmental 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 strength and extreme event limit states based on the Strength I and Extreme Event I Load Combinations using γ_p for overall stability as specified in Table 3.4.1-2 and an appropriate resistance factor as specified in Article 11.6.3.7. If the externally applied load(s) contribute to the stability of the slope, the externally applied load should not be included in the load combination used for design of the slope. However, if the foundation element is designed to resist the additional load induced on the element by the slope instability, design of the slope for overall stability may include the resisting load applied to the slope by the foundation element.

The evaluation of foundation movements as defined in Article 10.5.2.2 shall be conducted in accordance with Article 10.6.2.4 using the load factor, γ_{SE}, specified in Table 3.4.1-5.

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.

are the total force effects for which the bridge and its components should be designed.

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 3.4.1-1. Load factors for *PS, CR,* and *SH* are as shown in Table 3.4.1-3. 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.

Loads on foundations due to forces caused by slope instability should be determined in accordance with Liang (2010) or Vessely, et al. (2007) and Yamasaki, et al. (2013). Illustrations of this are provided in Figures C3.4.1-1 and C3.4.1-2.

Available slope stability design procedures and programs produce a single factor of safety, *FS,* using limit equilibrium. However, because the specific location of the critical failure surface varies depending on the overall slope geometry, subsurface stratigraphy, and external loads acting on the failure mass, and since a portion of the gravity force acting on the failure mass contributes to load and a portion contributes to resistance, there is no effective way to apply a load factor directly to the loads within the soil failure mass. Hence, the load factor of 1.0 for overall stability in Table 3.4.1-2 is combined with a resistance factor that is equal to 1/*FS* as provided in Article 10.5.5.2.1 and Article 11.6.3.7. Furthermore, slope stability analysis uses limit equilibrium concepts rather than slope deformation to assess shear failure of the soil. Therefore, overall stability is conducted for strength and extreme event limit states.

If reinforcement elements are placed through the slope failure mass to improve its stability, they should be considered to be resisting elements. Appropriate resistance factors should be applied to the resistance derived from those elements. For foundations, resistance factors that should be used are as specified in Article 10.5.5.2. For soil reinforcement elements such as are normally associated with walls, resistance factors that should be used are as specified in Article 11.5.7.

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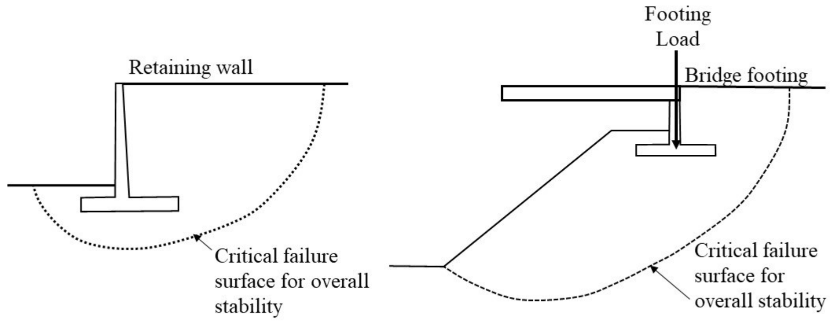

Figure C3.4.1-1—Examples Illustrating the Concept of Overall Stability as Applied to the Situation when a Slope Supports a Structural Element

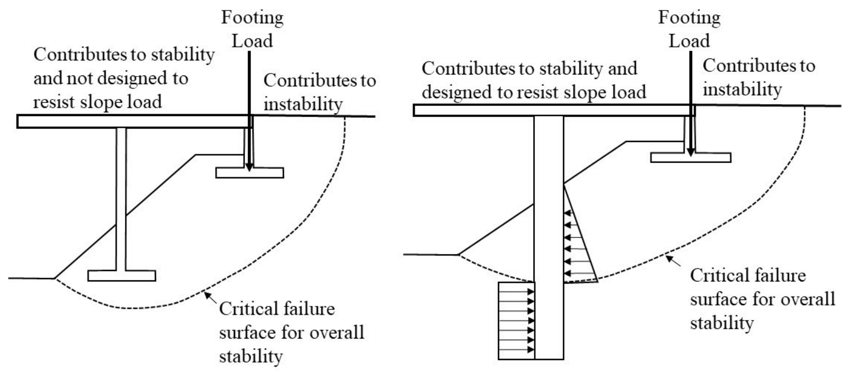

Figure C3.4.1-2—Examples Illustrating when Foundation Elements Contribute to Instability or to Stability of the Slope

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:

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

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

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- 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 effects of the foundation movements on the bridge superstructure, retaining walls, or other load bearing structures shall be evaluated at applicable strength and service limit states using the provisions of Article 10.5.2.2 and the load factor γ_{SE} specified in Table 3.4.1-5. For all bridges, stiffness should be appropriate to the considered limit state. Similarly, the effects of continuity with the substructure should be considered. In assessing the structural implications of foundation movements of concrete bridges, the determination of the stiffness of the bridge components should consider the effects of cracking, creep, and other inelastic responses.

Load combinations which include settlement shall also be applied without settlement. As specified in

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 factors specified for MSE wall internal stability are applicable to loads imparted to the soil reinforcement by vertical gravity forces from the soil backfill and surcharge. Since soil reinforcement always contributes to wall stability, no minimum load factors are specified. The magnitude of these load factors depend on the design method selected, as these load factors have been calibrated from measured data using reliability theory calibration procedures (Allen et al. 2005, Bathurst et al. 2008). The load factors for MSE wall internal stability are applicable to the Strength limit state except where specifically noted (i.e., soil failure for the Stiffness Method in Table 3.4.1-2).

The values for γ_{SE} provided in Table 3.4.1-5 are based on Samtani and Allen (2018).

For bridges in which estimated foundation total settlements are small, past typical design practice has been to not explicitly consider γ_{SE} and the factored force effect due to differential settlement, as based on long-term experience, that force effect is not likely to be a controlling factor in the design. This "legacy" design approach may be used if the geomaterials at a site are well understood, and past experience has shown that for the considered foundation and service bearing pressure, structural or geometric consequences are likely to be minor.

An Owner may choose to use a local method that provides better estimation of foundation movements for local geologic conditions compared to the methods specified in Table 3.4.1-5. In such cases, the owner will need to calibrate the γ_{SE} value for the local method using the procedures described in Kulicki, et al. (2015), Samtani and Kulicki (2018) and Samtani and Allen (2018).

The value of γ_{SE}=1.00 for consolidation (time-dependent long-term) settlement assumes that the estimation of consolidation settlement is based on appropriate laboratory and field tests to determine parameters (rather than correlations with index properties of soils) in the consolidation settlement equations in Article 10.6.2.4.3.

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Article 3.12.6, differential movements between and within substructure units shall be considered when determining the most critical combinations of force effects.

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*	*BL*	*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	1.00	—	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	1.00	1.00	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Extreme Event I	1.00	γ_{EQ}	1.00	—	—	1.00	—	—	—	1.00	—	—	—	—
Extreme Event II	1.00	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00	1.00
Service I	1.00	1.00	1.00	1.00	1.00	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	γ_{LL}	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service IV	1.00	—	1.00	1.00	—	1.00	1.00/1.20	—	1.00	—	—	—	—	—
Fatigue I—*LL*, *IM* & *CE* only	—	1.75	—	—	—	—	—	—	—	—	—	—	—	—
Fatigue II—*LL*, *IM* & *CE* only	—	0.80	—	—	—	—	—	—	—	—	—	—	—	—

Note: For Service I, the load factor for *EV* equals 1.2 for Stiffness Method Soil Failure as shown in Table 3.4.1-2.

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	Load Factor Minimum
DC: Component and Attachments		1.25	0.90
DC: Strength IV only		1.50	0.90
DD: Downdrag	Piles, α Tomlinson Method	1.40	0.25
	Piles, λ Method	1.05	0.30
	Drilled shafts, O'Neill and Reese (2010) 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 and Compound Stability		1.00	N/A
• Retaining Walls and Abutments		1.35	1.00
• MSE wall internal stability soil reinforcement loads			
o Stiffness Method			
▪ Reinforcement and connection rupture		1.35	N/A
▪ Soil failure – geosynthetics (Service I)		1.20	N/A
o Coherent Gravity Method		1.35	N/A
• Rigid Buried Structure		1.30	0.90
• Rigid Frames		1.35	0.90
• Flexible Buried Structures			
o Metal Box Culverts, Structural Plate Culverts with Deep Corrugations, and Fiberglass Culverts		1.50	0.90
o Thermoplastic Culverts		1.30	0.90
o All others		1.95	0.90
• Internal and Compound Stability for Soil Failure in Soil Nail Walls		1.00	N/A
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 3.4.1-2
Concrete Superstructures—nonsegmental	1.0	1.0
Substructures supporting non-segmental Superstructures		
• using I_g	0.5	0.5
• using $I_{effectuve}$	1.0	1.0
Steel Substructures	1.0	1.0

Table 3.4.1-4—Load Factors for Live Load for Service III Load Combination, γ_{LL}

Component	γ_{LL}
Prestressed concrete components designed using the refined estimates of time-dependent losses as specified in Article 5.9.5.4 in conjunction with taking advantage of the elastic gain	1.0
All other prestressed concrete components	0.8

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Table 3.4.1-5—Load Factors for Permanent Loads Due to Foundation Movements, γ_{SE}

Foundation Movement and Movement Estimation Method	*SE*
Immediate Settlement • Hough method • Schmertmann method • Local owner approved method	 1.00 1.40 *
Consolidation settlement	1.00
Lateral Movement • Soil-structure interaction method (P-y or Strain Wedge) • Local owner approved method	 1.00 *
*To be determined by the owner based on local geologic conditions.	

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 3.4.1-2 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 limit state load combinations in Table 3.4.1-1, modified as specified herein, shall be investigated.

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

Unless otherwise specified by the Owner, construction loads including dynamic effects (if applicable) shall be added in Strength Load Combination I with a load factor not less than 1.5 when investigating for maximum force effects.

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 loads that act on the structure only during construction. Often the construction loads are not accurately known at the time of design. Construction loads include but are not limited to the weight of materials, removable forms, personnel, and equipment such as deck finishing machines or loads applied to the structure through falsework or other temporary supports. The Owner may consider noting the construction loads assumed in the design on the contract documents. The weight of the wet

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The load factor for wind during construction in Strength Load Combination III shall be as specified by the Owner. Any applicable construction loads shall be included with a load factor not less than 1.25.

Unless otherwise specified by the Owner, primary steel superstructure components shall be investigated for maximum force effects during construction for an additional load combination consisting of the applicable *DC* loads and any construction loads that are applied to the fully erected steelwork. For this additional load combination, the load factor for *DC* and construction loads including dynamic effects (if applicable) shall not be less than 1.4.

concrete deck and any stay-in-place forms should be considered as *DC* loads.

For steel superstructures, the use of higher-strength steels, composite construction, and limit-states design approaches in which smaller factors are applied to dead load force effects than in previous service-load design approaches have generally resulted in lighter members overall.

To ensure adequate stability and strength of primary steel superstructure components during construction, an additional strength limit state load combination is specified for the investigation of loads applied to the fully erected steelwork.

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, Service Load Combination I shall apply. Except for segmentally constructed bridges, construction loads shall be added to the Service Load Combination I with a load factor of 1.00. Appropriate load combinations and allowable stresses for segmental bridges are addressed in Article 5.12.5.3. 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.

3.4.4—Load Factors for Orthotropic Decks

The Fatigue I live load factor (γ_{LL}) shall be multiplied by an additional factor of 1.3 when evaluating fatigue at the welded rib-to-floorbeam cut-out detail and the rib-to-deck weld.

C3.4.4

Evaluation of the maximum stress range in the rib-to-deck weld as well as in the vicinity of the cut-out for this type of detail has demonstrated that the use of a 1.75 load factor for *LL* is unconservative. For the rib-to-deck weld and when a cut-out is used to relive the secondary stresses imparted by the rotation of the rib relative to the floorbeam, the appropriate γ_{LL} should be increased to 2.25 (Connor, 2002). The increased Fatigue I load factor is based on stress range spectra monitoring of orthotropic decks. Studies indicate that the ratio of maximum stress range to effective stress range is increased as compared to standard bridge girders. This is due to a number of factors such as occasional heavy wheels and reduced local load distribution that occurs in deck elements. These Specifications produce a ratio that is consistent with the original findings of NCHRP Report 299 (Moses et al., 1987).

Earlier editions of these specifications used an additional factor of 1.5 that was applied to the then-current 1.5 load factor for Fatigue I resulting in an effective load factor of 2.25. The current additional factor of 1.3 results in essentially the same combined load factor when applied to the current load factor of 1.75 for Fatigue I.

3.5—PERMANENT LOADS

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

Dead loads 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 3.5.1-1, may be used for dead loads.

C3.5.1

Table 3.5.1-1 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 to 0.135
	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

Unless specified otherwise, the width of the design lanes should be taken as 12.0 ft. 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 feet between curbs, barriers, or both. 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 3.6.1.1.2-1:

- shall be used when investigating the effect of one lane loaded, and
- 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.

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 3.6.1.1.2-1 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 3.6.1.1.2-1 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

The factors specified in Table 3.6.1.1.2-1 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

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 3.6.1.1.2-1 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

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- 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; Kulicki, 2006), selected Weigh-in-Motion (WIM) data, and the 1991 Ontario Highway Bridge Design Code (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.

Earlier editions of the commentary included information about the background of the HL-93. This information can be found in Kulicki (2006).

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 3.6.1.2.2-1. 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.

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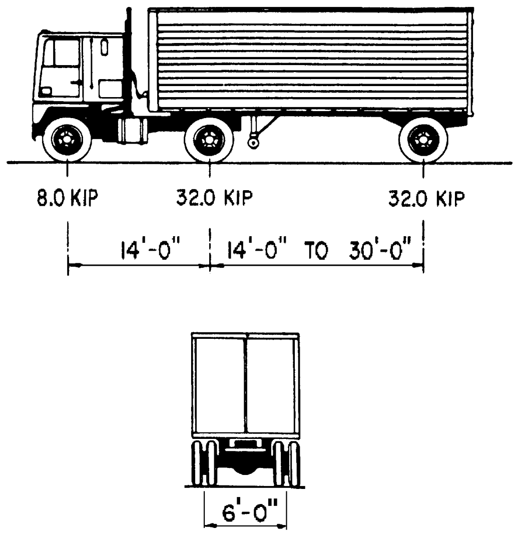

Figure 3.6.1.2.2-1—Characteristics of the Design Truck

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.

For the design of orthotropic decks and wearing surfaces on orthotropic decks, the front wheels shall be assumed to be a single rectangle whose width and length are both 10.0 in. as specified in Article 3.6.1.4.1.

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)

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3.6.1.2.6—Distribution of Wheel Load through Earth Fills

3.6.1.2.6a—General

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 inside faces of end walls.

Live load shall be distributed to the top slabs of flat top three-sided, or long-span concrete arch culverts with less than 2.0 ft of fill as specified in Article 4.6.2.10. Live load shall be distributed to concrete pipe culverts with less than 2.0 ft of cover in accordance with Eq. 4.6.2.10.2-1, regardless of the direction of travel. Round concrete culverts with 1.0 ft or more but less than 2.0 ft of cover shall be designed for a depth of 1.0 ft. Round culverts with less than 1.0 ft of fill shall be analyzed with more comprehensive methods.

Where the depth of fill over round, nonconcrete culverts is greater than 1.0 ft, or when the depth of fill over flat top three-sided, or long-span concrete arch culverts, or concrete pipe is 2.0 ft or greater the live load shall be distributed to the structure as wheel loads, uniformly distributed over a rectangular area with sides equal to the dimension of the tire contact area specified in Article 3.6.1.2.5 increased by the live load distribution factors (LLDF) specified in Table 3.6.1.2.6a-1, and the provisions of Articles 3.6.1.2.6b and 3.6.1.2.6c. More precise methods of analysis may be used.

For traffic parallel to the span, culverts shall be analyzed for a single loaded lane with the single lane multiple presence factor. For traffic perpendicular to the culvert span, analysis shall include consideration of multiple lane loadings with appropriate multiple presence factors. 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 as live load on culverts, regardless of traffic orientation.

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

C3.6.1.2.6a

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-degree 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.

Table 3.6.1.2.6a-1—Live Load Distribution Factor (LLDF) for Buried Structures

Structure Type	LLDF Transverse or Parallel to Span
Concrete Pipe with fill depth 2.0 ft or greater	1.15 for diameter 2.0 ft or less 1.75 for diameters 8.0 ft or greater Linearly interpolate for LLDF between these limits
All other culverts and buried structures	1.15

The rectangular area, A_{LL}, shall be determined as:

$$A_{LL} = l_w w_w \qquad (3.6.1.2.6a\text{-}1)$$

The term l_w and w_w shall be determined as specified in Articles 3.6.1.2.6b and 3.6.1.2.6c.

3.6.1.2.6b—Traffic Parallel to the Culvert Span

For live load distribution transverse to culvert spans, the wheel/axle load interaction depth $H_{int\text{-}t}$ shall be determined as:

$$H_{int} = \frac{s_w - \dfrac{w_t}{12} - \dfrac{0.06D_j}{12}}{LLDF} \qquad (3.6.1.2.6b\text{-}1)$$

in which:

- where $H < H_{int\text{-}t}$:

$$w_w = \frac{w_t}{12} + LLDF(H) + 0.06\frac{D_i}{12} \qquad (3.6.1.2.6b\text{-}2)$$

- where $H \geq H_{int\text{-}t}$:

$$w_w = \frac{w_t}{12} + s_w + LLDF(H) + 0.06\frac{D_i}{12} \qquad (3.6.1.2.6b\text{-}3)$$

For live load distribution parallel to culvert span, the wheel/axle load interaction depth $H_{int\text{-}p}$ shall be determined as:

$$H_{int\text{-}p} = \frac{s_a - \dfrac{l_t}{12}}{LLDF} \qquad (3.6.1.2.6b\text{-}4)$$

in which:

- where $H < H_{int\text{-}p}$:

$$l_w = \frac{l_t}{12} + LLDF(H) \qquad (3.6.1.2.6b\text{-}5)$$

C3.6.1.2.6b

The case where traffic is parallel to the culvert span applies to the vast majority of highway culverts.

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- where $H \geq H_{int}$:

$$l_w = \frac{l_t}{12} + s_a + LLDF(H) \qquad (3.6.1.2.6b\text{-}6)$$

where:

A_{LL} = rectangular area at depth H (ft²)
l_w = live load patch length at depth H (ft)
w_w = live load patch width at depth H (ft)
H_{int-t} = wheel interaction depth transverse to culvert span (ft)
s_w = wheel spacing, 6.0 ft
w_t = tire patch width, 20 (in.)
D_i = inside diameter or clear span of the culvert (in.)
$LLDF$ = live load distribution factor as specified in Table 3.6.1.2.6a-1
H = depth of fill over culvert (ft)
H_{int-p} = axle interaction depth parallel to culvert span (ft)
s_a = axle spacing (ft)
l_t = tire patch length, 10 (in.)

The live load vertical crown pressure shall be determined as:

$$P_L = \frac{P\left(1 + \frac{IM}{100}\right)(m)}{A_{LL}} \qquad (3.6.1.2.6b\text{-}7)$$

where:

P_L = live load vertical crown pressure (ksf)
P = live load applied at surface on all interacting wheels (kip)
IM = dynamic load allowance as specified in Article 3.6.2.2
m = multiple presence factor specified in Article 3.6.1.1.2
A_{LL} = rectangular area at depth H (ft²)

3.6.1.2.6c—Traffic Perpendicular to the Culvert Span

The provisions of Article 3.6.1.2.6b shall apply with the terms w_t and s_w in Eqs. 3.6.1.2.6b-1 through 3.6.1.2.6b-3 replaced by l_t and s_a respectively, and the terms l_t and s_a in Eqs. 3.6.1.2.6b-4 through 3.6.1.2.6b-6 replaced by w_t and s_w respectively.

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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
- For 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. The two design trucks shall be placed in adjacent spans to produce maximum force effects.

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.

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.

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. The design tandems should be placed in adjacent spans to produce maximum force effect. 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.

The HL-93 live load model was found to be appropriate for global analysis of long-span bridges (Nowak, 2010). In general, the design lane load portion of the HL-93 design load, which is the major contributor to live load force effects for long loaded lengths, is conservative. The conservatism is generally acceptable since members with long loaded lengths typically have much larger dead load than the live load. The conservatism could be somewhat less where the dead load has been mitigated, such as with cambered stiffening trusses on suspension bridges.

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:

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 for Highway Bridges* indicated no adverse effects of live load deflection per se. Therefore, there appears to be little reason to require that the past criteria be

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

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.

For the design of orthotropic decks and wearing surfaces on orthotropic decks, the loading pattern as shown in Figure 3.6.1.4.1-1 shall be used.

C3.6.1.4.1

For orthotropic steel decks, the governing 16.0-kip wheel loads should be modeled in more detail as two closely spaced 8.0-kip wheels 4.0 ft apart to more accurately reflect a modern tractor-trailer with tandem rear axles. Further, these wheel loads should be distributed over the specified contact area (20.0 in. wide × 10.0 in. long for rear axles and 10.0 in. square for front axles), which better approximates actual pressures applied from a dual tire unit (Kulicki and Mertz, 2006; Nowak, 2008). Note that the smaller 10.0 in. × 10.0 in. front wheels can be the controlling load for fatigue design of many orthotropic deck details.

This loading should be positioned both longitudinally and transversely on the bridge deck, ignoring the striped lanes, to create the worst stress or deflection, as applicable.

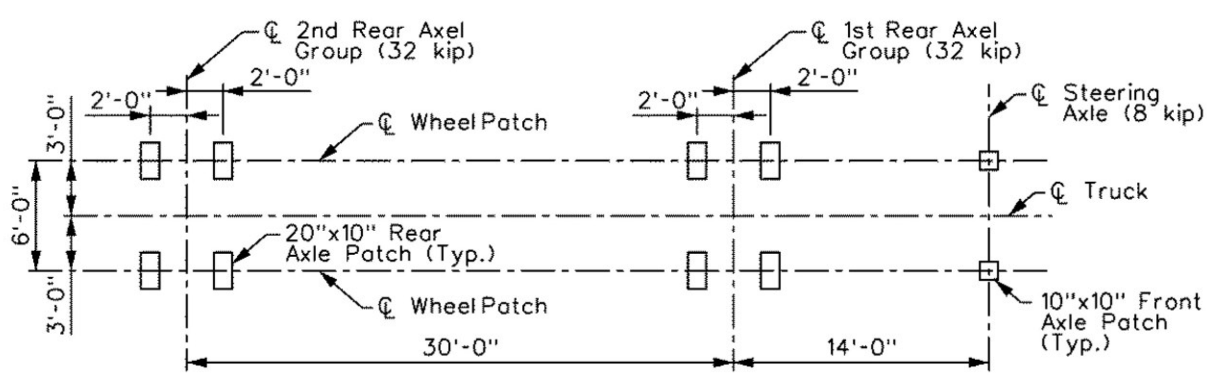

Figure 3.6.1.4.1-1—Refined Design Truck Footprint for Fatigue Design of Orthotropic Decks

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:

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.

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

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 3.6.1.4.2-1

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 yr 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 C3.6.1.4.2-1 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.

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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 1.0 ft from edge-of-deck for design of the overhang, and 2.0 ft from edge-of-deck for design of all other components. The pedestrian load shall not be considered to act concurrently with vehicles. The dynamic load allowance need not be considered for vehicles.

Bridges intended for only pedestrian, equestrian, light maintenance vehicle, and/or bicycle traffic should be designed in accordance with AASHTO's *LRFD Guide Specifications for the Design of Pedestrian Bridges*.

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.

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 3.6.2.1-1 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.

C3.6.2.1

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

The dynamic load allowance (*IM*) in Table 3.6.2.1-1 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,

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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.

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.

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 3.6.2.1-1 is the product of 4/3 and the basic 25 percent.

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)

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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:

$$C = f\frac{v^2}{gR} \qquad (3.6.3\text{-}1)$$

where:

v = highway design speed (ft/s)
f = 4/3 for load combinations other than fatigue and 1.0 for fatigue
g = gravitational acceleration: 32.2 (ft/s^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. 3.6.3-1 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/s = 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
- Five percent of the design truck plus lane load or five 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:

$$b = \frac{v^2}{2ga} \qquad (C3.6.4\text{-}1)$$

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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 s. 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 1940s 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 C3.6.4-1.

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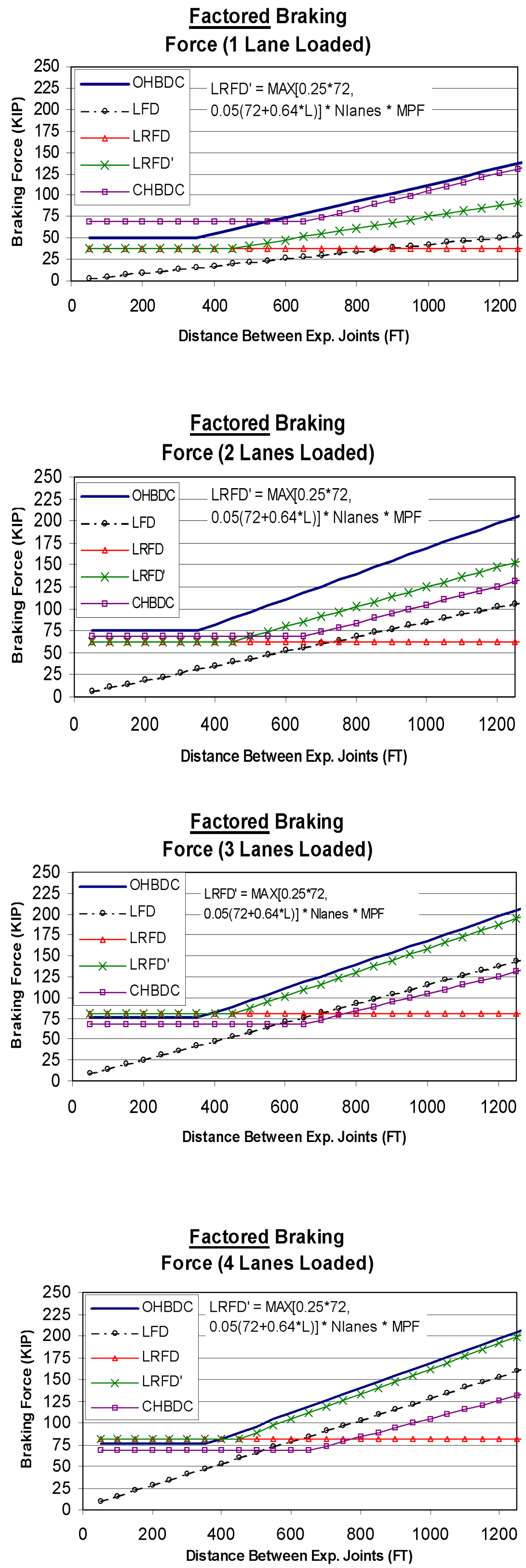

Figure C3.6.4-1—Comparison of Braking Force Models

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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 originally specified in the early versions of the LRFD Specifications (up to the 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

Unless the Owner determines that site conditions indicate otherwise, abutments and piers located within the clear zone as defined by the *AASHTO Roadside Design Guide* shall be investigated for collision. Collision shall be addressed by either providing structural resistance or by redirecting
or absorbing the collision load. The provisions of Article 2.3.2.2.1 shall apply as appropriate.

Where the design choice is to provide structural resistance, the pier or abutment shall be designed for an equivalent static force of 600 kip, which is assumed to act in a direction of zero to 15 degrees with the edge of the pavement in a horizontal plane, at a distance from 2.0 to 5.0 ft above the ground, whichever produces the critical shear or moment in the pier component and the connections to the foundation or pier cap.

Each of the following substructure components are considered to have adequate structural resistance to bridge collapse due to vehicular impacts and are not required to be designed for the vehicular impact load:

C3.6.5.1

Where an Owner chooses to make an assessment of site conditions for the purpose of implementing this provision, input from highway or highway safety engineers and structural engineers should be part of that assessment. These provisions address only protection of the pier. Guidance on occupant protection during an impact with a pier or other rigid object is included in the *AASHTO Roadside Design Guide*.

The equivalent static force of 600 kip is based on the information from full-scale crash tests of rigid columns impacted by 80.0-kip tractor trailers at 50 mph as well as finite element simulations of 80.0-kip tractor trailer trucks striking pier columns. The testing and analysis reveal that there are two distinct peak load impacts. One is associated with the engine striking the pier at a height of 2.0 ft and the other with the back of the cab / front of the trailer striking the pier at a height of 5.0 ft. For individual column shafts, the 600-kip load should be considered a point load. Field observations indicate shear failures are one important mode of failure for individual columns but field

1. Substructure components that are backed by soil (e.g., most abutments).

2. Reinforced concrete pier components that are at least 3.0-ft thick and have a concrete cross-sectional area greater than 30.0 ft^2 as measured in the horizontal plane at all elevations from the top of the pier foundation to a height of at least 5.0 ft above the grade.

3. Piers supporting a bridge superstructure where it is shown by calculation that the superstructure will not collapse with one column missing when subjected to the full dead load with a 1.1 load factor and the live load including dynamic load allowance in the permanent travel lanes with a load factor of 1.0.

4. Pier walls and multi-column piers with struts between columns that have been designed and detailed as *Manual for Assessing Safety Hardware* (MASH) Test Level 5 (TL-5) longitudinal traffic barriers according to Section 13.

Where the design choice is to redirect or absorb the collision load, protection shall consist of the following:

For new or retrofit construction, a minimum 42.0-in. high MASH crash tested rigid TL-5 barrier located such that the top edge of the traffic face of the barrier is 3.25 ft or more from the face of the pier component being protected.

Such rigid barriers shall be structurally and geometrically capable of surviving the crash test for MASH Test Level 5, as specified in Section 13.

observations have also indicated that flexural failures and failures between the column and cap or the column and footing can sometimes occur depending on the detailing. The designer should examine not only the shear capacity of the column but should also verify by calculation that the flexural capacity and connections are sufficient for the 600-kip design impact load. Columns that are 30.0 in. in diameter and smaller are the most vulnerable to impacts. 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 centered around the assumed impact point. These dimensions were determined by considering the size of a truck frame.

Requirements for train collision load found in previous editions have been removed. Designers are encouraged to consult the AREMA *Manual for Railway Engineering* or local railroad company guidelines for train collision requirements.

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

Prior guidance indicated that if pier components were located closer than 10.0 ft from the shielding barrier a 54.0 in. tall barrier should be used. Recent full-scale crash tests and finite element simulations indicate that the modern trailer suspensions result in much more stable impacts than was the case in crash tests of older trailer suspensions. If a trailer does lean over the barrier and contact a pier component, the interaction will not involve the entire mass of the vehicle and is unlikely to develop sufficient impact forces to put the pier system at risk of failure. It is preferable to allow 3.25 ft of space between the top traffic face of the barrier and the face of the nearest pier component to minimize contact with a heavy vehicle. In some retrofit situations, however, it may be impossible to provide this amount of space given the existing pier location and arrangement of the roadway. In such cases it is permissible to place the barrier closer to the face of at-risk pier components.

One way to determine whether site conditions qualify for exemption from protection is to evaluate the annual frequency of bridge collapse from impacts with heavy vehicles. With the approval of the Owner, the expected annual frequency of bridge collapse, AF_{BC}, can be calculated as follows:

1. Identify each approach direction i where a pier component is at risk of an impact from approaching traffic.

2. Calculate the annual frequency of bridge collapse as follows:

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$$AF_{BC} = \sum_{i=1}^{m} N_i \cdot HVE_i \cdot P\left(C \middle| HVE_i\right) \cdot P\left(Q_{CT} > R_{CPC} \middle| C\right)$$

(C3.6.5.1-1)

where:

N_i = the site-specific adjustment factor, N_i, computed from values in Table C3.6.5.1-1

HVE_i = the heavy vehicle base encroachment frequency from Table C3.6.5.1-2

$P(C|HVE_i)$ = the probability of a collision given a heavy vehicle encroachment from Table C3.6.5.1-3

$P(Q_{CT}>R_{CPC}|C)$ = the probability of the worst-case collision force, Q_{CT} (kip), exceeding the critical pier component capacity, R_{CPC} (kip), from Table C3.6.5.1-4

Design for vehicular collision force is not required if the annual frequency of bridge collapse, AF_{BC}, is less than 0.0001 for critical or essential bridges or 0.001 for typical bridges. If the annual frequency of bridge collapse, AF_{BC}, is greater than or equal to 0.0001 for critical or essential bridges or 0.001 for typical bridges and the pier components are not designed to resist vehicular collision forces, then the pier system must be shielded as described in Article 3.6.5.1.

The thresholds for AF_{BC} mirror those for vessel collision force found in Article 3.14.5.

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Table C3.6.5.1-1—Site Specific Adjustment Factor, N_i

Major Accesses‡			**Lane Width**			**Horizontal Curve Radius†**	
Number of Access Points within 300 ft upstream of the pier system	Undivided	Divided and One-way	Avg. Lane Width in feet	Undivided	Divided and One-way	Horizontal Curve Radius at Centerline in feet	All Highway Types
0	1.0	1.0	≤9	1.50	1.25	AR> 10,000	1.00
1	1.5	2.0	10	1.30	1.15	10,000≥AR>432	$e^{(474.4/AR)}$
≥2	2.2	4.0	11	1.05	1.03	432≥AR>0	3.00
			≥12	1.00	1.00	TR>10,000	1.00
						10,000≥TR>432	$e^{(173.6/TR)}$
						432≥TR>0	1.50
f_{ACC}=			f_{LW}=			f_{HC}=	
Lanes in One Direction			**Posted Speed Limit ¶**			**Grade Approaching the Pier System††**	
No. of Through Lanes in One Direction	Undivided	Divided and One-way	Posted Speed Limit	Undivided	Divided and One-way	Percent Grade	All Highway Types
1	1.00	1.00	<65	1.42	1.18	G≤-6	2.00
2	0.76	1.00	≥65	1.00	1.00	-6<G<-2	0.5-G/4
≥3	0.76	0.91				G≥-2	1.00
f_{LN}=			f_{PSL}=			f_G=	
$N_i = f_{ACC} \cdot f_{LN} \cdot f_{LW} \cdot f_G \cdot f_{HC} \cdot f_{PSL}$ =							

‡ Major accesses include ramps and intersections. Commercial and residential driveways should not be included as access points unless they are signalized or stop-sign controlled.

† The horizontal curve radius may either curve away (AR) from the pier system under consideration or toward it (TR). When the driver is turning the wheel of the vehicle away from the pier, the AR adjustments shall be used. When the driver is turning the wheel of the vehicle toward the pier, the TR adjustments should be used. This adjustment must be considered for each direction of travel (i) where an encroaching vehicle could approach the pier system.

†† The grade approaching the pier system must be considered for each direction of travel, i. Positive grades indicate an uphill grade and negative values indicate a downhill grade.

¶ For roads with unposted speed limits, use the adjustment for <65 mi/hr.

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Table C3.5.6.1-2—Base Annual Heavy Vehicle Encroachments in Direction *i* (HVE_i)†

Two-Way AADT	Undivided Highways							
	Percent Trucks (PT)							
veh/day	**5**	**10**	**15**	**20**	**25**	**30**	**35**	**≥40**
1,000	0.0009	0.0017	0.0019	0.0020	0.0021	0.0022	0.0022	0.0023
2,000	0.0014	0.0028	0.0031	0.0033	0.0034	0.0035	0.0036	0.0037
3,000	0.0017	0.0034	0.0038	0.0040	0.0042	0.0043	0.0044	0.0045
4,000	0.0019	0.0037	0.0041	0.0043	0.0045	0.0046	0.0048	0.0049
5,000-41,000	0.0019	0.0038	0.0042	0.0044	0.0046	0.0047	0.0048	0.0049
42,000	0.0020	0.0039	0.0043	0.0045	0.0047	0.0049	0.0050	0.0051
43,000	0.0020	0.0040	0.0044	0.0047	0.0048	0.0050	0.0051	0.0052
44,000	0.0020	0.0041	0.0045	0.0048	0.0049	0.0051	0.0052	0.0054
45,000	0.0021	0.0042	0.0046	0.0049	0.0051	0.0052	0.0054	0.0055
≥46,000	0.0021	0.0043	0.0047	0.0050	0.0052	0.0053	0.0055	0.0056
Two-Way AADT	**Divided Highways**							
	Percent Trucks (PT)							
veh/day	**5**	**10**	**15**	**20**	**25**	**30**	**35**	**≥40**
1,000	0.0006	0.0006	0.0006	0.0006	0.0007	0.0007	0.0007	0.0007
5,000	0.0026	0.0026	0.0027	0.0027	0.0028	0.0028	0.0028	0.0028
10,000	0.0042	0.0043	0.0044	0.0045	0.0045	0.0045	0.0046	0.0046
15,000	0.0051	0.0053	0.0054	0.0054	0.0055	0.0055	0.0056	0.0056
20,000	0.0055	0.0057	0.0058	0.0059	0.0060	0.0060	0.0060	0.0061
24,000-47,000	0.0056	0.0058	0.0059	0.0060	0.0061	0.0061	0.0062	0.0062
50,000	0.0060	0.0062	0.0064	0.0065	0.0065	0.0066	0.0066	0.0067
55,000	0.0066	0.0069	0.0070	0.0071	0.0072	0.0072	0.0073	0.0073
60,000	0.0072	0.0075	0.0076	0.0077	0.0078	0.0079	0.0079	0.0080
65,000	0.0078	0.0081	0.0083	0.0084	0.0085	0.0085	0.0086	0.0087
70,000	0.0084	0.0087	0.0089	0.0090	0.0091	0.0092	0.0093	0.0093
75,000	0.0090	0.0094	0.0095	0.0097	0.0098	0.0099	0.0099	0.0100
80,000	0.0096	0.0100	0.0102	0.0103	0.0104	0.0105	0.0106	0.0107
85,000	0.0102	0.0106	0.0108	0.0110	0.0111	0.0112	0.0113	0.0113
≥ 90,000	0.0108	0.0112	0.0115	0.0116	0.0117	0.0118	0.0119	0.0120

† Encroachment data is not available for one-way roadways. One-way roadways shall be evaluated using the encroachment model for divided highways where the one-way AADT value should be multiplied by 2 and used to determine HVE_i for use in the calculations.

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Table C3.6.5.1-3—Probability of a Heavy Vehicle Collision given a Heavy-Vehicle Encroachment as a Function of Pier Column Diameter or Wall Thickness and Offset from the Direction of Travel, $P(C|HVE_i)$

Offset‡ (ft)	Pier Column Size (ft)†				
	1	2	3	4	6
2	0.1763	0.1868	0.1978	0.2093	0.2337
4	0.1650	0.1750	0.1855	0.1964	0.2198
6	0.1543	0.1638	0.1738	0.1842	0.2064
8	0.1442	0.1532	0.1626	0.1725	0.1937
10	0.1347	0.1432	0.1521	0.1614	0.1816
15	0.1131	0.1204	0.1282	0.1363	0.1539
20	0.0946	0.1009	0.1075	0.1145	0.1297
25	0.0789	0.0842	0.0899	0.0958	0.1088
30	0.0656	0.0701	0.0749	0.0799	0.0910
35	0.0544	0.0582	0.0622	0.0665	0.0758
40	0.0450	0.0482	0.0515	0.0551	0.0630

$$P(C|HVE_i) = \frac{e^{-0.0398P_i+0.0709D_i-1.5331}}{1+e^{-0.0398P_i+0.0709D_i-1.5331}}$$

‡ P_i = Offset to critical pier component in direction i in ft where the distance is from the face of the critical pier component to the closest edge of travel lane i.

† D_i = Size of the critical component of the pier in direction i where size is either the diameter of the critical circular column or the smallest cross-sectional dimension of a rectangular column.

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Table C3.6.5.1-4—$P(Q_{CT}>R_{CPC}|C)$: Probability of Impact Force (Q_{CT}) Exceeding Critical Pier Component Nominal Lateral Resistance (R_{CPC})

	Rural Interstates and Primaries							Rural Collectors						
	Posted Speed Limit (mi/hr)							Posted Speed Limit (mi/hr)						
R_{CPC}	≤45	50	55	60	65	70	≥75	≤45	50	55	60	65	70	≥75
100	0.9999	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
150	0.9939	0.9989	0.9999	1.0000	1.0000	1.0000	1.0000	0.9817	0.9969	0.9993	1.0000	1.0000	1.0000	1.0000
200	0.9063	0.9629	0.9890	0.9966	0.9992	0.9996	0.9999	0.6980	0.8826	0.9609	0.9892	0.9960	0.9994	0.9998
250	0.8058	0.8422	0.9049	0.9565	0.9824	0.9935	0.9974	0.3710	0.5055	0.7018	0.8602	0.9431	0.9792	0.9930
300	0.7931	0.7928	0.8125	0.8566	0.9116	0.9533	0.9771	0.3322	0.3429	0.4023	0.5462	0.7134	0.8523	0.9283
350	0.7892	0.7884	0.7907	0.7996	0.8279	0.8684	0.9142	0.3302	0.3315	0.3350	0.3657	0.4455	0.5800	0.7291
400	0.7584	0.7832	0.7886	0.7902	0.7978	0.8079	0.8370	0.3179	0.3294	0.3300	0.3374	0.3464	0.3897	0.4873
450	0.6440	0.7550	0.7820	0.7887	0.7931	0.7914	0.7990	0.2720	0.3177	0.3280	0.3358	0.3327	0.3357	0.3622
500	0.4232	0.6620	0.7552	0.7817	0.7912	0.7894	0.7901	0.1797	0.2770	0.3163	0.3328	0.3313	0.3296	0.3360
550	0.1964	0.4754	0.6731	0.7570	0.7843	0.7879	0.7888	0.0817	0.1993	0.2837	0.3213	0.3290	0.3285	0.3323
600	0.0597	0.2628	0.5216	0.6903	0.7602	0.7810	0.7870	0.0254	0.1086	0.2163	0.2895	0.3183	0.3261	0.3313
650	0.0125	0.1054	0.3292	0.5582	0.6999	0.7584	0.7790	0.0056	0.0432	0.1397	0.2356	0.2942	0.3174	0.3287
700	0.0016	0.0312	0.1614	0.3816	0.5883	0.7076	0.7586	0.0008	0.0130	0.0657	0.1645	0.2463	0.2956	0.3193
750	0.0002	0.0067	0.0584	0.2132	0.4338	0.6144	0.7095	0.0000	0.0028	0.0253	0.0916	0.1833	0.2550	0.2998
800	0.0000	0.0008	0.0177	0.0958	0.2706	0.4781	0.6263	0.0000	0.0005	0.0070	0.0429	0.1129	0.1975	0.2666
850	0.0000	0.0001	0.0048	0.0361	0.1390	0.3246	0.5072	0.0000	0.0001	0.0016	0.0158	0.0610	0.1343	0.2167
900	0.0000	0.0000	0.0007	0.0098	0.0594	0.1934	0.3692	0.0000	0.0000	0.0003	0.0048	0.0269	0.0796	0.1571
950	0.0000	0.0000	0.0001	0.0024	0.0224	0.0988	0.2362	0.0000	0.0000	0.0001	0.0012	0.0107	0.0400	0.0998
1000	0.0000	0.0000	0.0000	0.0006	0.0065	0.0431	0.1363	0.0000	0.0000	0.0001	0.0002	0.0033	0.0165	0.0559
1050	0.0000	0.0000	0.0000	0.0000	0.0018	0.0155	0.0670	0.0000	0.0000	0.0000	0.0000	0.0010	0.0063	0.0260
1100	0.0000	0.0000	0.0000	0.0000	0.0006	0.0054	0.0285	0.0000	0.0000	0.0000	0.0000	0.0002	0.0018	0.0117
1150	0.0000	0.0000	0.0000	0.0000	0.0001	0.0015	0.0102	0.0000	0.0000	0.0000	0.0000	0.0000	0.0005	0.0042
1200	0.0000	0.0000	0.0000	0.0000	0.0000	0.0001	0.0034	0.0000	0.0000	0.0000	0.0000	0.0000	0.0002	0.0014
1250	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0011	0.0000	0.0000	0.0000	0.0000	0.0000	0.0001	0.0004
1300	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0002	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0001
	Urban Interstates and Primaries							Urban Collectors						
	Posted Speed Limit (mi/hr)							Posted Speed Limit (mi/hr)						
R_{CPC}	≤45	50	55	60	65	70	≥75	≤45	50	55	60	65	70	≥75
100	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
150	0.9924	0.9986	0.9996	0.9999	1.0000	1.0000	1.0000	0.9798	0.9961	0.9996	0.9997	0.9999	1.0000	1.0000
200	0.8599	0.9419	0.9813	0.9947	0.9987	0.9995	0.9998	0.6462	0.8638	0.9551	0.9870	0.9969	0.9990	0.9996
250	0.7093	0.7597	0.8573	0.9322	0.9743	0.9903	0.9966	0.2676	0.4239	0.6550	0.8368	0.9350	0.9763	0.9908
300	0.6915	0.6837	0.7196	0.7815	0.8663	0.9264	0.9673	0.2228	0.2396	0.3082	0.4745	0.6701	0.8248	0.9155
350	0.6876	0.6769	0.6858	0.6962	0.7394	0.7954	0.8723	0.2211	0.2260	0.2274	0.2599	0.3610	0.5123	0.6816
400	0.6622	0.6728	0.6832	0.6832	0.6890	0.7054	0.7587	0.2129	0.2245	0.2228	0.2237	0.2410	0.2901	0.3987
450	0.5611	0.6504	0.6791	0.6816	0.6826	0.6795	0.6997	0.1798	0.2166	0.2210	0.2216	0.2258	0.2295	0.2579
500	0.3724	0.5678	0.6562	0.6764	0.6812	0.6764	0.6869	0.1187	0.1887	0.2139	0.2199	0.2248	0.2223	0.2245
550	0.1718	0.4055	0.5845	0.6542	0.6758	0.6751	0.6850	0.0552	0.1377	0.1914	0.2128	0.2227	0.2211	0.2208
600	0.0513	0.2231	0.4522	0.5932	0.6544	0.6681	0.6833	0.0161	0.0742	0.1486	0.1932	0.2151	0.2191	0.2200
650	0.0110	0.0886	0.2836	0.4795	0.6024	0.6485	0.6775	0.0029	0.0284	0.0937	0.1574	0.1975	0.2134	0.2180
700	0.0010	0.0252	0.1410	0.3302	0.5068	0.6042	0.6589	0.0003	0.0079	0.0461	0.1071	0.1666	0.1998	0.2118
750	0.0002	0.0051	0.0529	0.1847	0.3724	0.5194	0.6170	0.0000	0.0019	0.0172	0.0592	0.1246	0.1741	0.1992
800	0.0000	0.0005	0.0155	0.0851	0.2344	0.4050	0.5437	0.0000	0.0003	0.0054	0.0266	0.0758	0.1356	0.1761
850	0.0000	0.0001	0.0038	0.0315	0.1200	0.2770	0.4387	0.0000	0.0000	0.0012	0.0100	0.0417	0.0924	0.1435
900	0.0000	0.0000	0.0008	0.0092	0.0529	0.1636	0.3200	0.0000	0.0000	0.0003	0.0026	0.0182	0.0554	0.1055
950	0.0000	0.0000	0.0001	0.0022	0.0184	0.0836	0.2076	0.0000	0.0000	0.0000	0.0005	0.0067	0.0279	0.0698
1000	0.0000	0.0000	0.0000	0.0003	0.0055	0.0356	0.1186	0.0000	0.0000	0.0000	0.0001	0.0018	0.0116	0.0411
1050	0.0000	0.0000	0.0000	0.0000	0.0016	0.0138	0.0584	0.0000	0.0000	0.0000	0.0000	0.0006	0.0041	0.0210
1100	0.0000	0.0000	0.0000	0.0000	0.0004	0.0042	0.0252	0.0000	0.0000	0.0000	0.0000	0.0001	0.0015	0.0088
1150	0.0000	0.0000	0.0000	0.0000	0.0000	0.0010	0.0104	0.0000	0.0000	0.0000	0.0000	0.0000	0.0004	0.0038
1200	0.0000	0.0000	0.0000	0.0000	0.0000	0.0001	0.0031	0.0000	0.0000	0.0000	0.0000	0.0000	0.0002	0.0013
1250	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0008	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0004
1300	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0002	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0001

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3.6.5.2—Vehicle Collision with Barriers

The provisions of Section 13 shall apply.

For assessing the strength and geometry of a rigid barrier used for pier component protection, Section 13 shall apply. The provisions of Article 2.3.2.2.1 shall apply as is appropriate, except if an independently supported rigid MASH TL-5 barrier is used it shall be placed such that there is at least 3.25 ft between the traffic face of the barrier and the nearest face of the pier component. Barriers used for pier protection shall be MASH tested TL-5 rigid concrete barriers.

Barriers should be placed on the site according to the recommendations of the *AASHTO Roadside Design Guide*. The MASH TL-5 rigid shielding barrier must be at least 42.0 in. tall. A minimum barrier length of 60.0 ft upstream of the leading edge of the pier system plus the entire length of the pier system shall be shielded.

C3.6.5.2

For retrofit construction, a minimum 42.0-in. high MASH crash tested rigid TL-5 barrier may be placed closer than 3.25 ft from the top edge of the traffic face of the barrier to the nearest traffic face of the pier component being protected when there is no other practical option.

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 3.7.3.1-1

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. 3.7.3.1-1 is:

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

where:

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/s)

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 degrees or less	0.8

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

w = specific weight of water (kcf)
V = velocity of water (ft/s)
g = gravitational acceleration constant—32.2 (ft/s^2)

As a convenience, Eq. 3.7.3.1-1 recognizes that $w/2g \sim 1/1{,}000$, but the dimensional consistency is lost in the simplification.

The drag coefficient, C_D, and the lateral drag coefficient, C_L, given in Tables 3.7.3.1-1 and 3.7.3.2-1, were adopted from the OHBDC (1991). The more favorable drag coefficients measured by some researchers for wedge-type pier nose angles of less than 90 degrees 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 following provisions were considered in the late 1980s for use in the New Zealand Highway Bridge Design Specification. These provisions 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 C3.7.3.1-1 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. 3.7.3.1-1, with $C_D = 0.5$. (Distances have been changed from SI.)

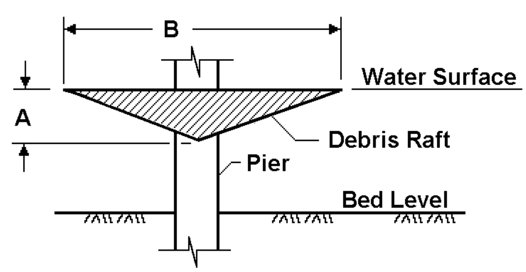

Figure C3.7.3.1-1—Debris Raft for Pier Design

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:

C3.7.3.2

The discussion of Eq. 3.7.3.1-1 also applies to Eq. 3.7.3.2-1.

$$p = \frac{C_L V^2}{1000} \qquad (3.7.3.2\text{-}1)$$

where:

p = lateral pressure (ksf)
C_L = lateral drag coefficient specified in Table 3.7.3.2-1

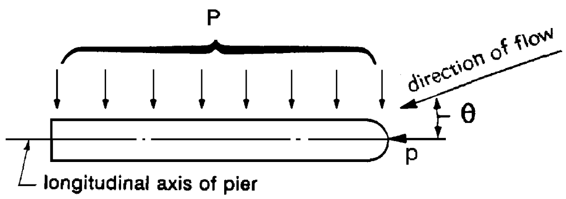

Figure 3.7.3.2-1—Plan View of Pier Showing Stream Flow Pressure

Table 3.7.3.2-1—Lateral Drag Coefficient

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

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

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 is not a force effect per se, but by changing the conditions of the substructure, it may significantly alter the consequences of force effects acting on structures.

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3.8—WIND LOAD: *WL* AND *WS*

3.8.1—Horizontal Wind Load

3.8.1.1—Exposure Conditions

3.8.1.1.1—General

Wind pressure 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, railing, and sound barriers, as seen in elevation taken perpendicular to the wind direction. The wind load shall be the product of the wind pressure and exposed area. The wind shall be assumed horizontal, except as otherwise specified in Article 3.8.2, and can come from any horizontal direction. Areas that do not contribute to the extreme force effect under consideration may be neglected in the analysis.

For typical bridges, wind loads on the substructure from the superstructure may be determined for the wind in the direction transverse to the bridge in elevation then adjusted for various angles of attack using the provisions of Article 3.8.1.2.3a.

C3.8.1.1.1

For most bridges, the same wind pressure will be used for all components of the structure and the wind load is applied as a uniformly distributed load on the entire exposed area of the structure. However, some situations may warrant using different wind pressures on different components. The most common cases are for long bridges or when the substructure is unusually tall which may warrant using different structure heights in determining the wind pressure on different portions of the superstructure or substructure.

3.8.1.1.2—Wind Speed

The design 3-second gust wind speed, *V*, used in the determination of design wind loads on bridges shall be determined from Figure 3.8.1.1.2-1. For areas designated as a special wind region in Figure 3.8.1.1.2-1, the owner shall approve the 3-second gust wind speed.

The wind speed shall be increased where records, experience, or site-specific wind studies indicate that wind speeds higher than those reflected in Figure 3.8.1.1.2-1, based upon 7 percent probability of exceedance in 50 years, are possible at the bridge location.

Unless a site-specific wind study is performed, wind speeds used for different load combinations shall be taken from Table 3.8.1.1.2-1.

Table 3.8.1.1.2-1—Design 3-Second Gust Wind Speed for Different Load Combinations, *V*

Load Combination	3-Second Gust Wind Speed (mph), *V*
Strength III	Wind speed taken from Figure 3.8.1.1.2-1
Strength V	80
Service I	70
Service IV	0.75 of the speed used for the Strength III limit state

C3.8.1.1.2

Previous editions of these Specifications were based on fastest-mile wind speed. Since that criteria was based on distance, the effect was to average the wind speed over different lengths of time. The fastest-mile wind speed is no longer utilized by modern wind codes and the provisions herein are based on 3-second gust wind speed which means that the wind speed is averaged over 3 seconds.

Figure 3.8.1.1.2-1 shows the reference 3-second gust wind speed, at an elevation of 33.0 ft, for wind exposure Category C, as defined in Article 3.8.1.1.5, with a mean recurrence interval (MRI) of 700 years. The figure is taken from ASCE 7-10 (2010).

The wind pressure determined using the design 3-second gust wind speed with a load factor of 1.0 is approximately equivalent to the wind pressure determined using fastest-mile wind speed with a MRI of 100 years and a load factor of 1.4. This is the reason the load factor for wind load on the structure, *WS*, in the Strength III load combination has been reduced from the 1.4 used in earlier editions of the specifications to the 1.0 currently shown. The change in the load factor was instituted when the design wind speed was changed from the fastest-mile wind used earlier to the design 3-second gust wind speeds shown in Figure 3.8.1.1.2-1.

The basis for the wind speeds specified for Strength V, Service I, and Service IV limit states is presented in Article C3.4.1.

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For buildings and other structures, local building officials typically determine the 3-second gust wind speed to be used in the special wind regions under their jurisdiction. Bridge Owners will need to develop their own policy for the 3-second gust wind speed to be used in these regions.

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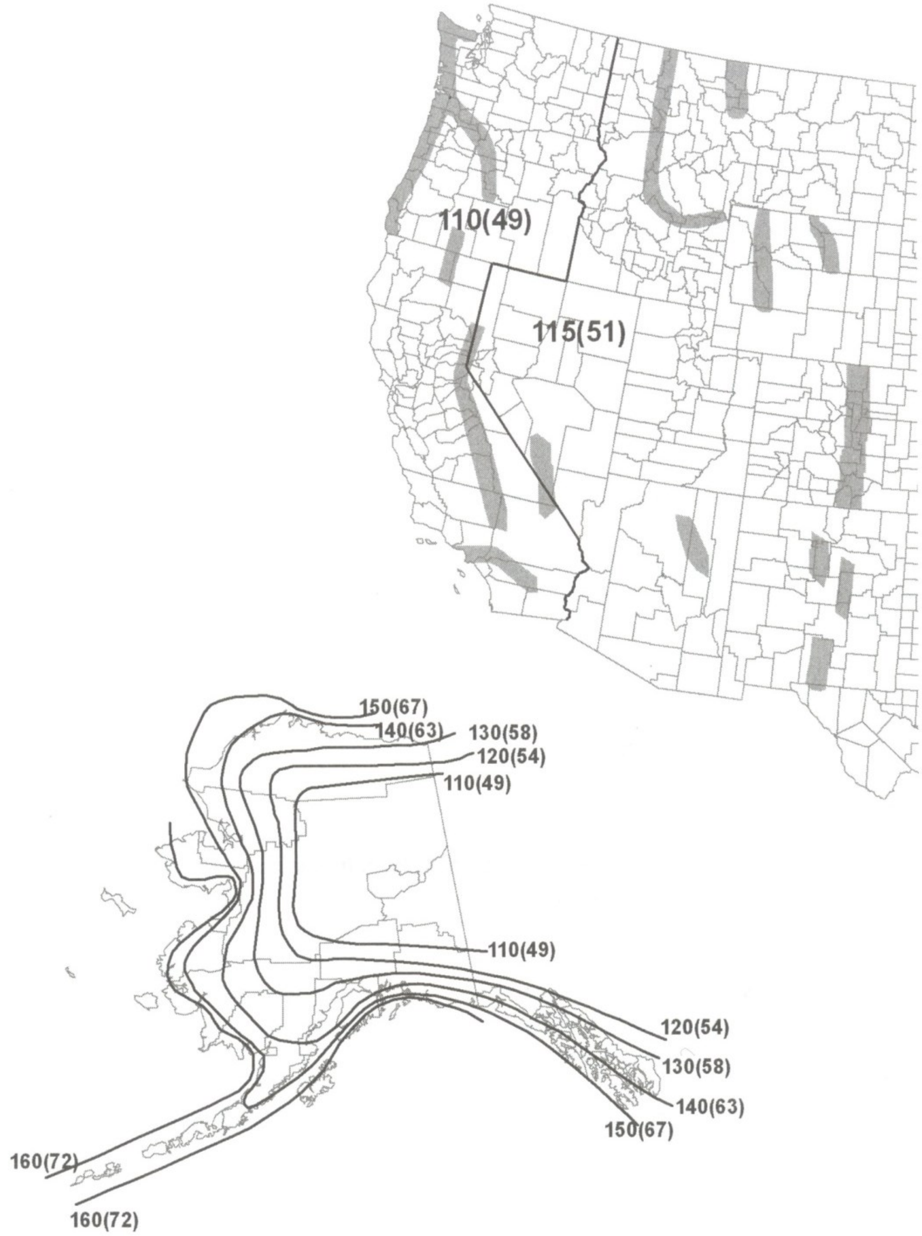

Notes:

1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00143, MRI = 700 Years).

Figure 3.8.1.1.2-1—Design Wind Speed, *V*, in mph (m/s)

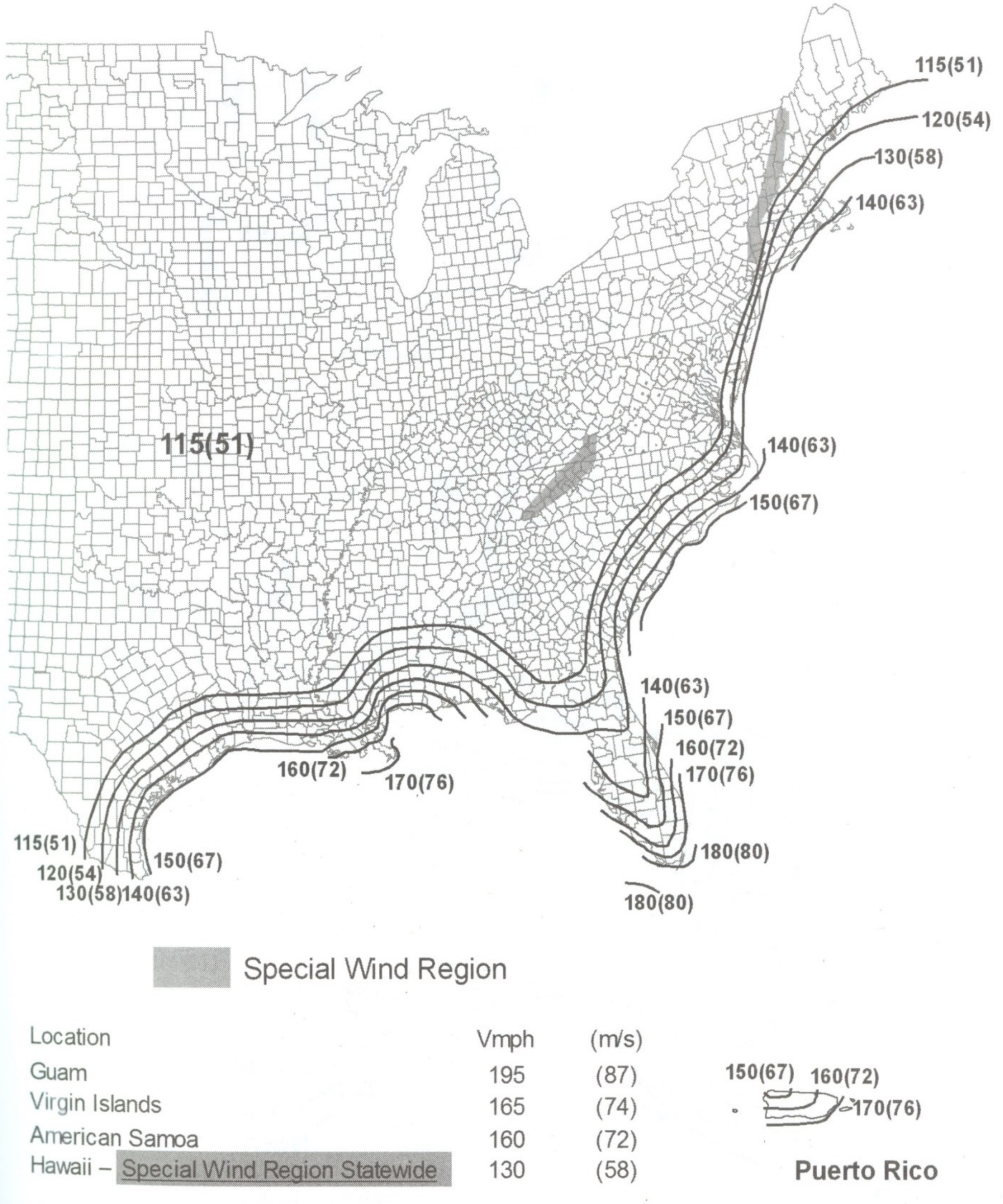

Figure 3.8.1.1.2-1 (cont'd.)—Design Wind Speed, V, in mph (m/s)

3.8.1.1.3—Wind Direction for Determining Wind Exposure Category

For each wind direction to be investigated, the wind exposure category of the bridge shall be determined for the two upwind sectors extending 45 degrees to either side of wind direction. The wind exposure category in the two sectors shall be determined in accordance with Articles 3.8.1.1.4 and 3.8.1.1.5. The wind exposure category which results in higher wind loads shall be used in determining the wind load for wind blowing from the direction being investigated.

For typical bridges, the wind exposure category as specified in Article 3.8.1.1.5 shall be perpendicular to the bridge. For long span bridges and for high level bridges, different wind exposure categories shall be investigated to determine the most critical wind direction.

C3.8.1.1.3

For a given bridge, the ground surface roughness category may be different for different wind directions based on the obstructions that exist along the direction of the wind. However, for typical bridges, the difference in wind pressure will not be significant and determining the wind exposure category perpendicular to the bridge is sufficient.

Determining various wind exposure categories may be warranted for long spans and high level bridges. In such cases, for each wind direction to be investigated, the wind exposure category of the bridge may be determined for the two upwind sectors extending 45 degrees to either side of the wind direction. The wind exposure category in the two sectors should be determined in accordance with Articles 3.8.1.1.4 and 3.8.1.1.5. The wind exposure category which results in higher wind pressure should be used in determining the wind load for wind blowing from the direction being investigated. The purpose of examining the two 45-degree sectors is to determine the maximum wind pressure associated with the wind direction being investigated.

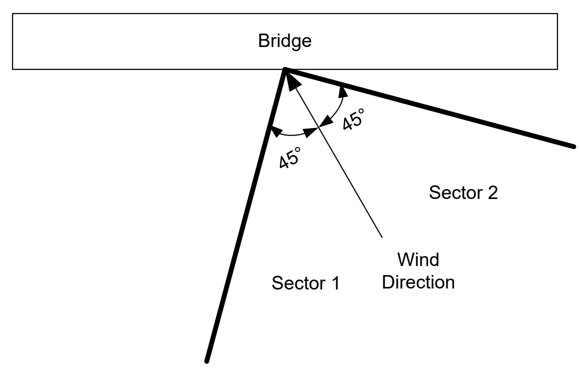

Figure C3.8.1.1.3-1—Wind Sectors for Wind from Any Direction

3.8.1.1.4—Ground Surface Roughness Categories

A ground surface roughness within each of the 45 degree sectors defined in Article 3.8.1.1.3 shall be determined as follows:

- Ground Surface Roughness B: Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger;
- Ground Surface Roughness C: Open terrain with scattered obstructions having heights generally less than 33.0 ft, including flat open country and grasslands; and

C3.8.1.1.4

The ground surface roughness categories are used in determining the wind exposure category of the structure as defined in Article 3.8.1.1.5.

The ground surface roughness categories shown herein match those in ASCE 7-10 (2010). Some of the earlier editions of ASCE 7 included a separate category for urban exposure. However, for buildings, particularly tall ones, it is thought that there is not enough urban surface roughness, even in large urban areas, to develop true urban exposure conditions. Hence, the urban exposure category was eliminated from ASCE 7. However, for a low bridge in an urban area, an urban

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- Ground Surface Roughness D: Flat, unobstructed areas and water surfaces; this category includes smooth mud flats, salt flats, and unbroken ice.

exposure could occur in nature. Using Ground Surface Roughness Category B for both urban and suburban areas results in slightly conservative wind loads for structures in urban areas.

3.8.1.1.5—Wind Exposure Categories

The exposure category of the structure shall be determined as follows:

- Wind Exposure Category B: Wind Exposure Category B shall apply where the Ground Surface Roughness Category B, as defined in Article 3.8.1.1.4, prevails in the upwind direction for a distance greater than 1,500 ft for structures with a mean height of less than or equal to 33 ft, and for a distance greater than 2,600 ft or 20 times the height of the structure, whichever is greater, for structures with a mean height greater than 33 ft.
- Wind Exposure Category C: Wind Exposure Category C shall apply for all cases where Wind Exposure Categories B or D do not apply.
- Wind Exposure Category D: Wind Exposure Category D shall apply where the Ground Surface Roughness Category D, as defined in Article 3.8.1.1.4, prevails in the upwind direction for a distance greater than 5,000 ft or 20 times the height of the structure, whichever is greater. Wind Exposure Category D shall also apply where the structure is within a distance of 600 ft or 20 times the height of the structure, whichever is greater, from a Ground Surface Roughness Category D condition, even if Ground Surface Roughness Category B or C exist immediately upwind of the structure.

C3.8.1.1.5

Where Ground Surface Roughness Category D prevails in the upwind direction—except when Ground Surface Roughness Category B or C exist for a relatively short distance immediately upwind from the structure—the effect of the presence of Ground Surface Roughness Category B or C may not be significant. Ground Surface Roughness Category D is conservatively specified for these situations.

3.8.1.2—Wind Load on Structures: *WS*

3.8.1.2.1—General

The wind pressure shall be determined as:

$$P_Z = 2.56 \times 10^{-6} V^2 K_z G C_D \qquad (3.8.1.2.1\text{-}1)$$

where:

P_Z = design wind pressure (ksf)
V = design 3-second gust wind speed specified in Table 3.8.1.1.2-1 (mph)
K_Z = pressure exposure and elevation coefficient to be taken equal to K_Z (B), K_Z (C), or K_Z (D) determined using Eqs. 3.8.1.2.1-2, 3.8.1.2.1-3, or 3.8.1.2.1-4, respectively, for Strength III and Service IV load combinations and to be taken as 1.0 for other load combinations

C3.8.1.2.1

The basis for the development of wind load provisions exists in Wassef and Raggett (2014).

For structure heights less than 33.0 ft, the proximity to the ground surface causes turbulence for which the effect on wind pressure cannot be accurately determined. Therefore, no reduction in the value of K_Z is shown in Table C3.8.1.2.1-1 for structure heights less than 33.0 ft.

Strength V and Service I load combinations are based on constant wind speeds that are not functions of the bridge type, bridge height, or the wind exposure category at the location of the bridge. Therefore, the pressure exposure and elevation coefficient, K_Z, is taken as 1.0 for these load combinations.

Unlike ASCE 7-10 (2010), which is based on power law wind profiles, these Specifications have always been based on logarithmic wind profiles. Therefore, logarithmic wind profiles were assumed in the development of Eqs. 3.8.1.2.1-2, 3.8.1.2.1-3, and 3.8.1.2.1-4.

The value of K_z at different elevations for different wind exposure categories are shown in Table C3.8.1.2.1-1.

G = gust effect factor determined using a structure-specific study or as specified in Table 3.8.1.2.1-1 for Strength III and Service IV load combinations and 1.0 for other load combinations

C_D = drag coefficient determined using a structure-specific study or as specified in Table 3.8.1.2.1-2

The gust effect factor, G, is a function of the size and dynamic characteristics of the structure including bridge natural frequency and damping. The values specified in Table 3.8.1.2.1-1 are average values for sound barriers and typical bridge structures. For long-span arches, and cable-stayed and suspension bridges, the use of wind tunnel testing to determine a project-specific gust effect factor is warranted.

The 0.85 gust effect factor specified for sound barriers in Table 3.8.1.2.1-1 is consistent with the gust effect factor in ASCE 7-10 (2010) for walls and implies that wind gusts are not likely to engulf the entire barrier. However, the loaded area required to produce the maximum wind load on a sound barrier panel and the panel's vertical supports, if used, is relatively small. A higher gust effect factor may be justifiable because wind gusts may engulf the entire panel.

Table C3.8.1.2.1-1—Pressure Exposure and Elevation Coefficients, K_Z

Structure Height, Z (ft)	Wind Exposure Category B	Wind Exposure Category C	Wind Exposure Category D
≤ 33	0.71	1.00	1.15
40	0.75	1.05	1.20
50	0.81	1.10	1.25
60	0.85	1.14	1.29
70	0.89	1.18	1.32
80	0.92	1.21	1.35
90	0.95	1.24	1.38
100	0.98	1.27	1.41
120	1.03	1.32	1.45
140	1.07	1.36	1.49
160	1.11	1.40	1.52
180	1.15	1.43	1.55
200	1.18	1.46	1.58
250	1.24	1.52	1.63
300	1.30	1.57	1.68

When the wind speed, K_Z, and G specified for Strength V and Service I load combinations are substituted in Eq. 3.8.1.2.1-1, the resulting wind pressure on bridge structures, P_Z, becomes a multiple of the drag

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coefficient, C_D, for the structure being considered. The wind pressure in these cases may be calculated using Table C3.8.1.2.1-2.

Table C3.8.1.2.1-2—Wind Pressure on the Bridge Structures for Strength V and Service I Load Combinations

Load Combination	Wind Pressure on the Structure, P_Z, for the Specified Wind Speed (ksf)
Strength V	0.0163 C_D
Service I	0.0125 C_D

The pressure exposure and elevation coefficient, K_Z, for Strength III and Service IV load combinations shall be determined as follows:

$$K_Z(B) = \frac{\left[2.5\ln\left(\dfrac{Z}{0.9834}\right) + 6.87\right]^2}{345.6} \qquad (3.8.1.2.1\text{-}2)$$

$$K_Z(C) = \frac{\left[2.5\ln\left(\dfrac{Z}{0.0984}\right) + 7.35\right]^2}{478.4} \qquad (3.8.1.2.1\text{-}3)$$

$$K_Z(D) = \frac{\left[2.5\ln\left(\dfrac{Z}{0.0164}\right) + 7.65\right]^2}{616.1} \qquad (3.8.1.2.1\text{-}4)$$

where:

$K_Z(B)$, $K_Z(C)$, and $K_Z(D)$ are K_Z for wind exposure Category B, C, and D, respectively.

The structure height, Z, used in determining the pressure exposure and elevation coefficient, K_z, shall be taken as:

- For bridge superstructures: the average height of the top of the superstructure above the surrounding ground or water surface.

In the case of a long multi-span bridge with large variation in the ground surface elevation under the bridge, such as a bridge crossing a valley, the structure height, Z, may be varied from a span to span. For each span, the structure height, Z, may be taken as the largest value in the span.

- For bridge substructures not extending above the elevation of the superstructure: unless otherwise approved by the Owner, the height used in determining the wind pressure on the superstructure.

Determining the wind pressure on substructures not extending above the elevation of the superstructure using the structure height used to determine the wind pressure on the superstructure results in slightly conservative values for most substructures. For extremely tall substructures, using a different height, including varying the height used for different segments of the substructure, may be allowed with the approval of the Owner.

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- For bridge substructures extending above the elevation of the superstructure: unless otherwise approved by the Owner, the height of the top of the substructure.

- For ground-mounted sound barriers: The height of the top of the sound barrier above the lower surrounding ground surface.

- For structure- or traffic-barrier-mounted sound barriers: The height of the top of the sound barrier above the low ground or water surface surrounding the support structure.

In no case shall the structure height, Z, used in calculating K_Z be taken less than 33.0 ft.

Substructures extending above the elevation of the superstructure are typically associated with cable-stayed bridges and suspension bridges. Wind loads on such structures are typically determined using a structure-specific wind tunnel test.

Where the sound barrier is constructed directly atop an embankment, the height of the sound barrier should be measured from the lower ground surface surrounding the embankment.

Table 3.8.1.2.1-1—Gust Effect Factor, *G*

Structure Type	Gust Effect Factor, G
Sound Barriers	0.85
All other structures	1.00

Table 3.8.1.2.1-2—Drag Coefficient, C_D

		Drag Coefficient, C_D	
Component		Windward	Leeward
I-Girder and Box-Girder Bridge Superstructures		1.3	N/A
Trusses, Columns, and Arches	Sharp-Edged Member	2.0	1.0
	Round Member	1.0	0.5
Bridge Substructure		1.6	N/A
Sound Barriers		1.2	N/A

The term “columns” in Table 3.8.1.2.1-2 refers to columns in superstructures such as spandrel columns in arches.

3.8.1.2.2—Loads on the Superstructure

In the general case of wind analysis, the wind load shall be determined as specified in Article 3.8.1.1 and the wind direction shall be varied. The wind loads shall be taken as the algebraic transverse and longitudinal components of the wind load. The wind direction for design shall be that which produces the maximum force effect in the component under investigation. The transverse and longitudinal components of the wind load shall be applied simultaneously.

C3.8.1.2.2

For superstructure components, the wind load on different members should be calculated separately and used in designing the members themselves. For trusses, the wind loads from different members and from the flooring system are transferred to the top and bottom planes of wind bracing and are used in designing the wind bracing system, including the end portals and cross-frames.

3.8.1.2.3—Loads on the Substructure

3.8.1.2.3a—Loads from the Superstructure

The transverse and longitudinal wind load components transmitted by the superstructure to the substructure for various angles of wind directions may be taken as the product of the skew coefficients specified in Table 3.8.1.2.3a-1, the wind pressure calculated using Eq. 3.8.1.2.1-1, and the depth of the bridge. The depth of the bridge shall be as seen in elevation perpendicular to the longitudinal axis of the bridge.

Both components of the wind loads shall be applied as line loads at the mid-depth of the superstructure. In plan, the longitudinal components of wind loads shall be applied as line loads along the longitudinal axis of the superstructure.

The skew angle shall be taken as measured from the perpendicular to the longitudinal axis of the bridge in plan.

The wind direction for design shall be that which produces the maximum force effect in the substructure. The transverse and longitudinal wind load components on the superstructure shall be applied simultaneously.

C3.8.1.2.3a

The Seventh Edition of *AASHTO Standard Specifications for Highway Bridges* was the first edition to incorporate the wind loads per skew angle of wind (Vincent, 1953). Alternatively, the wind load on the exposed area may be determined using an algebraic summation of transverse and longitudinal components of wind load. However, a wind directionality factor and drag coefficient other than the one specified in Article 3.8.1.2.1 should be considered.

For girder bridges, the wind pressure may be taken as one line load whose intensity is equal to the product of the wind pressure, skew coefficients, and the depth of the superstructure including the depth of the girders, deck, floor system, railing, and sound barriers, as seen in elevation perpendicular to the longitudinal axis of the bridge. For trusses, columns, and arches, the wind load is the sum of the wind loads on the windward and leeward areas.

The purpose of applying the line load along the longitudinal axis of the bridge in plan is to avoid introducing a moment in the horizontal plane of the superstructure.

Table 3.8.1.2.3a-1—Skew Coefficients for Various Skew Angles of Attack

	Trusses, Columns, and Arches		Girders	
Skew Angle (degree)	Transverse Skew Coefficient	Longitudinal Skew Coefficient	Transverse Skew Coefficient	Longitudinal Skew Coefficient
0	1.000	0.000	1.000	0.000
15	0.933	0.160	0.880	0.120
30	0.867	0.373	0.820	0.240
45	0.627	0.547	0.660	0.320
60	0.320	0.667	0.340	0.380

For usual girder and slab bridges having an individual span length of not more than 150 ft and a maximum height of 33.0 ft above low ground or water level, the following wind load components may be used:

- Transverse: 100 percent of the wind load calculated based on wind direction perpendicular to the longitudinal axis of the bridge.
- Longitudinal: 25 percent of the transverse load.

Both forces shall be applied simultaneously.

3.8.1.2.3b—Loads Applied Directly to the Substructure

The transverse and longitudinal forces to be applied directly to the substructure shall be

C3.8.1.2.3b

When combining the wind forces applied directly to the substructure with the wind forces transmitted to the calculated using the wind pressure determined using Eq. 3.8.1.2.1-1. For wind directions taken skewed to the substructure, the wind pressure shall be resolved into components perpendicular to the end and front elevations of the substructure. The component perpendicular to the end elevation shall act on the exposed substructure area as seen in end elevation, and the component perpendicular to the front elevation shall act on the exposed substructure area as seen in front elevation. The two substructure wind force components shall be applied simultaneously with the wind loads from the superstructure.

substructure from the superstructure, all wind forces should correspond to wind blowing from the same direction.

3.8.1.2.4—Wind Loads on Sound Barriers

The wind pressure on ground-mounted or structure-mounted sound barriers shall be determined using Eq. 3.8.1.2.1-1 and assuming the wind direction perpendicular to the plane of the sound barrier.

The sound barrier panels shall be designed assuming the wind pressure is applied as a uniform load to the entire area of the panels.

The vertical support elements (if used), the foundations, and the connection of the panel or the vertical support elements to the foundations or the supporting structure shall be designed for a line load equal in value to the wind pressure multiplied by the sound barrier height. The line load shall be applied at a distance equal to 0.55 times the sound barrier height measured from the bottom of the sound barrier. For determining the location of the line load, the height of the sound barrier shall be taken as the distance from the top of the sound barrier to:

- The ground surface immediately adjacent to the sound barrier for ground-mounted sound barriers.
- The elevation of the sound barrier connection to the supporting structure for structure-mounted sound barriers.

Where the sound barrier is mounted on top of a traffic railing or a retaining wall extending above ground, the magnitude and location of the wind loads transmitted to the base of the supporting traffic railing or retaining wall shall be determined as specified above, assuming that the height of the exposed area is the sum of the height of the sound barrier plus the height of the supporting railing or retaining wall.

The height of the supporting railing or retaining wall to be considered in determining the magnitude and location of the wind load shall be that measured from the top surface of the ground, bridge deck, or roadway pavement to the top of the supporting railing or retaining wall.

C3.8.1.2.4

The wind pressure is applied as a constant pressure over the entire area of the sound barrier. In reality, the wind speed and, consequently, the wind pressure, increase with the increase in height above the surrounding ground surface. Applying the wind load as a line load at a location above mid-height of the sound barrier better reflects the effect of the uneven pressure distribution.

Where the ground surface elevation is not the same in the front and in the back of a ground-mounted sound barrier, the wind forces will need to be determined for each direction as a separate case of loading. The design of all components must satisfy the demand from both cases.

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3.8.1.3—Wind Load on Live Load: *WL*

Wind load on live load shall be represented by an interruptible, moving force of 0.10 klf acting transverse to, and 6.0 ft above, the roadway and shall be transmitted to the structure.

For various angles of wind direction, the transverse and longitudinal components of the wind load on live load may be taken as specified in Table 3.8.1.3-1, with the skew angle measured from the perpendicular to the longitudinal axis of the bridge in plan.

The wind direction for design shall be that which produces the extreme force effect on the component under investigation. The transverse and longitudinal wind load components on the live load shall be applied simultaneously.

Table 3.8.1.3-1—Wind Load Components on Live Load

Skew Angle (degrees)	Transverse Component (klf)	Longitudinal Component (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

For the usual girder and slab bridges having an individual span length of not more than 150 ft and a maximum height of 33.0 ft above low ground or water level, the following wind load components on live load may be used:

- 0.10 klf, transverse
- 0.04 klf, longitudinal

Both forces shall be applied simultaneously.

C3.8.1.3

Historically, the 0.10 klf wind load has been used to determine the wind load on live loads. The value was based on a long row of randomly sequenced passenger cars, commercial vans, and trucks exposed to the maximum wind speed that vehicles can safely travel. The wind load on live load specified herein has not been changed from its value in earlier editions of these Specifications.

This horizontal live load should be applied only to the tributary areas producing a force effect of the same kind, similar to the design lane load.

3.8.2—Vertical Wind Load

The effect of forces tending to overturn structures, unless otherwise determined in Article 3.8.3, shall be calculated as a vertical upward wind load equal to:

- 0.020 ksf for Strength III load combination and
- 0.010 ksf for Service IV load combination

times the width of the deck, including parapets and sidewalks, shall be applied as a line load. This force shall be applied only when the direction of horizontal wind is taken to be perpendicular to the longitudinal axis of the bridge. This line load 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.

The vertical wind load shall not be applied for load combinations other than Strength III and Service IV.

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.

For flexible bridges, such as cable-stayed and suspension bridges, the vertical force should be checked as an upward or downward force, whichever may control the design.

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3.8.3—Wind-Induced Bridge Motions

3.8.3.1—General

The provisions of this Article shall apply to bridges in service and to bridges during construction after the deck and all features affecting their aeroelastic behavior are installed.

Force effects of wind-induced vibrations shall be taken into account in the design of bridges and structural components apt to be wind-sensitive. For the purpose of this Article, the following bridges shall be deemed to be wind-sensitive:

- all bridges with a span-to-depth ratio, and structural components thereof with a length-to-width ratio, exceeding 30.0,
- all cable-supported bridges, and
- all bridges with fundamental vertical or translational periods greater than 1 second.

The potential of wind-induced vibrations of cables, due to any causative mechanism, shall also be considered.

C3.8.3.1

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

The most common cable vibrations are due to vortex shedding, rain- or wind-induced vibrations, galloping due to the inclination of the cable to the wind, wake galloping, galloping due to aerodynamically unsymmetrical cross-section, excitation from the cable anchorage motion, and buffeting from wind turbulence.

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 wind tunnel testing are part of the existing wind tunnel state of the art and are beyond the scope of this commentary.

3.8.3.2—Wind-Induced Motions

Wind-induced vibrations due to buffeting, vortex excitation, galloping, flutter, and static divergence of wind-sensitive bridges and wind-sensitive components 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 that do not have round cross-sections. Cable stays, having circular sections, will gallop when the wind is inclined to

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the axis of the cable, and when 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

For wind-sensitive bridges, peak vertical wind-induced accelerations of the superstructure due to vortex shedding or buffeting should be less than 5 percent of the acceleration of gravity, *g*, for steady wind speeds less than or equal to 30 mph, and should be less than 10 percent of the acceleration of gravity, *g*, for steady wind speeds greater than 30 mph and less than or equal to 50 mph. Wind-sensitive bridges and wind-sensitive 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, galloping, and catastrophic flutter up to a steady, 10-minute averaged wind speed numerically equal to 0.85 times the design wind speed applicable at the completed bridge at the superstructure elevation.

For the purpose of determining the 10-minute averaged wind speed, the design wind speed applicable for the completed bridge at the superstructure elevation shall be taken equal to $V(K_z)^{1/2}$, for which V and K_z are as defined in Article 3.8.1.2.1.

C3.8.3.3

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

The 5 percent and 10 percent of the acceleration of gravity, *g*, for winds below 30 mph and winds between 30 mph and 50 mph, respectively, are typical limits for the motion criteria for pedestrian comfort. They were used successfully in the past in the design of vehicular flexible bridge systems such as cable-stayed and suspension bridges. For higher wind speeds, strength considerations, not motion considerations, govern the design.

The specified 10-minute averaged wind speed, numerically equal to 0.85 $V(K_z)^{1/2}$, is that with an approximate mean recurrence interval of 10,000 years. Since catastrophic flutter vibrations take some time to develop, a 10-minute averaged wind speed is used to evaluate the stability of the bridge.

3.8.4—Site-Specific and Structure-Specific Studies

The requirements of Article 3.8.3 may be satisfied using:

- a site-specific analysis of historical wind data in nonhurricane areas and a site-specific numerical simulation of potential hurricane wind speeds may be used to determine design wind criteria, or
- representative wind tunnel tests using approved procedures may be utilized to determine wind loads and to evaluate aeroelastic stability.

C3.8.4

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

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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 five 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/s (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, 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 breakup 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 (degrees)
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
C_n = coefficient accounting for the inclination of the pier nose with respect to a vertical

where $\alpha \leq 15$ degrees, 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.

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. 3.9.2.2-1 or 3.9.2.2-2 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:

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

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 (degrees 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 C3.9.2.3-1. 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 (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
1,000	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:

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

where:

β = nose angle in a horizontal plane for a round nose taken as 100 (degrees)
θ_f = friction angle between ice and pier nose (degrees)

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 C3.9.2.4.1-1 taken from the same source.

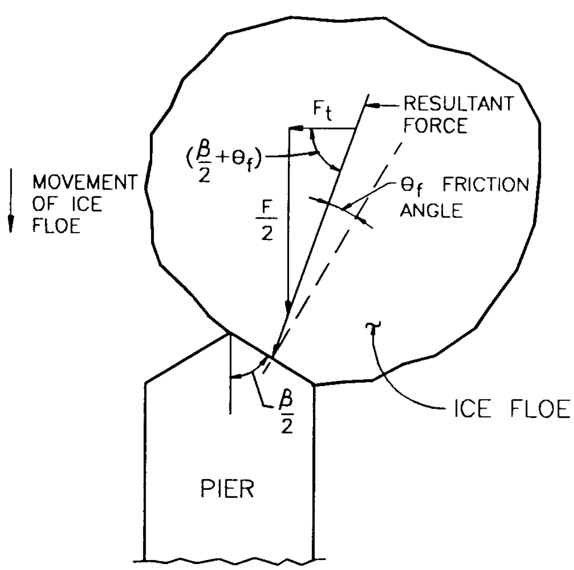

Figure C3.9.2.4.1-1—Transverse Ice Force Where a Floe Fails over a Portion of a Pier

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:

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

C3.9.5

Eq. 3.9.5-1 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).

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

For an oblong pier:

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

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:

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

where:

σ_T = tensile strength of ice (ksf)
t = maximum thickness of ice (ft)
δ = angle of the truncated wedge (degrees)
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. 3.9.5-1, the vertical force is summed for four wedges, each with a truncated angle of 90 degrees. 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. 3.9.5-2 is the sum of two expressions:

- Eq. 3.9.5-1, 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. 3.9.5-1 and 3.9.5-2 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).

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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 snowpack. 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 snowpack is subjected to freeze-thaw cycles or rain. Probable densities for several snowpack conditions are indicated in Table C3.9.6-1, ASCE (1980).

Table C3.9.6-1—Snow Density

Condition of Snowpack	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

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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 seven 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. When seismic isolation is used, the design shall be in accordance with the *Guide Specifications for Seismic Isolation Design*, unless otherwise specified by the 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, 2009), 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.

3.10.2.1—General Procedure

The General Procedure shall use the peak ground acceleration coefficient (*PGA*) and the short- and long-period spectral acceleration coefficients (S_S and S_1, respectively) to calculate the spectrum as specified in Article 3.10.4. Values of *PGA*, S_S, and S_1 shall be determined from either Figures 3.10.2.1-1 to 3.10.2.1-21 as appropriate, or from state ground motion maps approved by the Owner.

Linear interpolation shall be used for sites located between contour lines or between a contour line and a local maximum or minimum.

The effect of site class on the seismic hazard shall be as specified in Article 3.10.3.

C3.10.2.1

Values for the coefficients *PGA*, S_S, and S_1 are expressed in percent in Figures 3.10.2.1-1 to 3.10.2.1-21. Numerical values are obtained by dividing contour values by 100. Local maxima and minima are given inside the highest and lowest contour for a particular region.

The above coefficients are based on a uniform risk model of seismic hazard. The probability that a coefficient will not be exceeded at a given location during a 75-year period is estimated to be about 93 percent, i.e., a seven percent probability of exceedance. The use of a 75-year interval to characterize this probability is an arbitrary convenience and does not imply that all bridges are thought to have a useful life of 75 years.

It can be shown that an event with the above probability of exceedance has a return period of about 1,000 yr and is called the design earthquake. Larger earthquakes than that implied by the above set of coefficients have a finite probability of occurrence throughout the United States.

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Values for the ground coefficient (PGA) and the spectral coefficients (S_S and S_1) are also available on the *USGS 2007 Seismic Parameters* CD. Coefficients are given by the longitude and latitude of the bridge site, or by the ZIP code for the site.

An error has been identified in the "Spectral Response Accelerations S_{DS} and S_{D1}" results produced by the CD-ROM software. Specifically, the A_s value is erroneously calculated as $A_s = F_a\ PGA$. Although the corrected value for A_s is presented in the tabulated design spectrum table, designers should be aware of this error until the problem is corrected. The software error will likely have negligible effects on bridge analysis results because:

- F_{pga} is approximately equal to F_a,
- A_s is properly calculated and displayed in the tabulated design spectra, and
- Bridges have fundamental periods greater than the effected period range ($T_m < T_o$).

In lieu of using the national ground motion maps in Figures 3.10.2.1-1 to 3.10.2.1-21, values for the coefficients PGA, S_S, and S_1 may be derived from approved state ground motion maps. To be acceptable, the development of state maps should conform to the following:

- The definition of design ground motions should be the same as described in Articles 3.10.1 and 3.10.2.
- Ground motion maps should be based on a detailed analysis demonstrated to lead to a quantification of ground motion, at a regional scale, that is as accurate or more so, as is achieved in the national maps. The analysis should include: characterization of seismic sources and ground motion that incorporates current scientific knowledge; incorporation of uncertainty in seismic source models, ground motion models, and parameter values used in the analysis; and detailed documentation of map development.

Detailed peer review should be undertaken as deemed appropriate by the Owner. The peer review process should include one or more individuals from the U.S. Geological Survey who participated in the development of the national maps.

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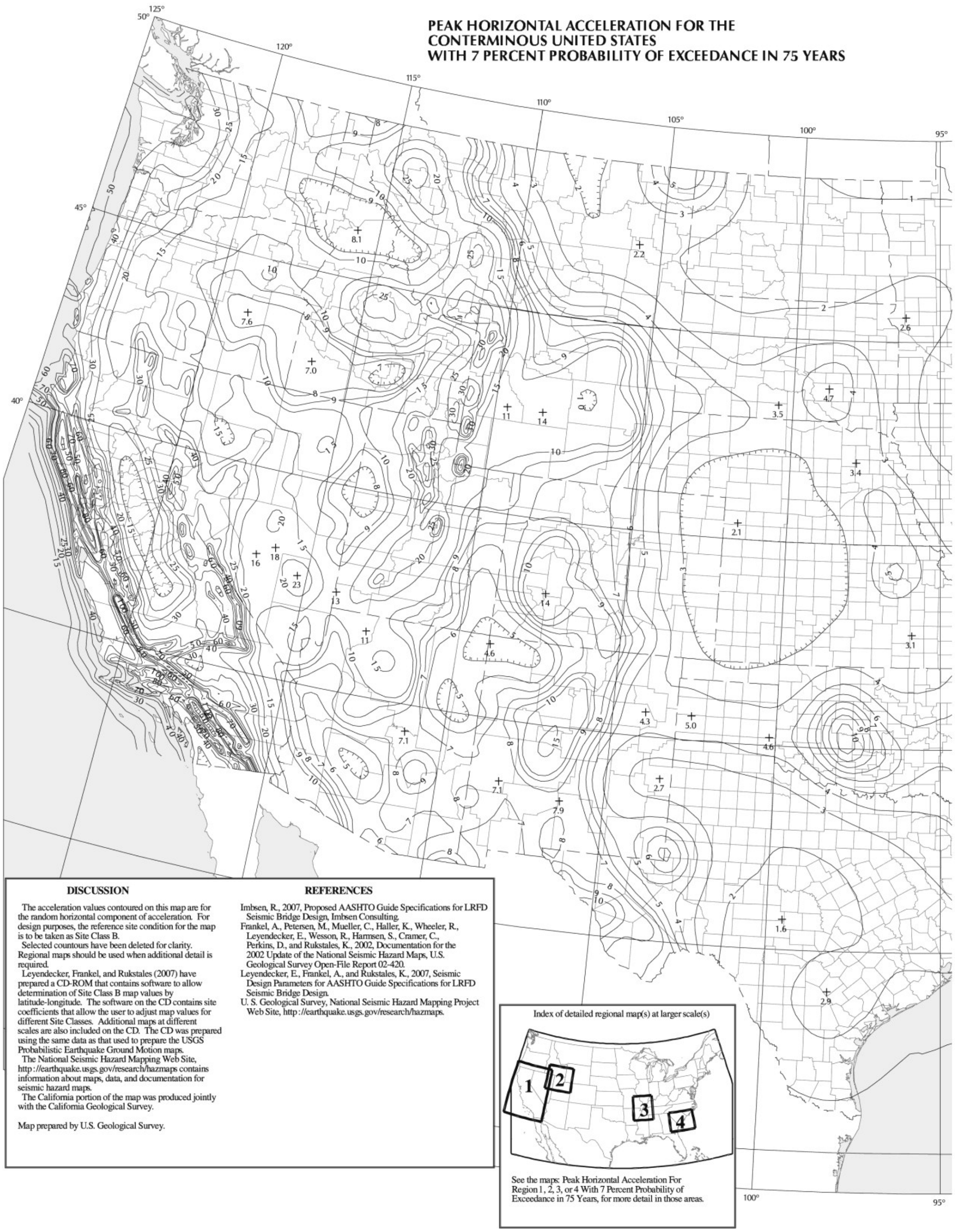


Figure 3.10.2.1-1—Horizontal Peak Ground Acceleration Coefficient for the Conterminous United States (*PGA*) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period)

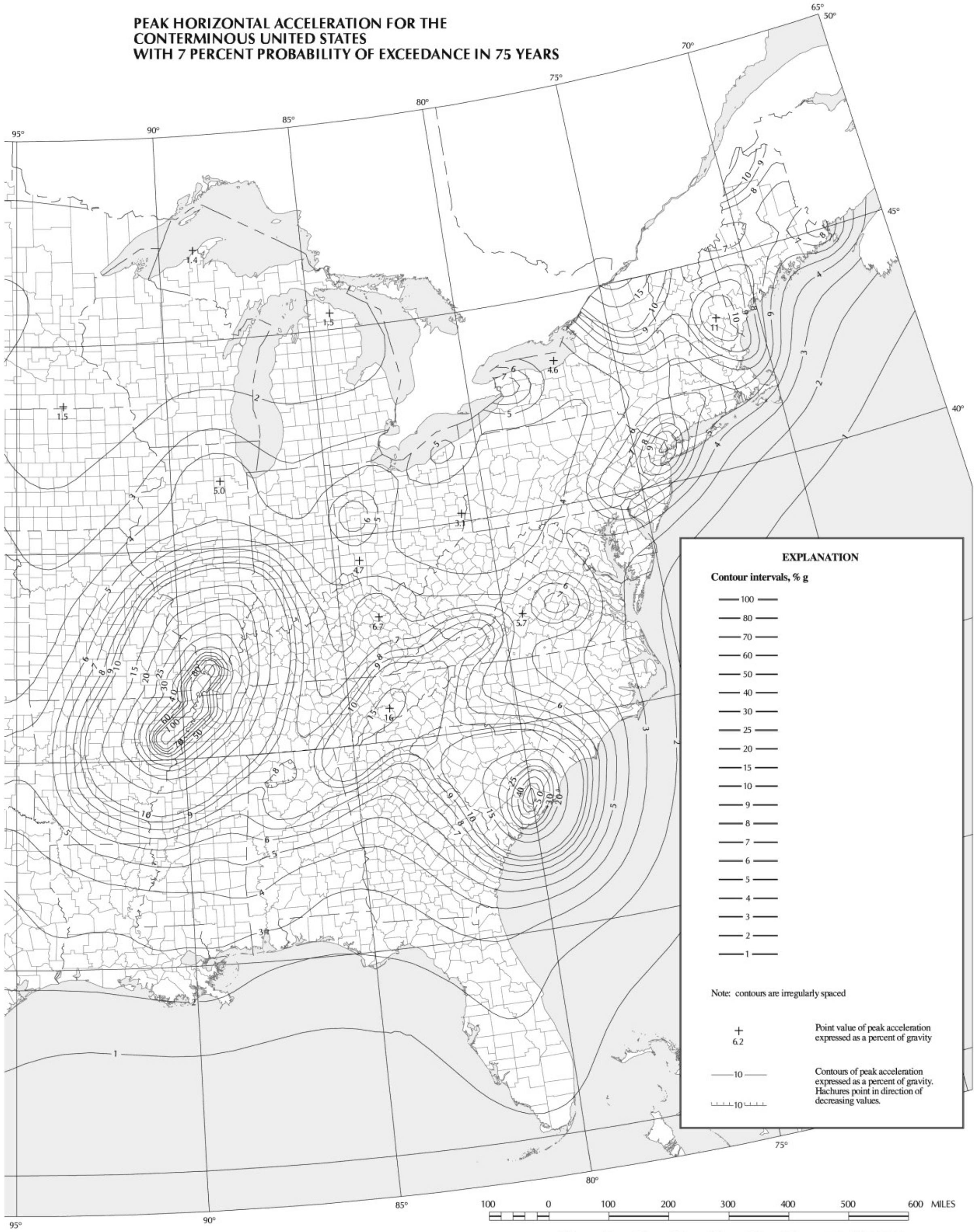


Figure 3.10.2.1-1 (continued)—Horizontal Peak Ground Acceleration Coefficient for the Conterminous United States (*PGA*) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period)

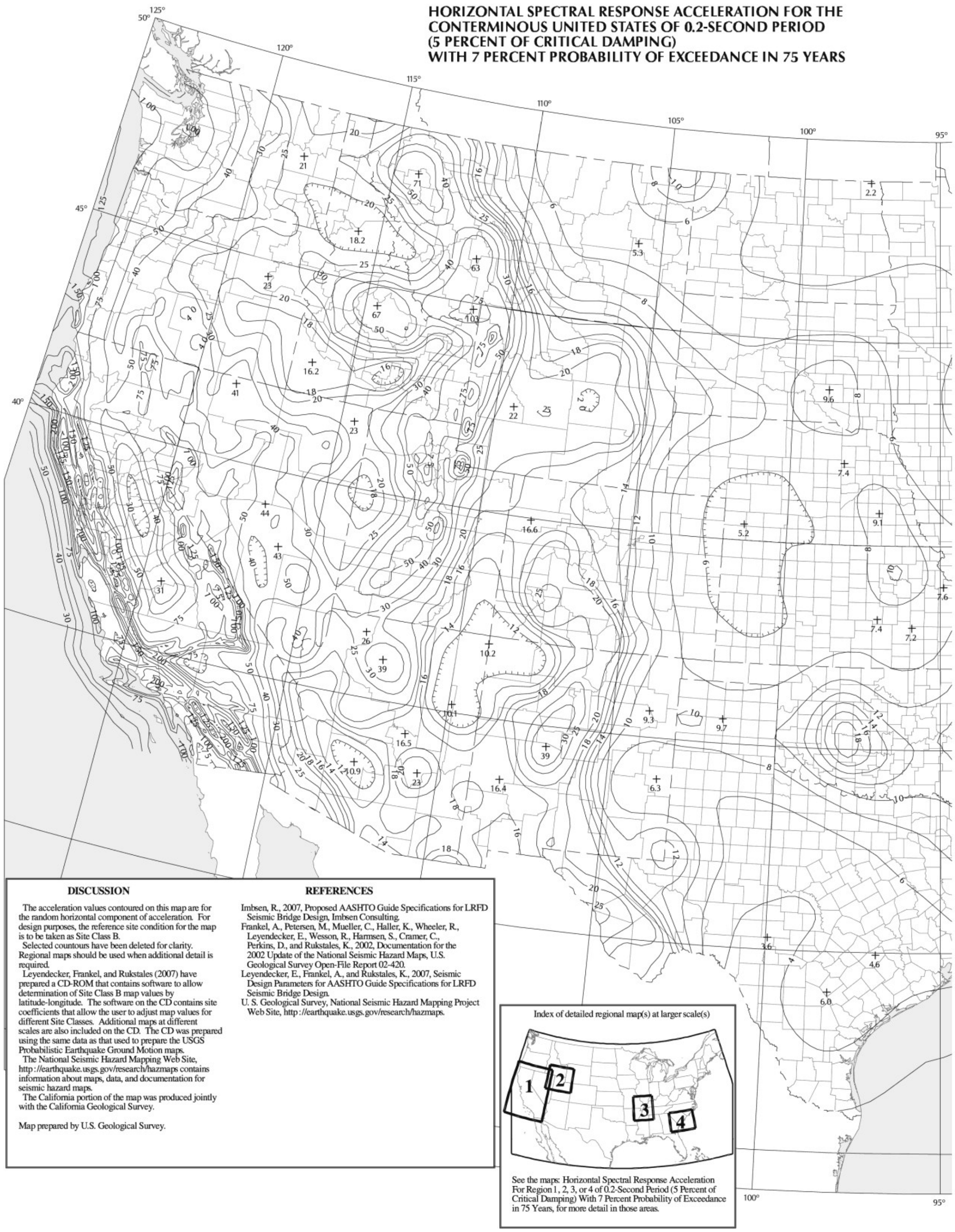


Figure 3.10.2.1-2—Horizontal Response Spectral Acceleration Coefficient for the Conterminous United States at Period of 0.2 s (S_S) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period) and Five Percent Critical Damping

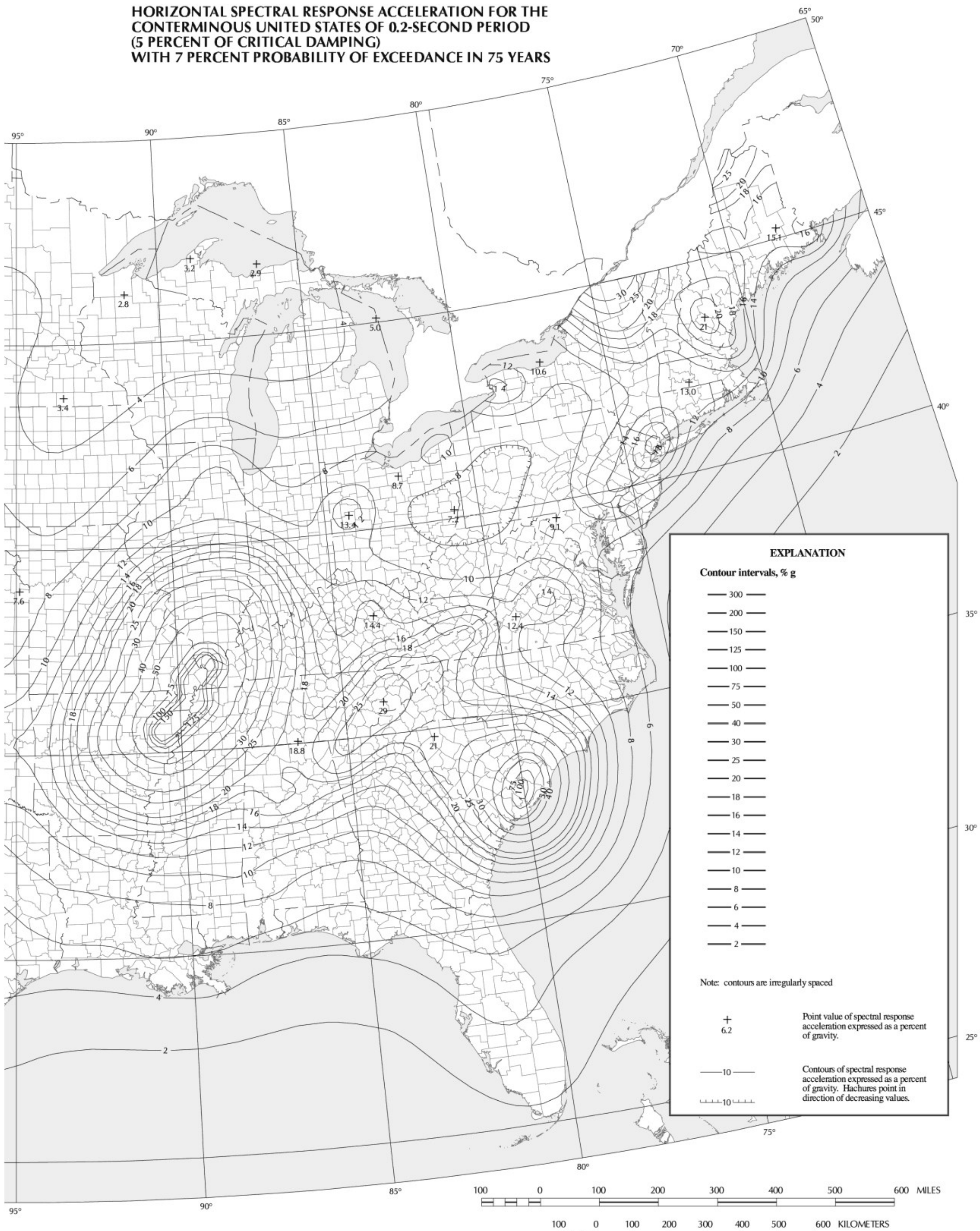


Figure 3.10.2.1-2 (continued)—Horizontal Response Spectral Acceleration Coefficient for the Conterminous United States at Period of 0.2 s (S_S) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period) and Five Percent Critical Damping

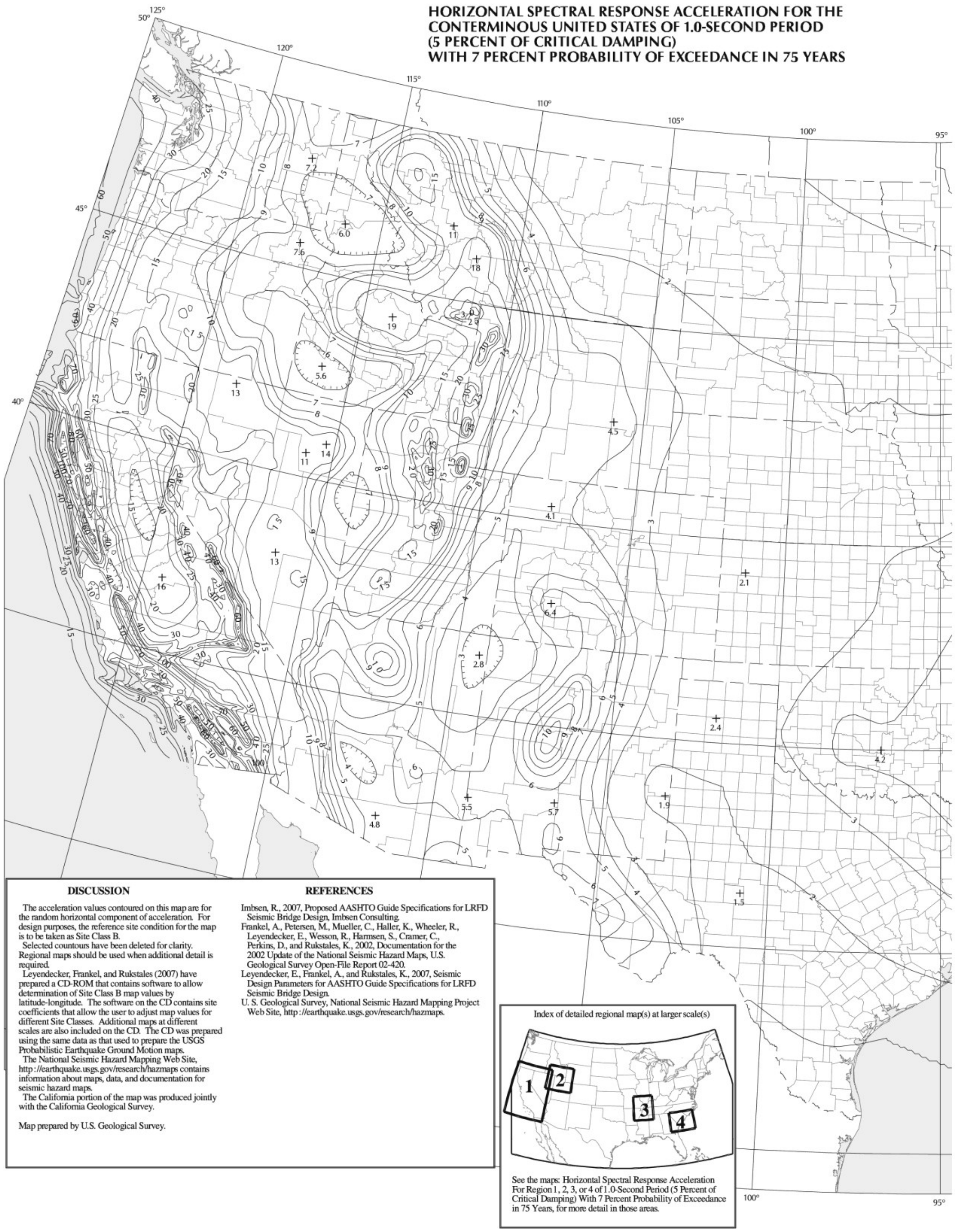


Figure 3.10.2.1-3—Horizontal Response Spectral Acceleration Coefficient for the Conterminous United States at Period of 1.0 s (S_1) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period) and Five Percent Critical Damping

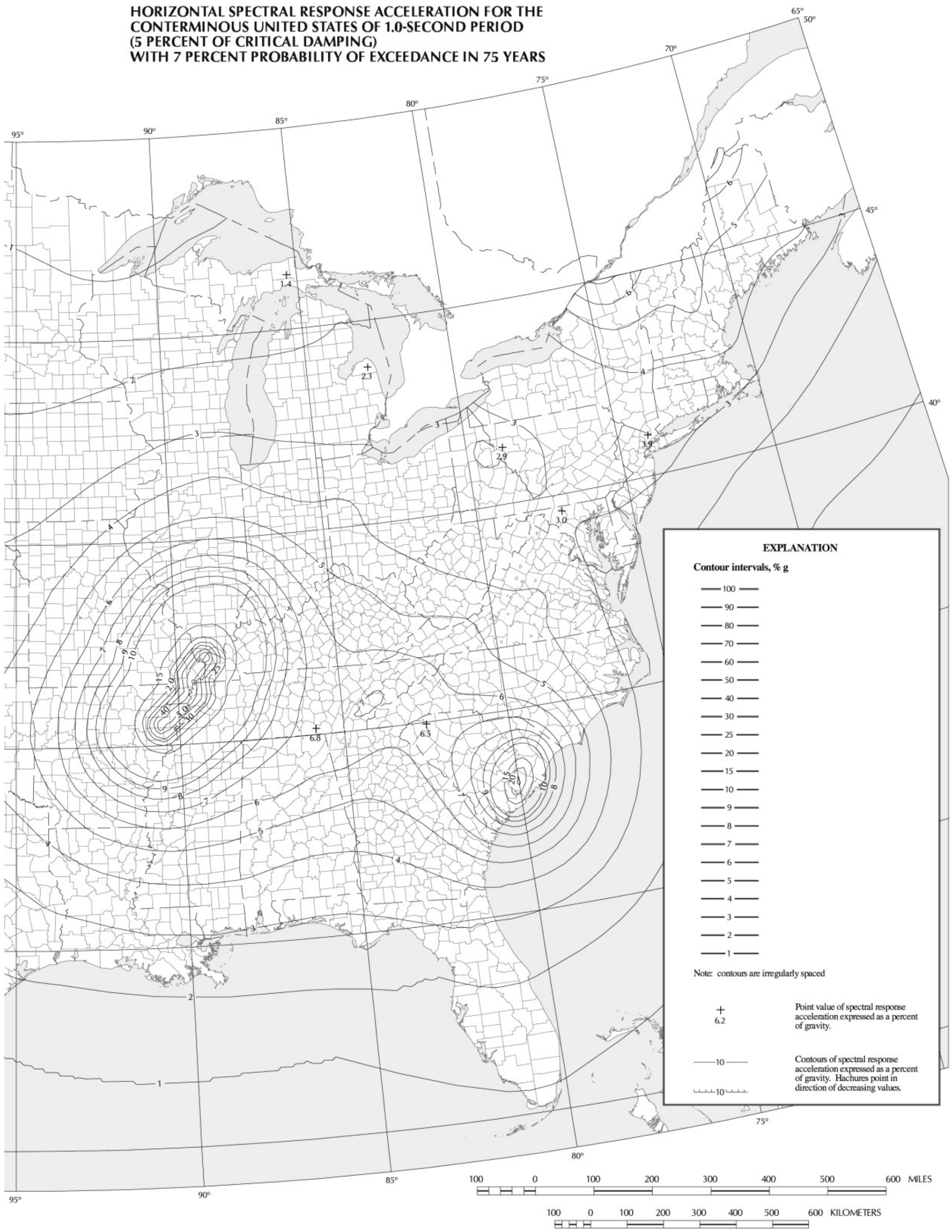


Figure 3.10.2.1-3 (continued)—Horizontal Response Spectral Acceleration Coefficient for the Conterminous United States at Period of 1.0 s (S_1) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period) and Five Percent Critical Damping

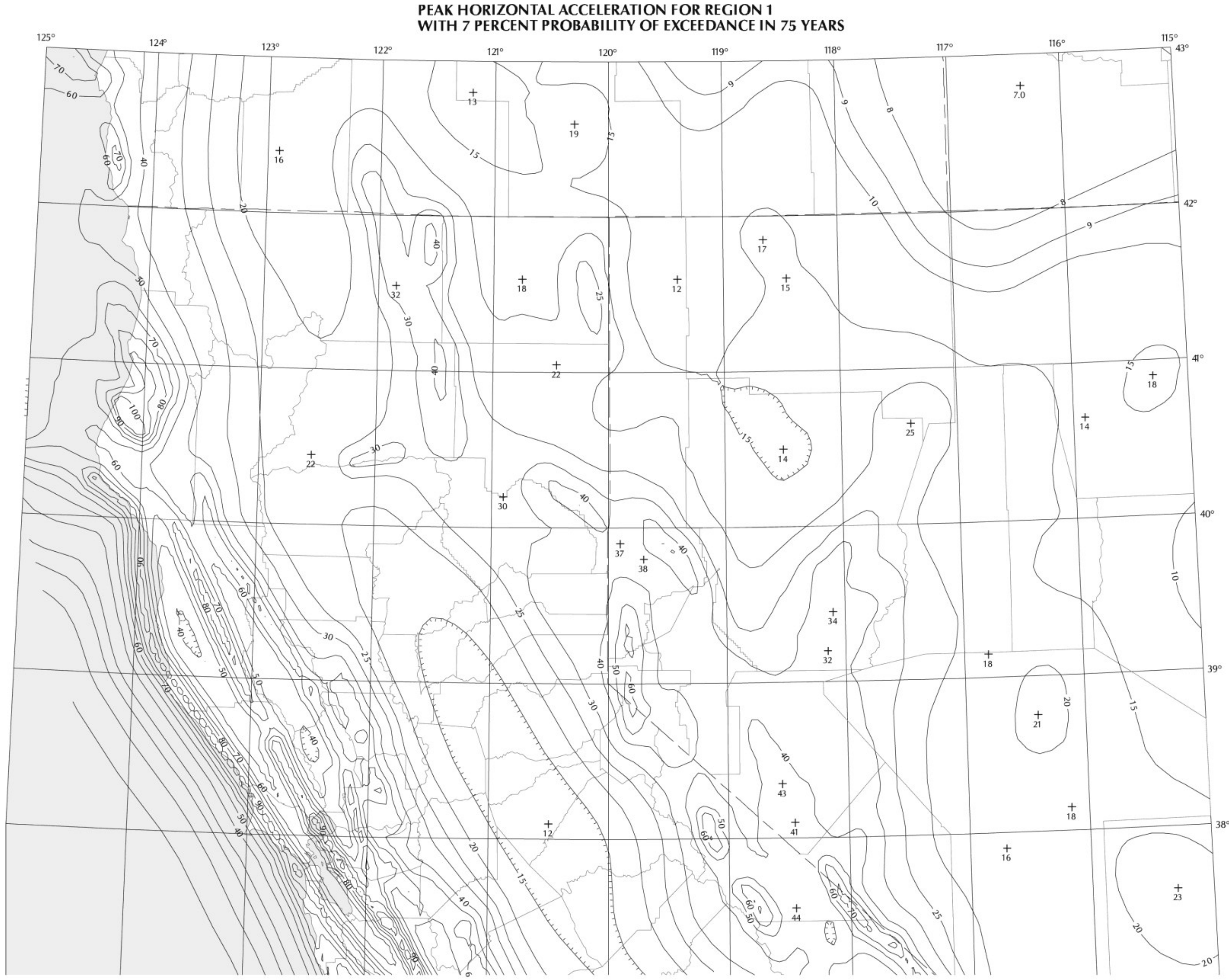
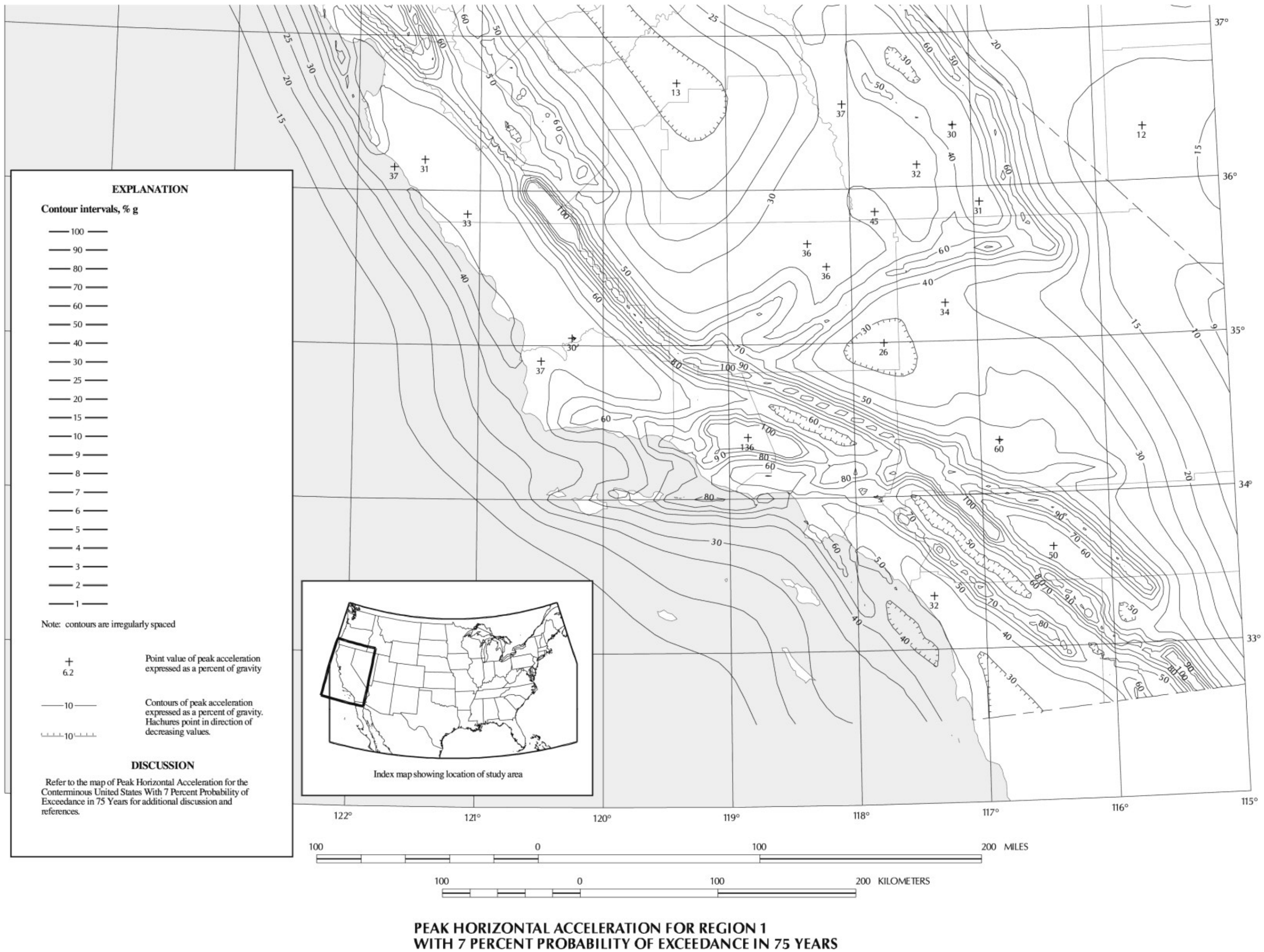


Figure 3.10.2.1-4—Horizontal Peak Ground Acceleration Coefficient for Region 1 (*PGA*) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period)

Figure 3.10.2.1-4 (continued)—Horizontal Peak Ground Acceleration Coefficient for Region 1 (*PGA*) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period)



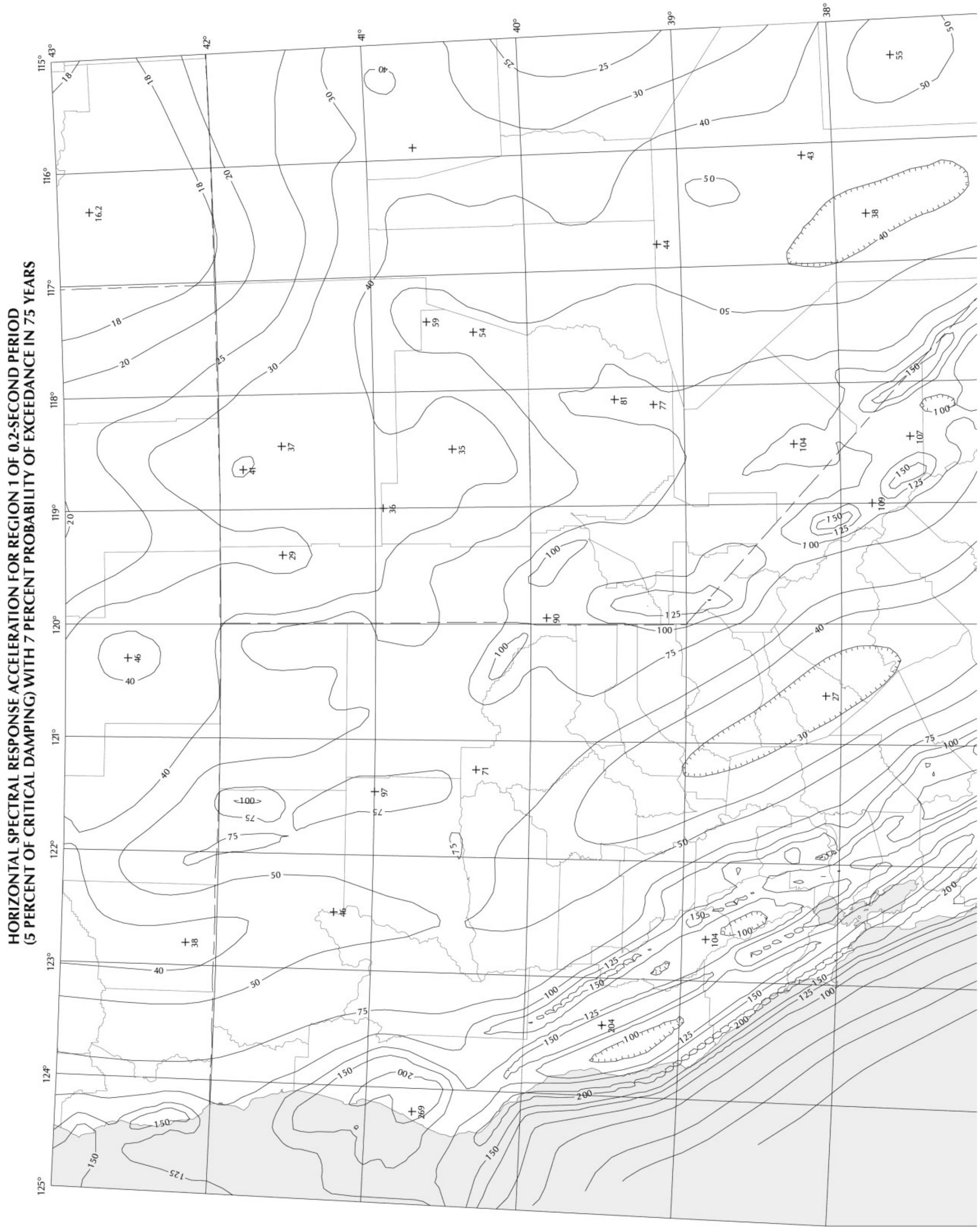
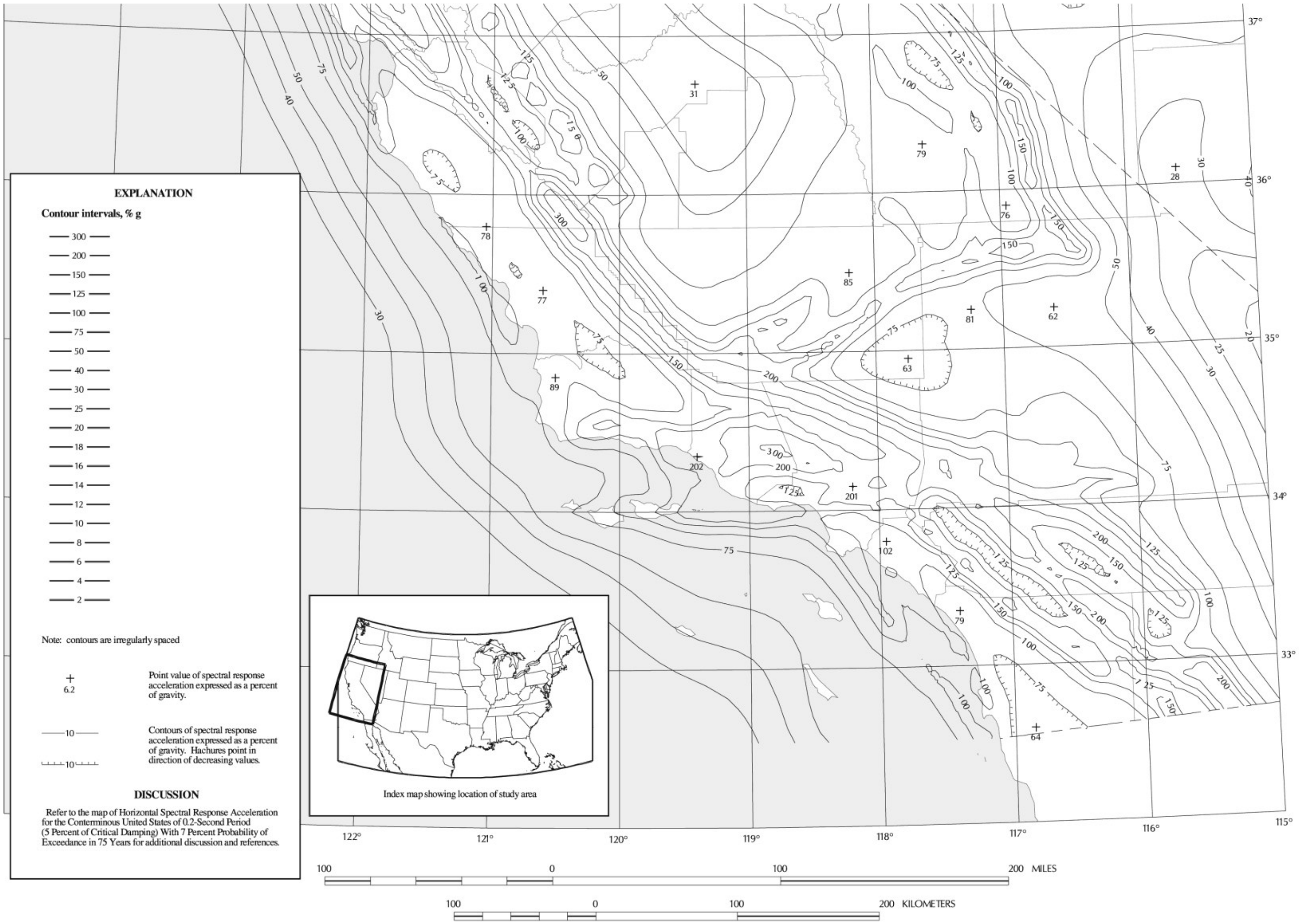


Figure 3.10.2.1-5—Horizontal Response Spectral Acceleration Coefficient for Region 1 at Period of 0.2 s (S_S) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period) and Five Percent Critical Damping



HORIZONTAL SPECTRAL RESPONSE ACCELERATION FOR REGION 1 OF 0.2-SECOND PERIOD
(5 PERCENT OF CRITICAL DAMPING) WITH 7 PERCENT PROBABILITY OF EXCEEDANCE IN 75 YEARS

Figure 3.10.2.1-5 (continued)—Horizontal Response Spectral Acceleration Coefficient for Region 1 at Period of 0.2 s (S_S) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period) and Five Percent Critical Damping

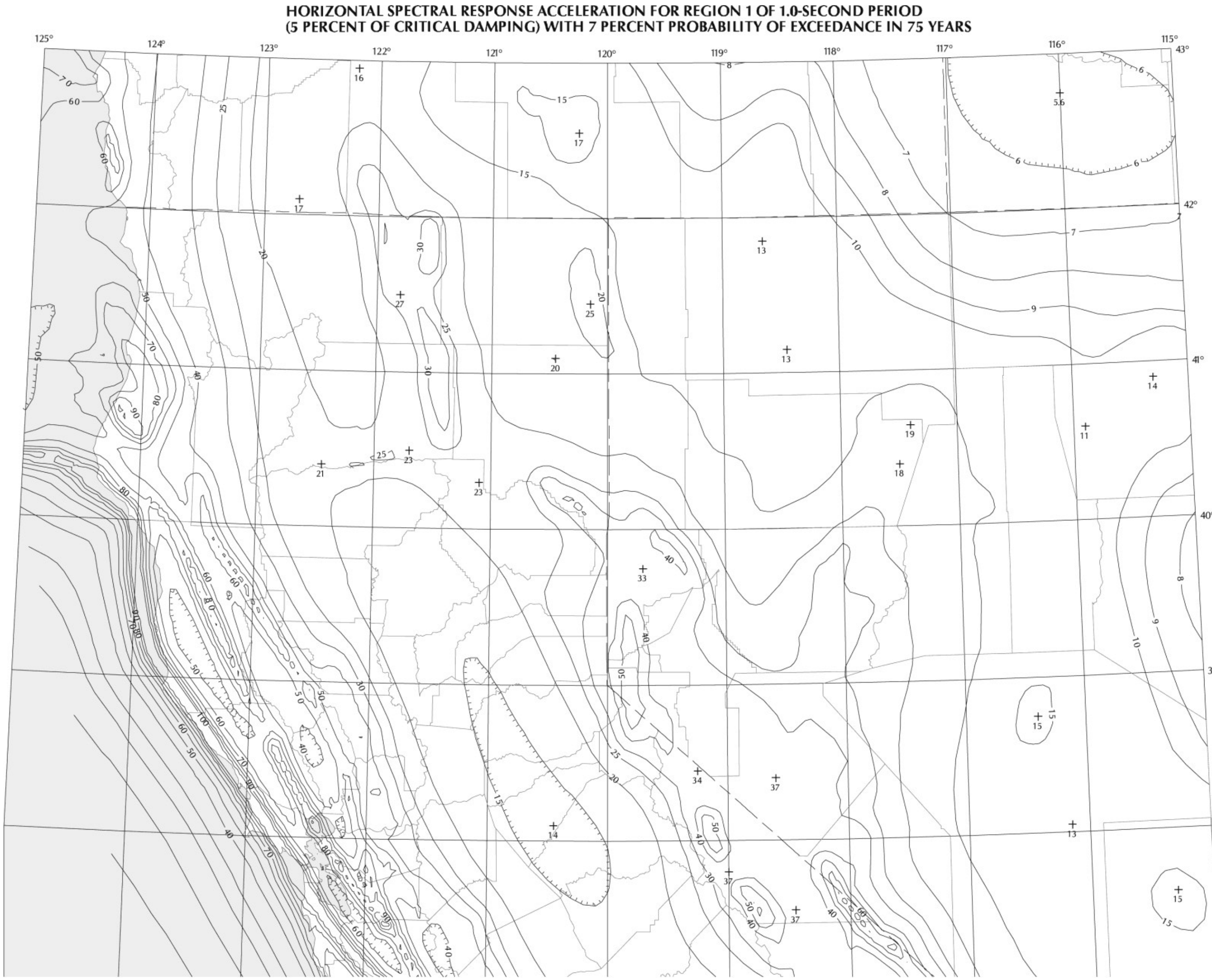
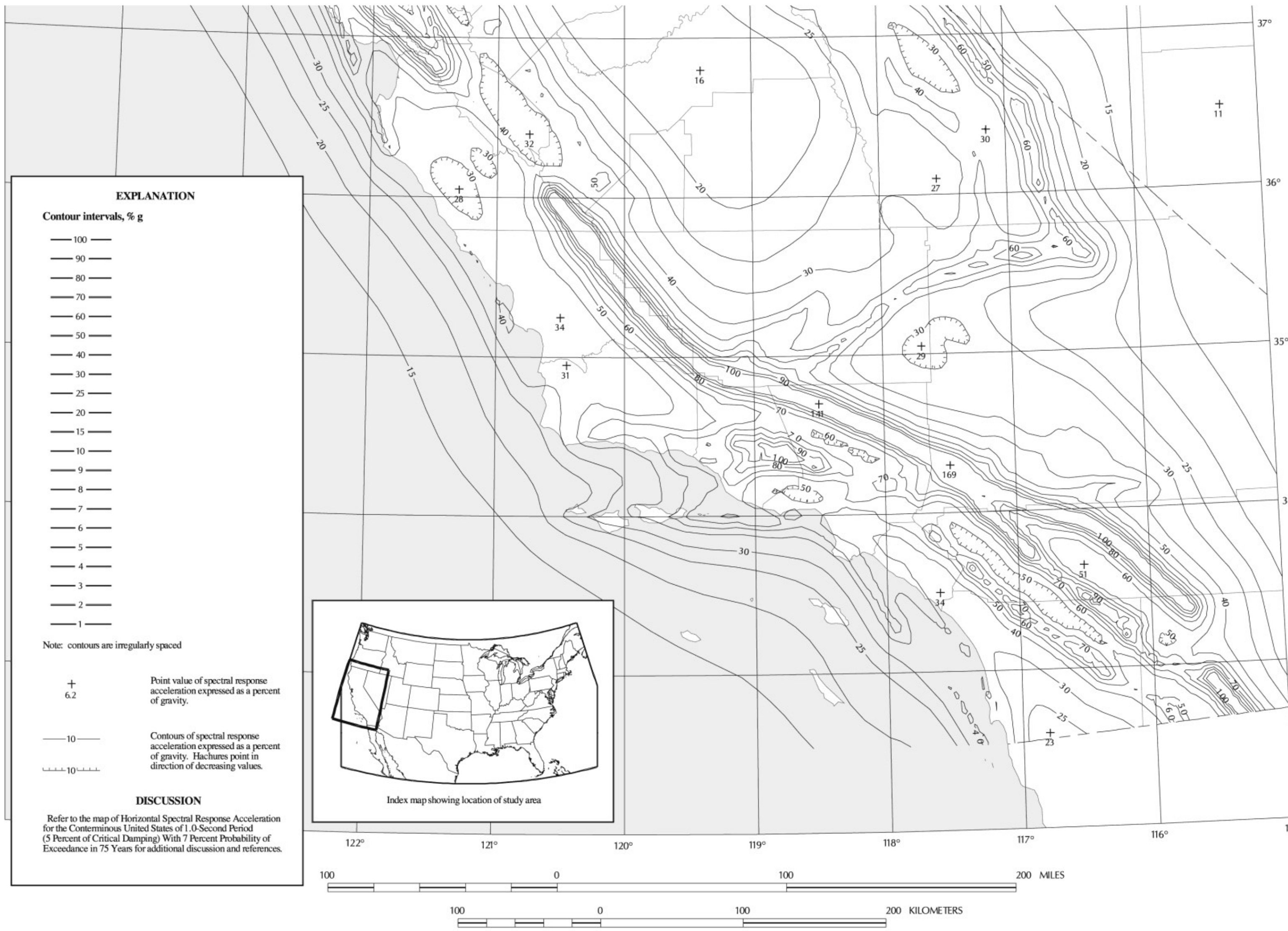


Figure 3.10.2.1-6—Horizontal Response Spectral Acceleration Coefficient for Region 1 at Period of 1.0 s (S_1) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period) and Five Percent Critical Damping



HORIZONTAL SPECTRAL RESPONSE ACCELERATION FOR REGION 1 OF 1.0-SECOND PERIOD
(5 PERCENT OF CRITICAL DAMPING) WITH 7 PERCENT PROBABILITY OF EXCEEDANCE IN 75 YEARS

Figure 3.10.2.1-6 (continued)—Horizontal Response Spectral Acceleration Coefficient for Region 1 at Period of 1.0 s (S_1) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period) and Five Percent Critical Damping

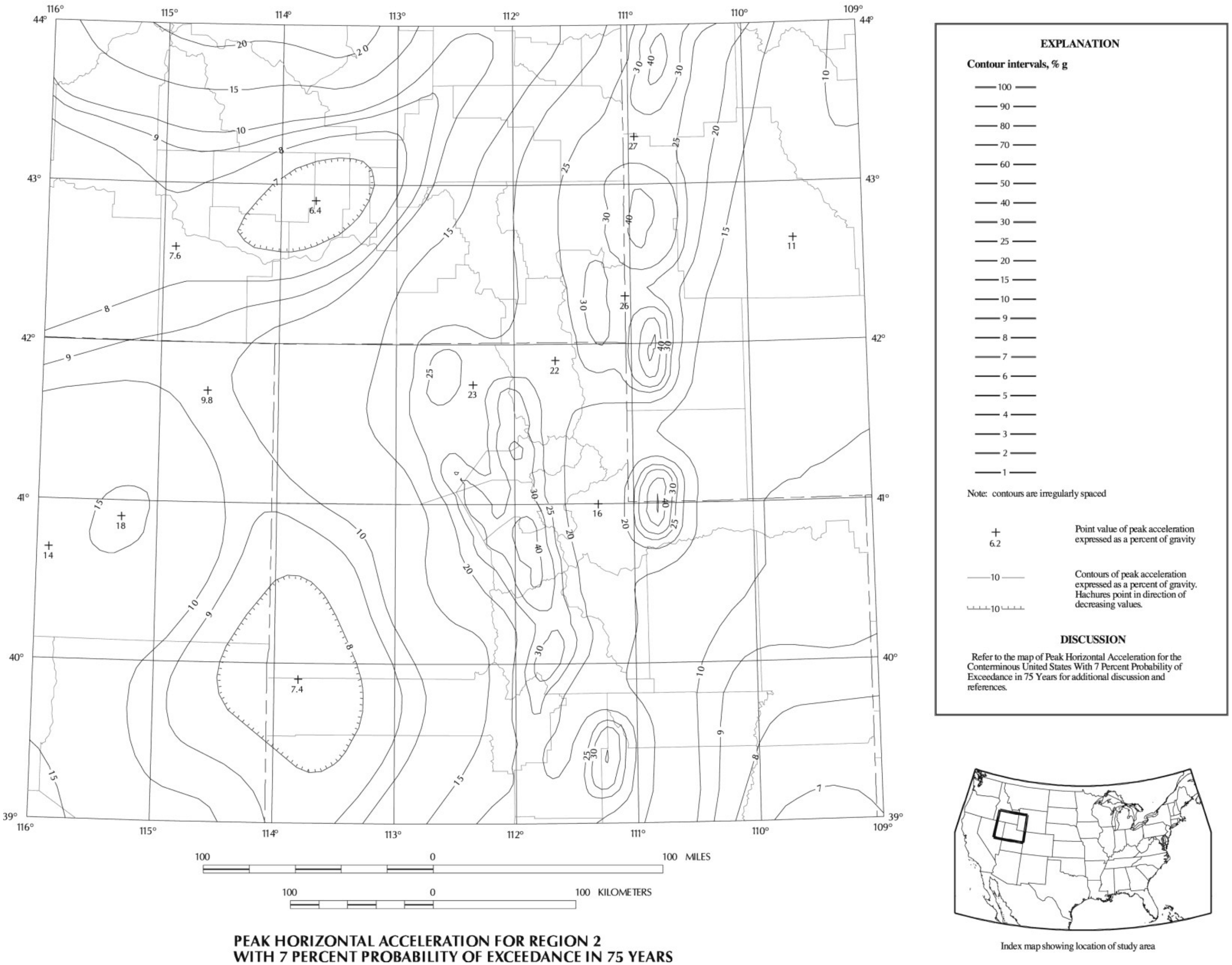
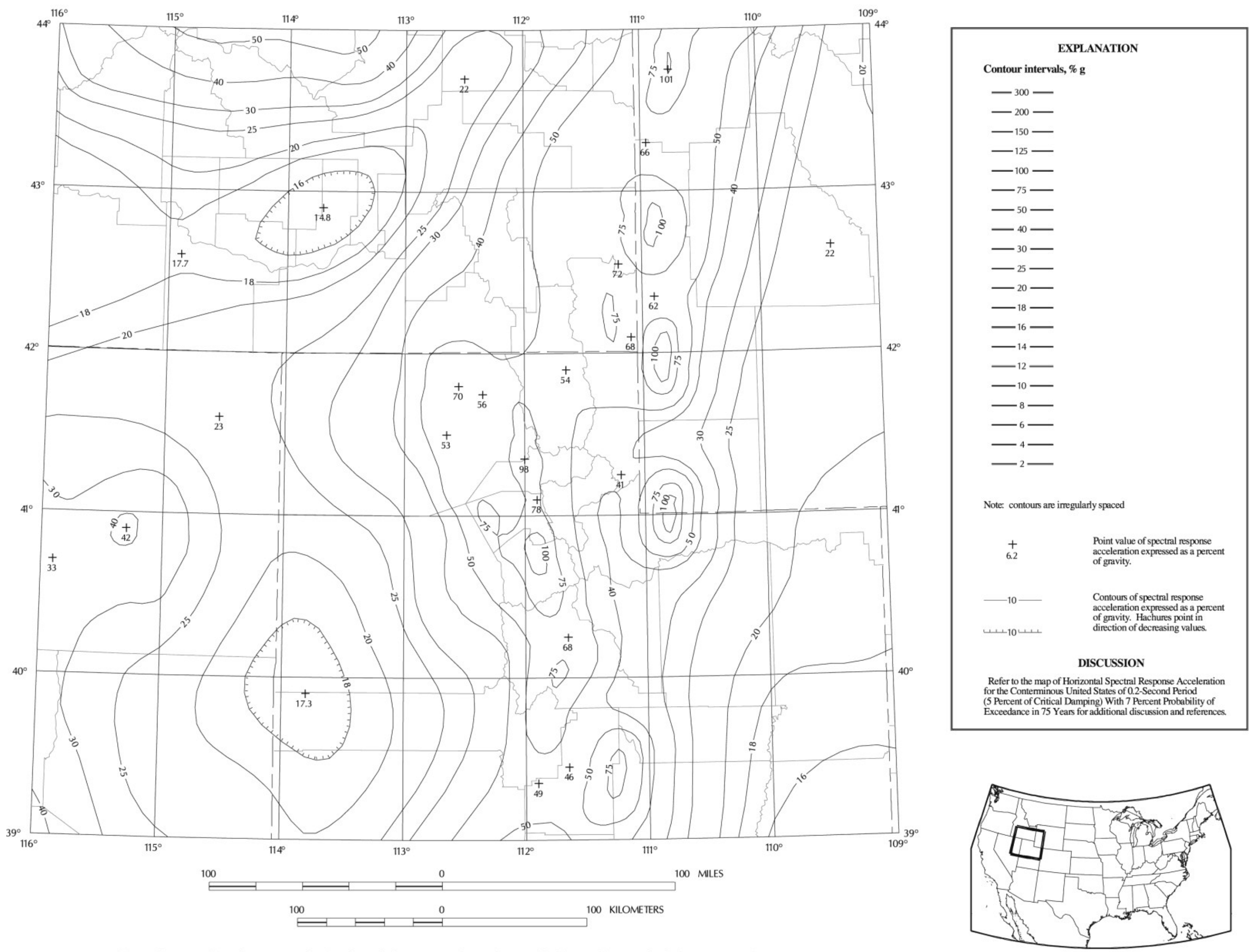


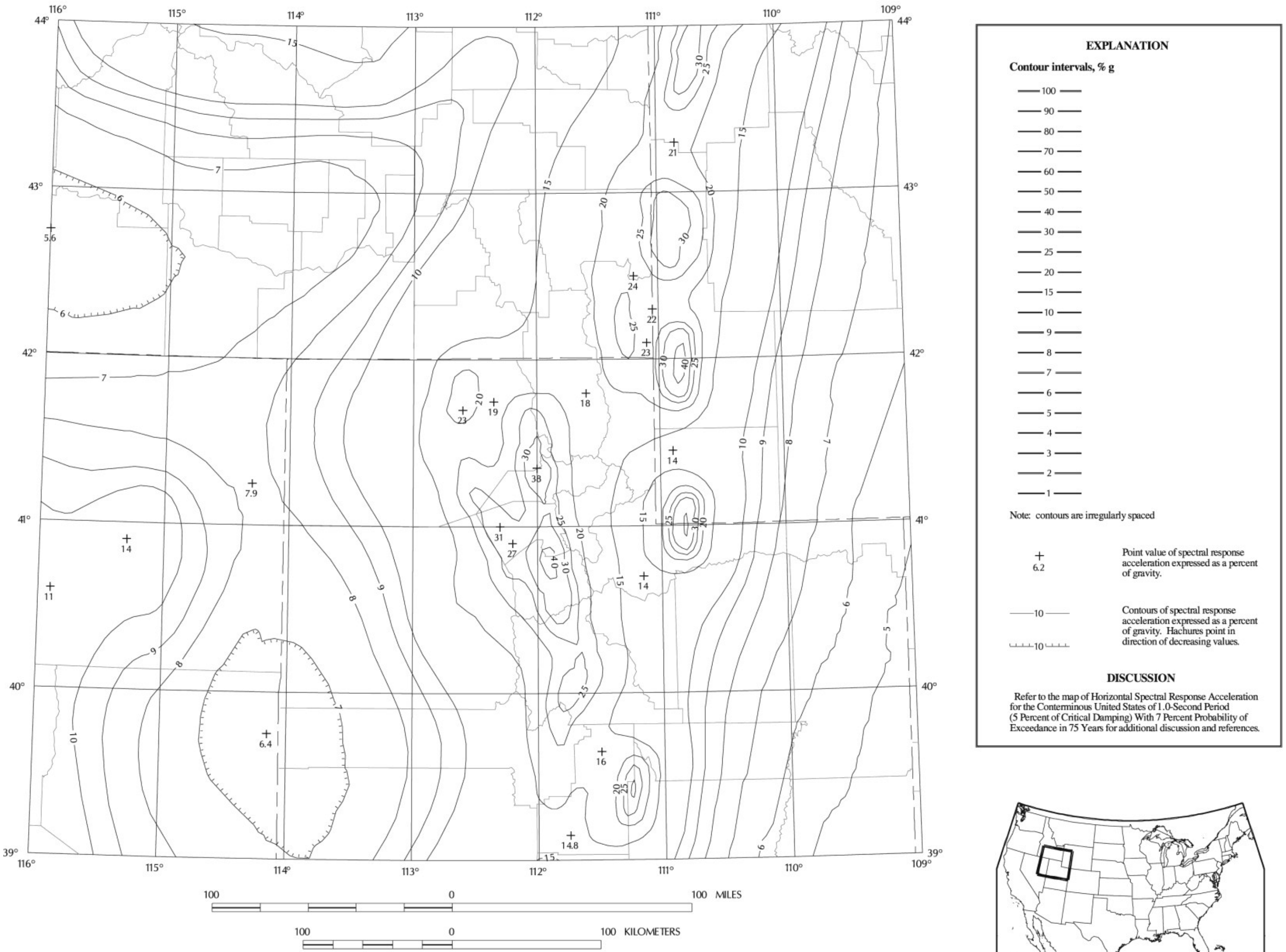
Figure 3.10.2.1-7—Horizontal Peak Ground Acceleration Coefficient for Region 2 (*PGA*) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period)



Index map showing location of study area

HORIZONTAL SPECTRAL RESPONSE ACCELERATION FOR REGION 2 OF 0.2-SECOND PERIOD (5 PERCENT OF CRITICAL DAMPING) WITH 7 PERCENT PROBABILITY OF EXCEEDANCE IN 75 YEARS

Figure 3.10.2.1-8—Horizontal Response Spectral Acceleration Coefficient for Region 2 at Period of 0.2 s (S_S) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period) and Five Percent Critical Damping



HORIZONTAL SPECTRAL RESPONSE ACCELERATION FOR REGION 2 OF 1.0-SECOND PERIOD (5 PERCENT OF CRITICAL DAMPING) WITH 7 PERCENT PROBABILITY OF EXCEEDANCE IN 75 YEARS

Figure 3.10.2.1-9—Horizontal Response Spectral Acceleration Coefficient for Region 2 at Period of 1.0 s (S_1) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period) and Five Percent Critical Damping

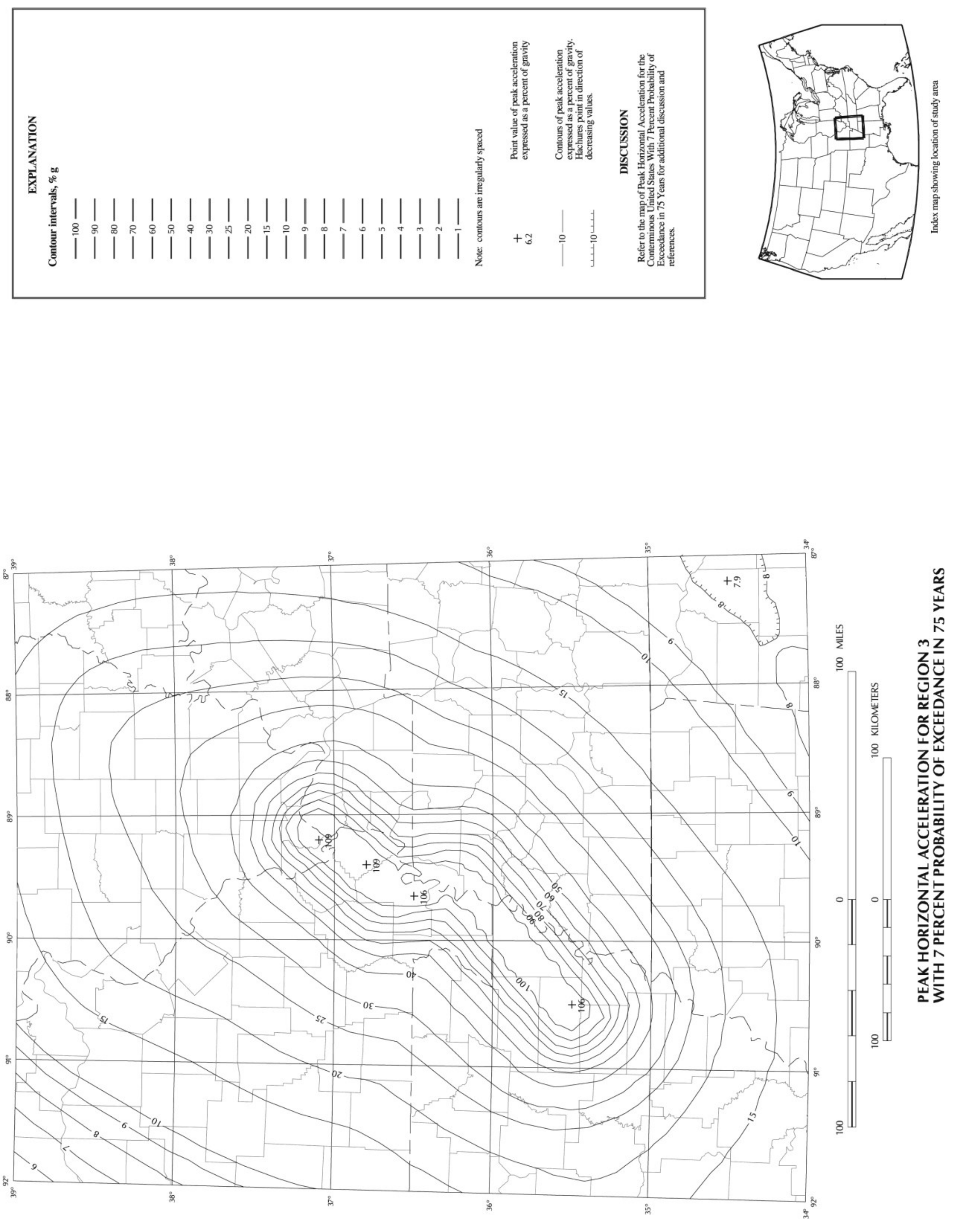
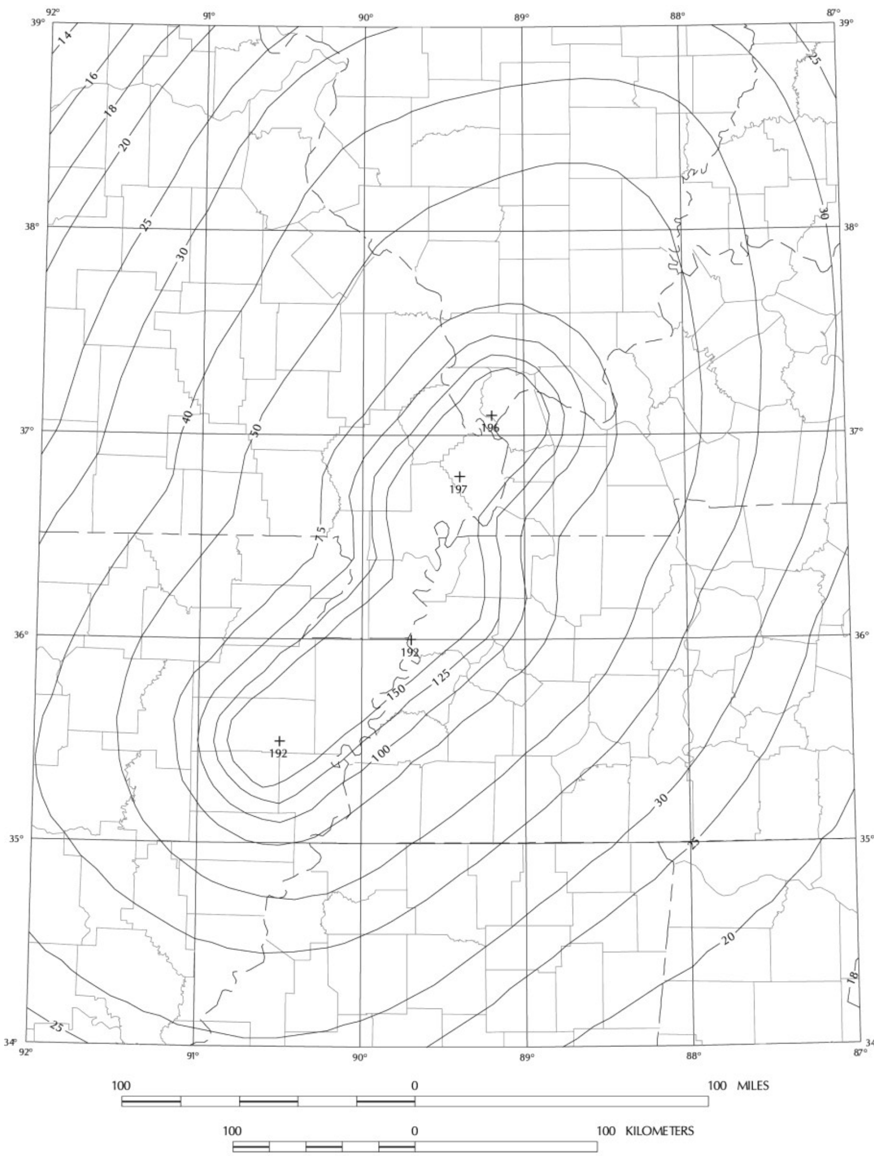


Figure 3.10.2.1-10—Horizontal Peak Ground Acceleration Coefficient for Region 3 (*PGA*) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period)

Figure 3.10.2.1-11—Horizontal Response Spectral Acceleration Coefficient for Region 3 at Period of 0.2 s (S_S) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period) and Five Percent Critical Damping



HORIZONTAL SPECTRAL RESPONSE ACCELERATION FOR REGION 3 OF 0.2-SECOND PERIOD
(5 PERCENT OF CRITICAL DAMPING) WITH 7 PERCENT PROBABILITY OF EXCEEDANCE IN 75 YEARS

EXPLANATION

Contour intervals, % g

300
200
150
125
100
75
50
40
30
25
20
18
16
14
12
10
8
6
4
2

Note: contours are irregularly spaced

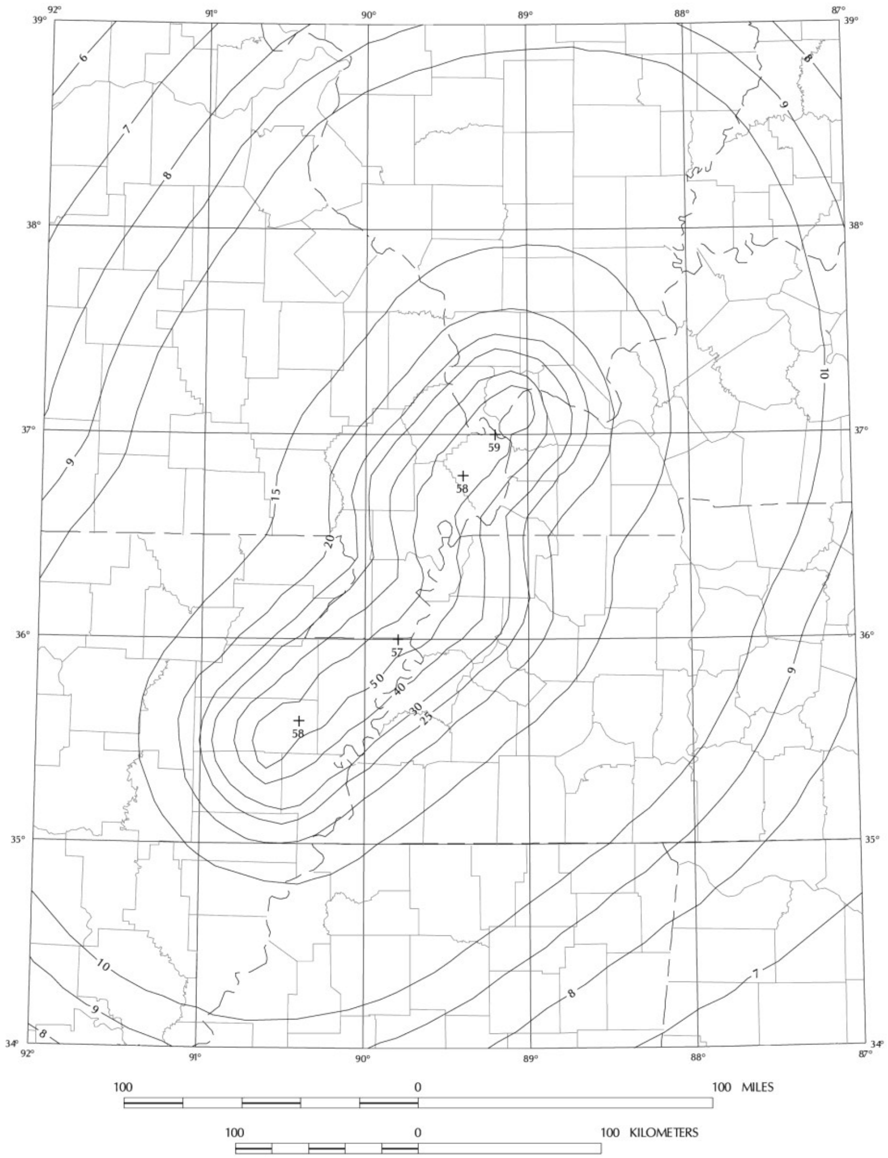
+ 6.2 — Point value of spectral response acceleration expressed as a percent of gravity.

10 — Contours of spectral response acceleration expressed as a percent of gravity. Hachures point in direction of decreasing values.

DISCUSSION

Refer to the map of Horizontal Spectral Response Acceleration for the Conterminous United States of 0.2-Second Period (5 Percent of Critical Damping) With 7 Percent Probability of Exceedance in 75 Years for additional discussion and references.

Index map showing location of study area



HORIZONTAL SPECTRAL RESPONSE ACCELERATION FOR REGION 3 OF 1.0-SECOND PERIOD (5 PERCENT OF CRITICAL DAMPING) WITH 7 PERCENT PROBABILITY OF EXCEEDANCE IN 75 YEARS

EXPLANATION

Contour intervals, % g

100
90
80
70
60
50
40
30
25
20
15
10
9
8
7
6
5
4
3
2
1

Note: contours are irregularly spaced

+ 6.2 — Point value of spectral response acceleration expressed as a percent of gravity.

—10— / ⊥⊥⊥10⊥⊥⊥ — Contours of spectral response acceleration expressed as a percent of gravity. Hachures point in direction of decreasing values.

DISCUSSION

Refer to the map of Horizontal Spectral Response Acceleration for the Conterminous United States of 1.0-Second Period (5 Percent of Critical Damping) With 7 Percent Probability of Exceedance in 75 Years for additional discussion and references.

Index map showing location of study area

Figure 3.10.2.1-12—Horizontal Response Spectral Acceleration Coefficient for Region 3 at Period of 1.0 s (S_1) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period) and Five Percent Critical Damping

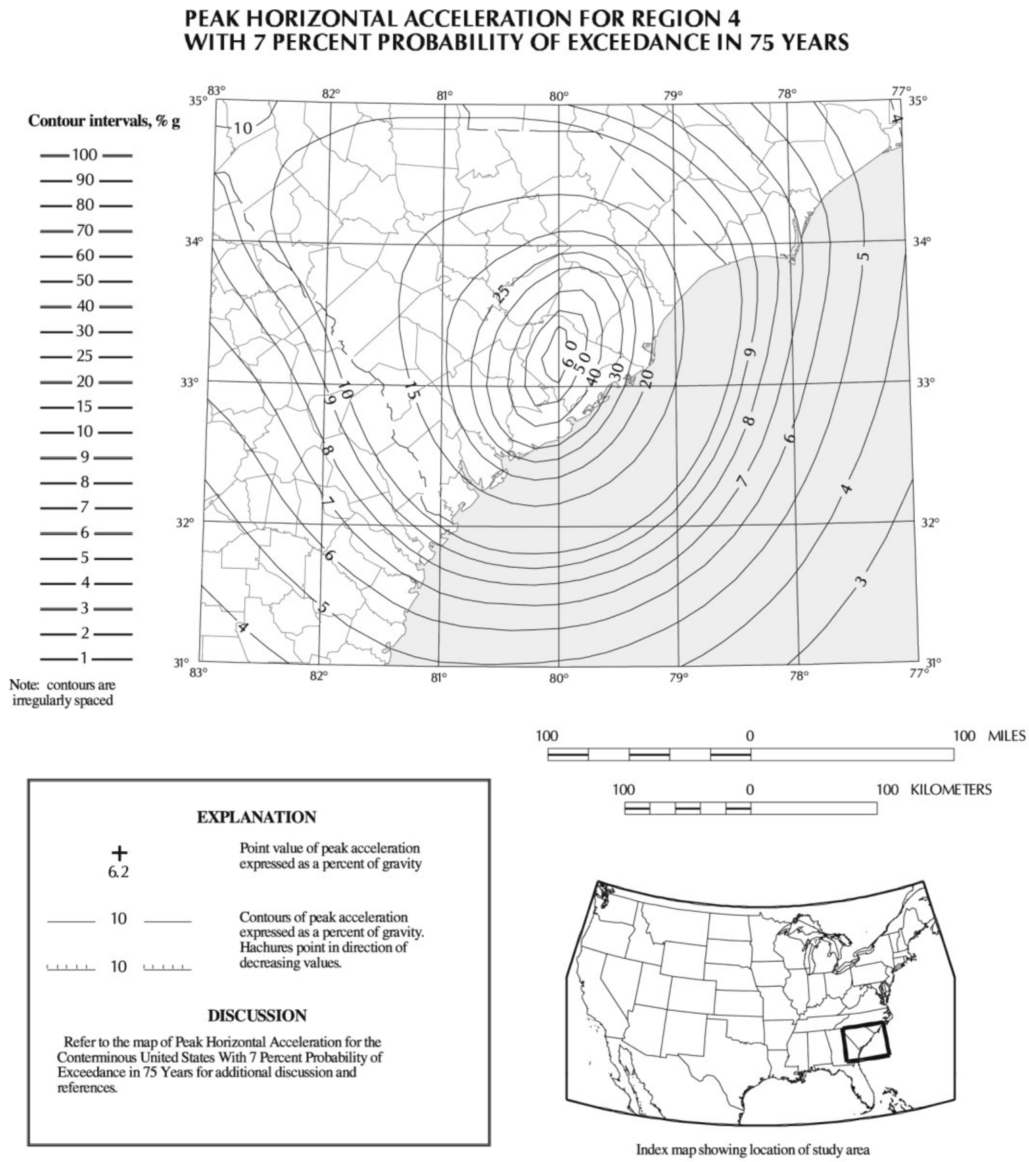


Figure 3.10.2.1-13—Horizontal Peak Ground Acceleration Coefficient for Region 4 (*PGA*) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period)

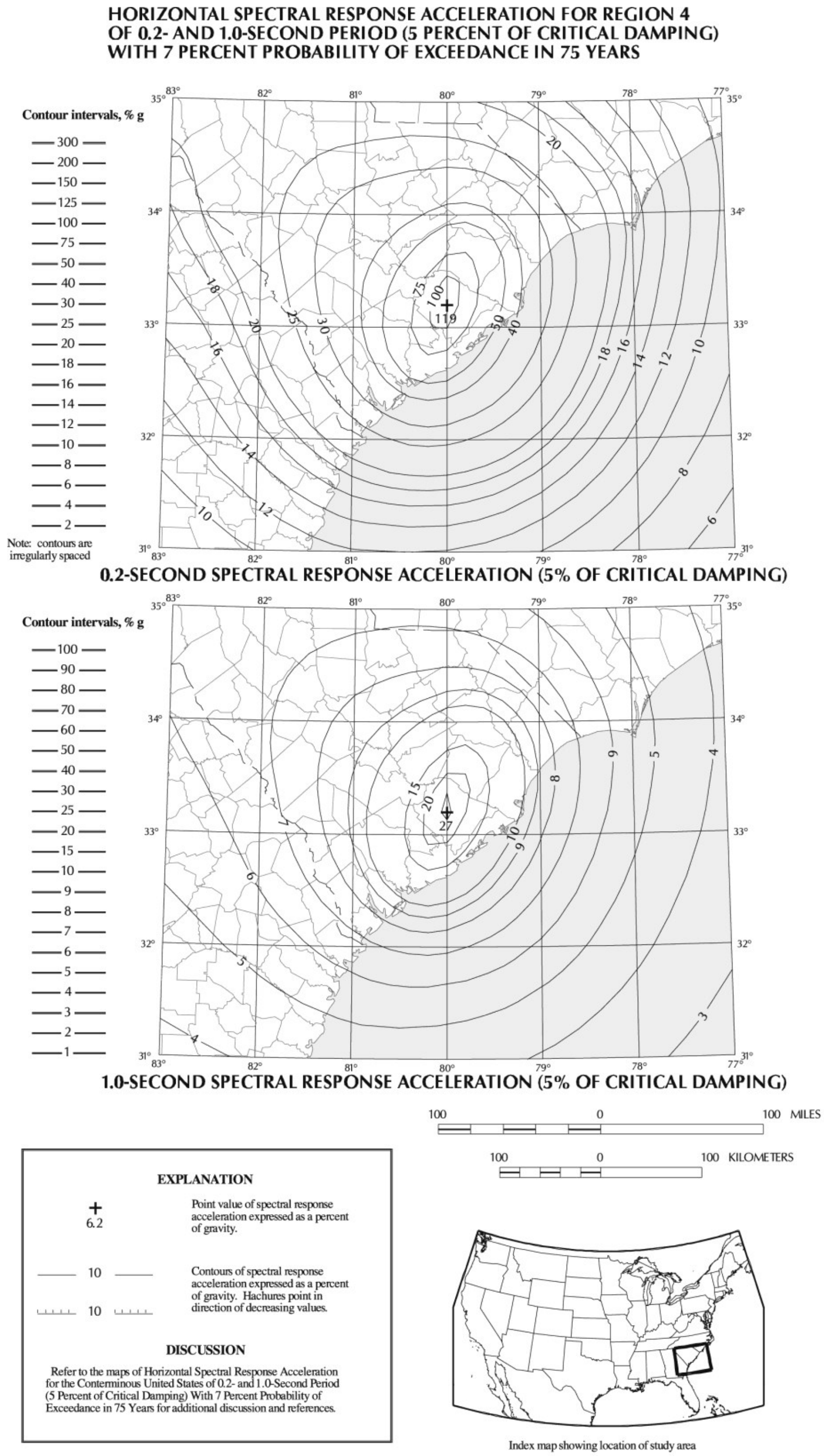
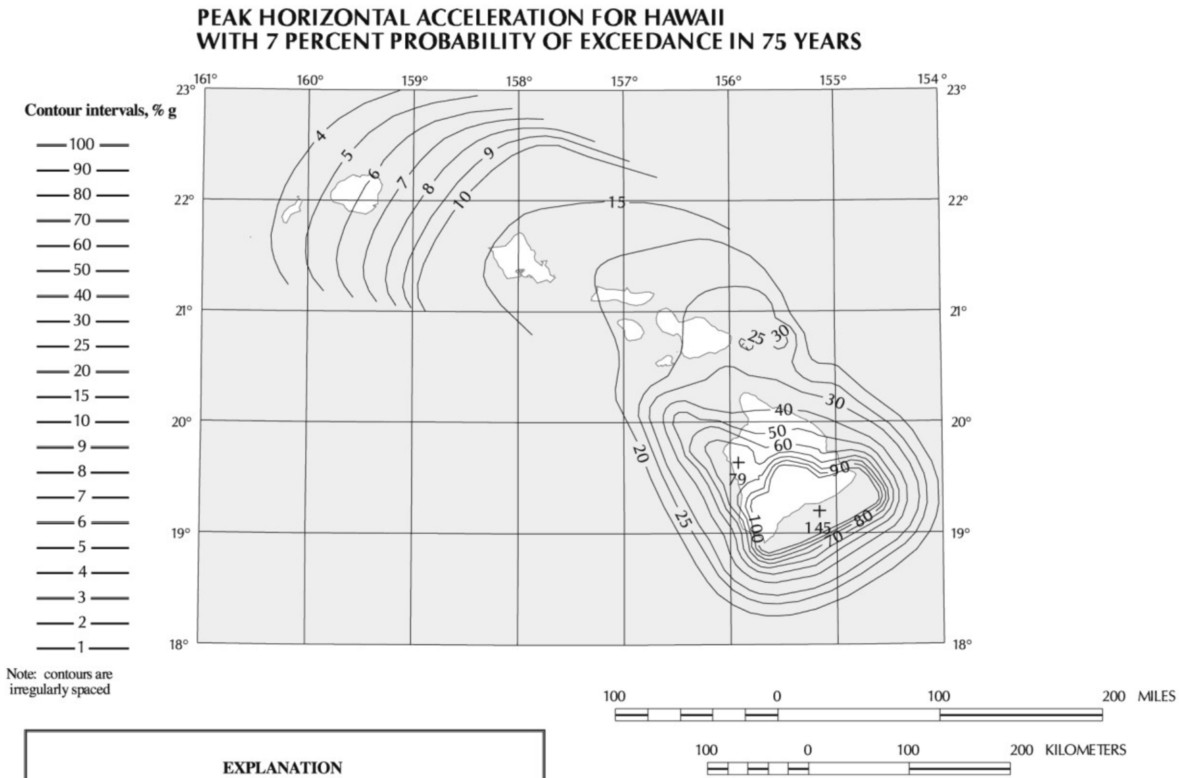


Figure 3.10.2.1-14—Horizontal Response Spectral Acceleration Coefficients for Region 4 at Periods of 0.2 s (S_S) and 1.0 s (S_1) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period) and Five Percent Critical Damping



EXPLANATION

+ 6.2 — Point value of peak acceleration expressed as a percent of gravity

10 — Contours of peak acceleration expressed as a percent of gravity. Hachures point in direction of decreasing values.

DISCUSSION

The acceleration values contoured on this map are for the random horizontal component of acceleration. For design purposes, the reference site condition for the map is to be taken as Site Class B.

Leyendecker, Frankel, and Rukstales (2007) have prepared a CD-ROM that contains software to allow determination of Site Class B map values by latitude-longitude. The software on the CD contains site coefficients that allow the user to adjust map values for different Site Classes. Additional maps at different scales are also included on the CD. The CD was prepared using the same data as that used to prepare the USGS Probabilistic Earthquake Ground Motion maps.

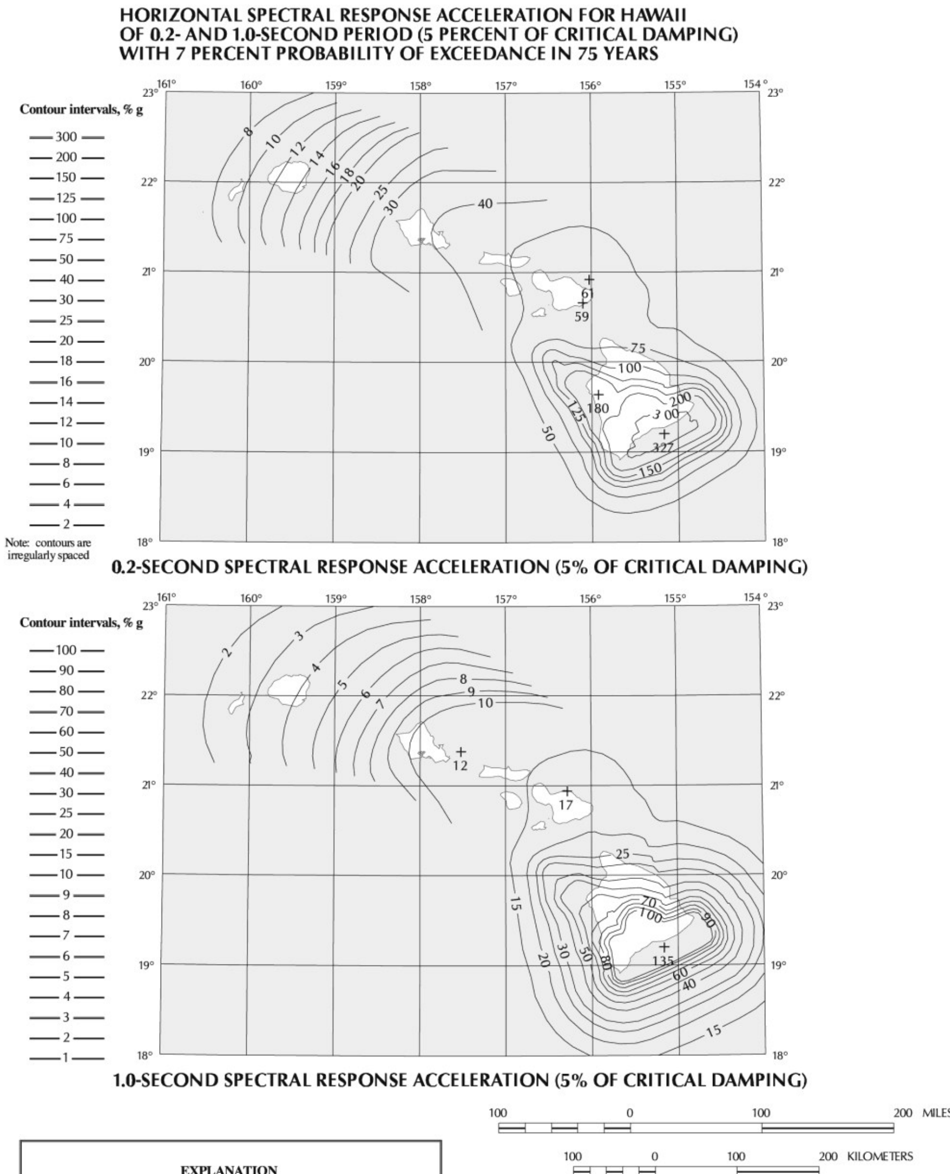
The National Seismic Hazard Mapping Web Site, http://earthquake.usgs.gov/research/hazmaps contains information about maps, data, and documentation for seismic hazard maps.

Map prepared by U.S. Geological Survey.

REFERENCES

Imbsen, R., 2007, Proposed AASHTO Guide Specifications for LRFD Seismic Bridge Design, Imbsen Consulting.
Klein, F., Frankel, A., Mueller, C., Wesson, R. and Okubo, P., 2001, Seismic hazard in Hawaii: high rate of large earthquakes and probabilistic ground-motion maps, Bull. Seism. Soc. Am., v. 91, pp. 479-498.
Leyendecker, E., Frankel, A., and Rukstales, K., 2007, Seismic Design Parameters for AASHTO Guide Specifications for LRFD Seismic Bridge Design.
U. S. Geological Survey, National Seismic Hazard Mapping Project Web Site, http://earthquake.usgs.gov/research/hazmaps.

Figure 3.10.2.1-15—Horizontal Peak Ground Acceleration Coefficient for Hawaii (*PGA*) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period)



EXPLANATION

+ 6.2 — Point value of spectral response acceleration expressed as a percent of gravity.

—— 10 —— Contours of spectral response acceleration expressed as a percent of gravity. Hachures point in direction of decreasing values.

DISCUSSION

The acceleration values contoured on this map are for the random horizontal component of acceleration. For design purposes, the reference site condition for the map is to be taken as Site Class B.

Leyendecker, Frankel, and Rukstales (2007) have prepared a CD-ROM that contains software to allow determination of Site Class B map values by latitude-longitude. The software on the CD contains site coefficients that allow the user to adjust map values for different Site Classes. Additional maps at different scales are also included on the CD. The CD was prepared using the same data as that used to prepare the USGS Probabilistic Earthquake Ground Motion maps.

The National Seismic Hazard Mapping Web Site, http://earthquake.usgs.gov/research/hazmaps contains information about maps, data, and documentation for seismic hazard maps.

Map prepared by U.S. Geological Survey.

REFERENCES

Imbsen, R., 2007, Proposed AASHTO Guide Specifications for LRFD Seismic Bridge Design, Imbsen Consulting.

Klein, F., Frankel, A., Mueller, C., Wesson, R. and Okubo, P., 2001, Seismic hazard in Hawaii: high rate of large earthquakes and probabilistic ground-motion maps, Bull. Seism. Soc. Am., v. 91, pp. 479-498.

Leyendecker, E., Frankel, A., and Rukstales, K., 2007, Seismic Design Parameters for AASHTO Guide Specifications for LRFD Seismic Bridge Design.

U. S. Geological Survey, National Seismic Hazard Mapping Project Web Site, http://earthquake.usgs.gov/research/hazmaps.

Figure 3.10.2.1-16—Horizontal Response Spectral Acceleration Coefficients for Hawaii at Periods of 0.2 s (S_S) and 1.0 s (S_1) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period) and Five Percent Critical Damping

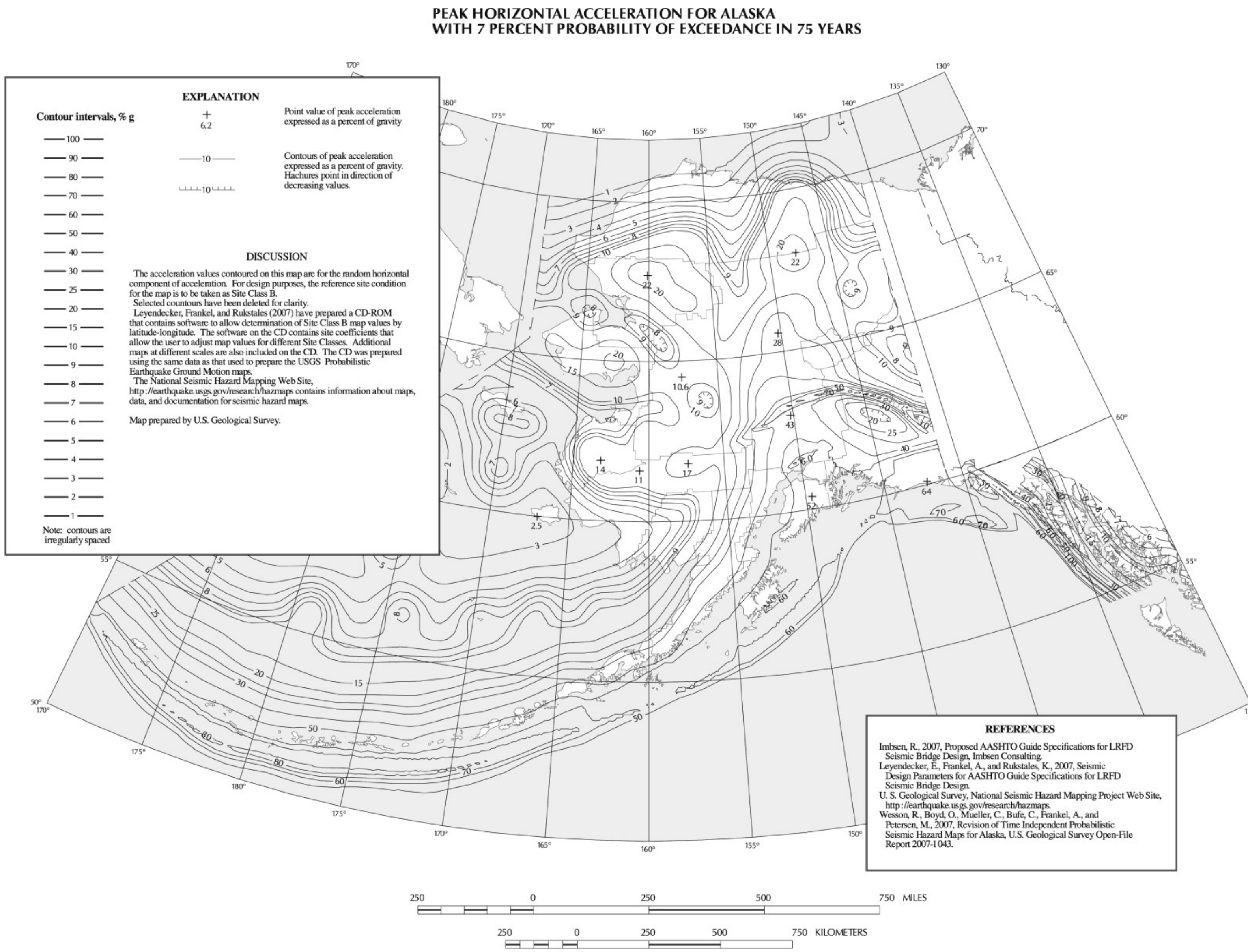


Figure 3.10.2.1-17—Horizontal Peak Ground Acceleration Coefficient for Alaska (*PGA*) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period)

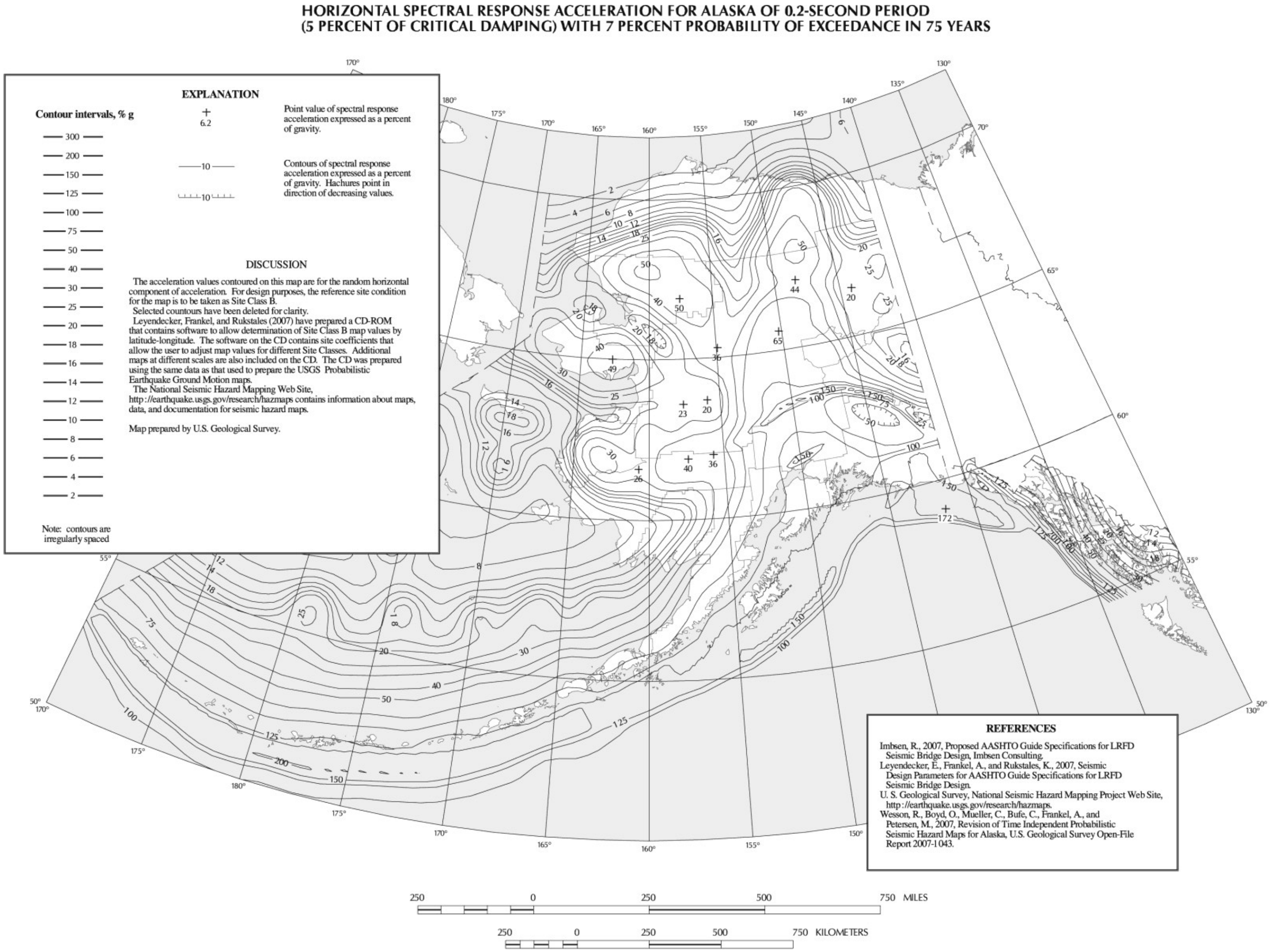


Figure 3.10.2.1-18—Horizontal Response Spectral Acceleration Coefficient for Alaska at Period of 0.2 s (S_S) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period) and Five Percent Critical Damping

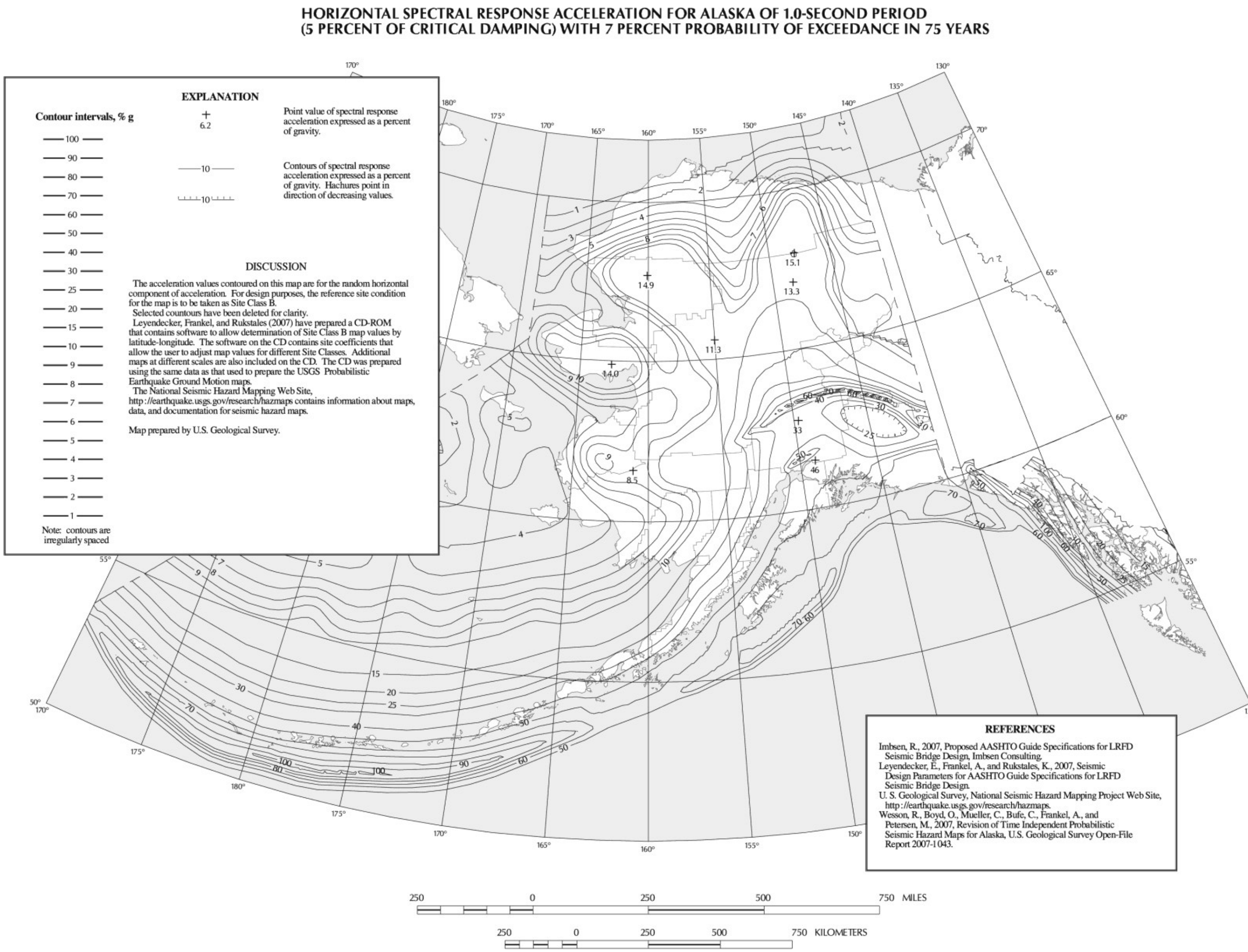
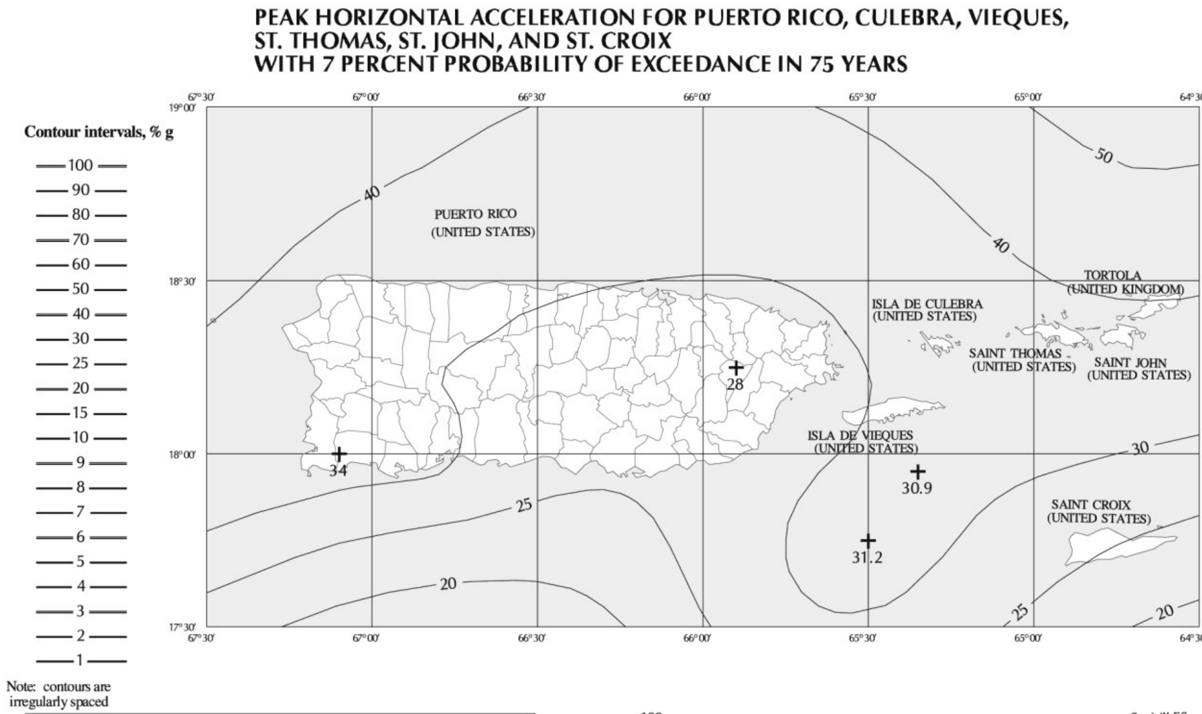


Figure 3.10.2.1-19—Horizontal Response Spectral Acceleration Coefficient for Alaska at Period of 1.0 s (S_1) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period) and Five Percent Critical Damping



100 0 MILES

100 0 KILOMETERS

EXPLANATION

+ 6.2 — Point value of peak acceleration expressed as a percent of gravity

—— 10 —— / ⊥⊥⊥ 10 ⊥⊥⊥ — Contours of peak acceleration expressed as a percent of gravity. Hachures point in direction of decreasing values.

DISCUSSION

The acceleration values contoured on this map are for the random horizontal component of acceleration. For design purposes, the reference site condition for the map is to be taken as Site Class B.

Leyendecker, Frankel, and Rukstales (2007) have prepared a CD-ROM that contains software to allow determination of Site Class B map values by latitude-longitude. The software on the CD contains site coefficients that allow the user to adjust map values for different Site Classes. Additional maps at different scales are also included on the CD. The CD was prepared using the same data as that used to prepare the USGS Probabilistic Earthquake Ground Motion maps.

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Map prepared by U.S. Geological Survey.

REFERENCES

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Leyendecker, E., Frankel, A., and Rukstales, K., 2007, Seismic Design Parameters for AASHTO Guide Specifications for LRFD Seismic Bridge Design.
Mueller, C., Frankel, A., Petersen, M., and Leyendecker, E., 2003, Documentation for 2003 USGS Seismic Hazard Maps for Puerto Rico and the U.S. Virgin Islands, U.S. Geological Survey Open-File Report 03-379.
U. S. Geological Survey, National Seismic Hazard Mapping Project Web Site, http://earthquake.usgs.gov/research/hazmaps.

Figure 3.10.2.1-20—Horizontal Peak Ground Acceleration Coefficient for Puerto Rico, Culebra, Vieques, St. Thomas, St. John, and St. Croix (*PGA*) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period)

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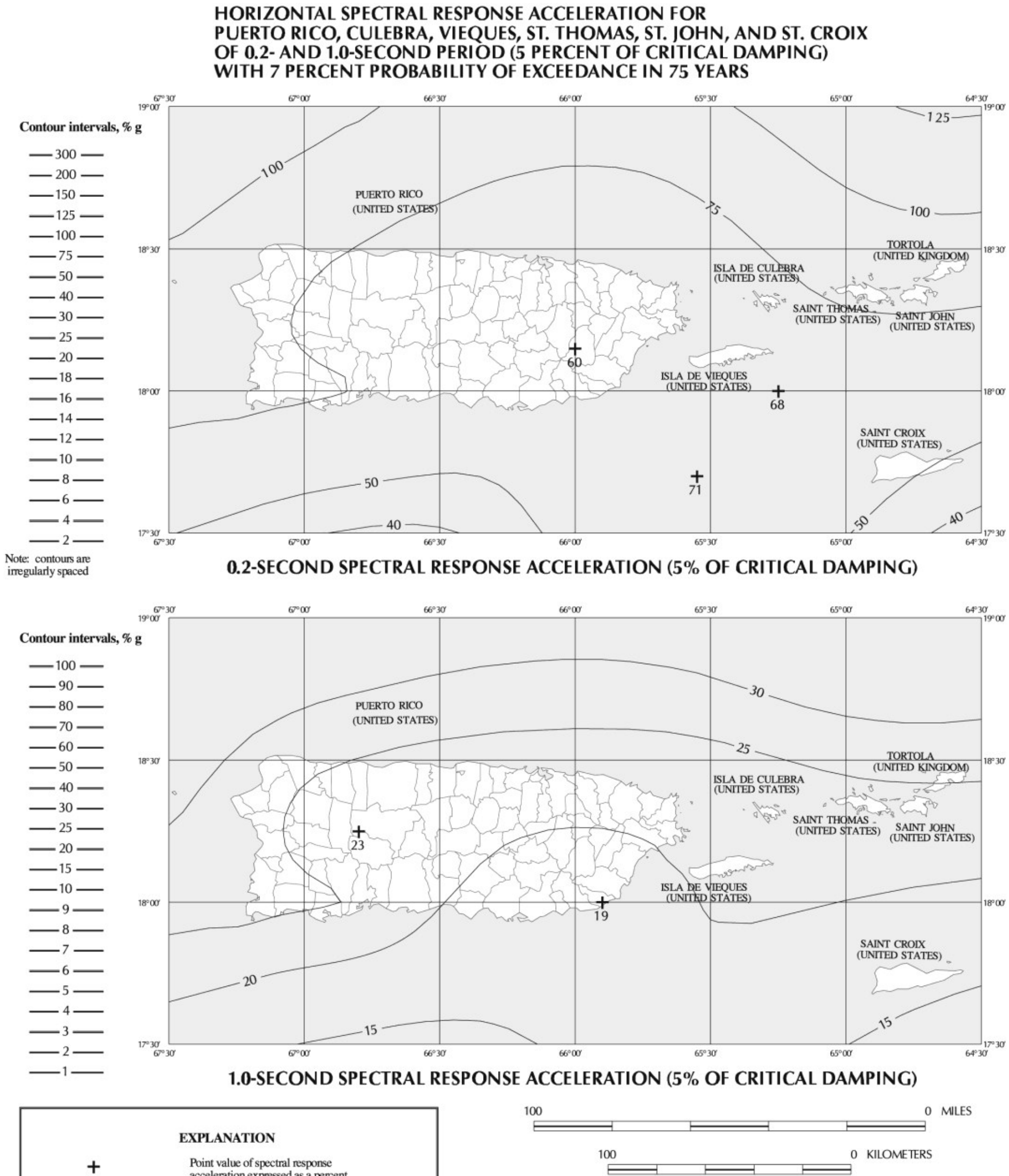

EXPLANATION

+ 6.2 — Point value of spectral response acceleration expressed as a percent of gravity.

—— 10 —— / ⊥⊥⊥ 10 ⊥⊥⊥ — Contours of spectral response acceleration expressed as a percent of gravity. Hachures point in direction of decreasing values.

DISCUSSION

The acceleration values contoured on this map are for the random horizontal component of acceleration. For design purposes, the reference site condition for the map is to be taken as Site Class B.

Leyendecker, Frankel, and Rukstales (2007) have prepared a CD-ROM that contains software to allow determination of Site Class B map values by latitude-longitude. The software on the CD contains site coefficients that allow the user to adjust map values for different Site Classes. Additional maps at different scales are also included on the CD. The CD was prepared using the same data as that used to prepare the USGS Probabilistic Earthquake Ground Motion maps.

The National Seismic Hazard Mapping Web Site, http://earthquake.usgs.gov/research/hazmaps contains information about maps, data, and documentation for seismic hazard maps.

Map prepared by U.S. Geological Survey.

REFERENCES

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Leyendecker, E., Frankel, A., and Rukstales, K., 2007, Seismic Design Parameters for AASHTO Guide Specifications for LRFD Seismic Bridge Design.

Mueller, C., Frankel, A., Petersen, M., and Leyendecker, E., 2003, Documentation for 2003 USGS Seismic Hazard Maps for Puerto Rico and the U.S. Virgin Islands, U.S. Geological Survey Open-File Report 03-379.

U. S. Geological Survey, National Seismic Hazard Mapping Project Web Site, http://earthquake.usgs.gov/research/hazmaps.

Figure 3.10.2.1-21—Horizontal Response Spectral Acceleration Coefficients for Puerto Rico, Culebra, Vieques, St. Thomas, St. John, and St. Croix at Periods of 0.2 s (S_S) and 1.0 s (S_1) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period) and Five Percent Critical Damping

3.10.2.2—Site-Specific Procedure

A site-specific procedure to develop design response spectra of earthquake ground motions shall be performed when required by Article 3.10.2 and may be performed for any site. The objective of the site-specific probabilistic ground-motion analysis should be to generate a uniform-hazard acceleration response spectrum considering a seven percent probability of exceedance in 75 years for spectral values over the entire period range of interest. This analysis should involve establishing:

- the contributing seismic sources;
- an upper-bound earthquake magnitude for each source zone;
- median attenuation relations for acceleration response spectral values and their associated standard deviations;
- a magnitude-recurrence relation for each source zone; and
- a fault-rupture-length relation for each contributing fault.

Uncertainties in source modeling and parameter values shall be taken into consideration. Detailed documentation of ground-motion analysis is required and shall be peer reviewed.

Where analyses to determine site soil response effects are required by Article 3.10.3.1 for Site Class F soils, the influence of the local soil conditions shall be determined based on site-specific geotechnical investigations and dynamic site response analyses.

For sites located within 6 miles of an active surface or a shallow fault, as depicted in the USGS Active Fault Map, studies shall be considered to quantify near-fault effects on ground motions to determine if these could significantly influence the bridge response.

A deterministic spectrum may be utilized in regions having known active faults if the deterministic spectrum is no less than two-thirds of the probabilistic spectrum in the region of $0.5T_F$ to $2T_F$ of the spectrum where T_F is the bridge fundamental period. Where use of a deterministic spectrum is appropriate, the spectrum shall be either:

- the envelope of a median spectra calculated for characteristic maximum magnitude earthquakes on known active faults; or
- a deterministic spectra may be defined for each fault, and, in the absence of a clearly controlling spectra, each spectrum should be used.

C3.10.2.2

The intent in conducting a site-specific probabilistic ground motion study is to develop ground motions that are more accurate for the local seismic and site conditions than can be determined from national ground motion maps and the procedure of Article 3.10.2.1. Accordingly, such studies should be comprehensive and incorporate current scientific interpretations at a regional scale. Because there are typically scientifically credible alternatives for models and parameter values used to characterize seismic sources and ground-motion attenuation, it is important to incorporate these uncertainties formally in a site-specific probabilistic analysis. Examples of these uncertainties include seismic source location, extent and geometry; maximum earthquake magnitude; earthquake recurrence rate; and ground-motion attenuation relationship.

Near-fault effects on horizontal response spectra include:

- Higher ground motions due to the proximity of the active fault;
- Directivity effects that increase ground motions for periods greater than 0.5 s if the fault rupture propagates toward the site; and
- Directionality effects that increase ground motions for periods greater than 0.5 s in the direction normal (perpendicular) to the strike of the fault.

If the active fault is included and appropriately modeled in the development of national ground motion maps, then the first effect above is already included in the national ground motion maps. The second and third effects are not included in the national maps. These effects are significant only for periods longer than 0.5 s and normally would be evaluated only for essential or critical bridges having natural periods of vibration longer than 0.5 s. Further discussions of the second and third effects are contained in Somerville (1997) and Somerville et al. (1997).

The fault-normal component of near-field ($D < 6$ mi) motion may contain relatively long-duration velocity pulses which can cause severe nonlinear structural response, predictable only through nonlinear time-history analyses. For this case the recorded near-field horizontal components of motion need to be transformed into principal components before modifying them to be response-spectrum-compatible.

The ratio of vertical-to-horizontal ground motions increases for short-period motions in the near-fault environment.

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Where response spectra are determined from a site-specific study, the spectra shall not be lower than two-thirds of the response spectra determined using the general procedure of Article 3.10.2.1 in the region of $0.5T_F$ to $2T_F$ of the spectrum where T_F is the bridge fundamental period.

3.10.3—Site Effects

Site classes and site factors specified herein shall be used in the General Procedure for characterizing the seismic hazard specified in Article 3.10.4.

C3.10.3

The behavior of a bridge during an earthquake is strongly related to the soil conditions at the site. Soils can amplify ground motions in the underlying rock, sometimes by factors of two or more. The extent of this amplification is dependent on the profile of soil types at the site and the intensity of shaking in the rock below. Sites are classified by type and profile for the purpose of defining the overall seismic hazard, which is quantified as the product of the soil amplification and the intensity of shaking in the underlying rock.

3.10.3.1—Site Class Definitions

A site shall be classified as A though F in accordance with the site class definitions in Table 3.10.3.1-1. Sites shall be classified by their stiffness as determined by the shear wave velocity in the upper 100 ft Standard Penetration Test (SPT), blow counts and undrained shear strengths of soil samples from soil borings may also be used to classify sites as indicated in Table 3.10.3.1-1.

C3.10.3.1

Steps that may be followed to classify a site are given in Table C3.10.3.1-1.

Table 3.10.3.1-1—Site Class Definitions

Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, $\bar{v}_s > 5{,}000$ ft/s
B	Rock with 2,500 ft/sec < $\bar{v}_s$ < 5,000 ft/s
C	Very dense soil and soil rock with 1,200 ft/sec < $\bar{v}_s$ < 2,500 ft/s, or with either $\bar{N}$ > 50 blows/ft, or $\bar{s}_u$ > 2.0 ksf
D	Stiff soil with 600 ft/s < $\bar{v}_s$ < 1,200 ft/s, or with either 15 < $\bar{N}$ < 50 blows/ft, or 1.0 < $\bar{s}_u$ < 2.0 ksf
E	Soil profile with $\bar{v}_s$ < 600 ft/s or with either $\bar{N}$ < 15 blows/ft or $\bar{s}_u$ < 1.0 ksf, or any profile with more than 10.0 ft of soft clay defined as soil with $PI > 20$, $w > 40$ percent and $\bar{s}_u$ < 0.5 ksf
F	Soils requiring site-specific evaluations, such as: • Peats or highly organic clays ($H > 10.0$ ft of peat or highly organic clay where H = thickness of soil) • Very high plasticity clays ($H > 25.0$ ft with $PI > 75$) • Very thick soft/medium stiff clays ($H > 120$ ft)

Exceptions: Where the soil properties are not known in sufficient detail to determine the site class, a site investigation shall be undertaken sufficient to determine the site class. Site classes E or F should not be assumed unless the authority having jurisdiction determines that site classes E or F could be present at the site or in the event that site classes E or F are established by geotechnical data.

where:

$\bar{v}_s$ = average shear wave velocity for the upper 100 ft of the soil profile
$\bar{N}$ = average Standard Penetration Test (SPT) blow count (blows/ft) (ASTM D1586) for the upper 100 ft of the soil profile
$\bar{s}_u$ = average undrained shear strength in ksf (ASTM D2166 or ASTM D2850) for the upper 100 ft of the soil profile
PI = plasticity index (ASTM D4318)
w = moisture content (ASTM D2216)

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Table C3.10.3.1-1—Steps for Site Classification

<table>
<tr><th>Step</th><th>Description</th></tr>
<tr><td>1</td><td>Check for the three categories of Site Class F in Table 3.10.3.1-1 requiring site-specific evaluation. If the site corresponds to any of these categories, classify the site as Site Class F and conduct a site-specific evaluation.</td></tr>
<tr><td>2</td><td>Check for existence of a soft layer with total thickness > 10 ft, where soft layer is defined by $s_u < 0.5$ ksf, $w > 40$ percent, and $PI > 20$. If these criteria are met, classify site as Site Class E.</td></tr>
<tr><td>3</td><td>Categorize the site into one of the site classes in Table 3.10.3.1-1 using one of the following three methods to calculate:

• $\bar{v}_s$ for the top 100 ft ($\bar{v}_s$ method)
• $\bar{N}$ for the top 100 ft ($\bar{N}$ method)
• $\bar{N}_{ch}$ for cohesionless soil layers ($PI < 20$) in the top 100 ft and $\bar{s}_u$ for cohesive soil layers ($PI > 20$) in the top 100 ft ($\bar{s}_u$ method)

To make these calculations, the soil profile is subdivided into n distinct soil and rock layers, and in the methods below the symbol i refers to any one of these layers from 1 to n.

Method A: $\bar{v}_s$ method

The average $\bar{v}_s$ for the top 100 ft is determined as:

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}}$$
where:

$$\sum_{i=1}^{n} d_i = 100 \text{ ft}$$
v_{si} = shear wave velocity in ft/s of a layer

d_i = thickness of a layer between 0 and 100 ft

Method B: $\bar{N}$ method

The average $\bar{N}$ for the top 100 ft shall be determined as:

$$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}}$$
where:

N_i = Standard Penetration Test blow count of a layer (not to exceed 100 blows/ft in the above expression)</td></tr>
</table>

Note: When using Method B, $\bar{N}$ values are for cohesionless soils and cohesive soil and rock layers within the upper 100 ft. Where refusal is met for a rock layer, N_i should be taken as 100 blows/ft.

(continued on next page)

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Table C3.10.3.1-1 (continued)—Steps for Site Classification

	Method C: $\bar{s}_u$ method The average $\bar{N}_{ch}$ for cohesionless soil layers in the top 100 ft is determined as: $$\bar{N}_{ch} = \frac{d_s}{\sum_{i=1}^{m} \frac{d_i}{N_{chi}}}$$ in which: $$\sum_{i=1}^{m} d_i = d_s,$$ where: m = number of cohesionless soil layers in the top 100 ft N_{chi} = blow count for a cohesionless soil layer (not to exceed 100 blows/ft in the above expression) d_s = total thickness of cohesionless soil layers in the top 100 ft The average $\bar{s}_u$ for cohesive soil layers in the top 100 ft is determined as: $$\bar{s}_u = \frac{d_c}{\sum_{i=1}^{k} \frac{d_i}{s_{ui}}}$$ in which: $$\sum_{i=1}^{k} d_i = d_c,$$ where: k = number of cohesive soil layers in the top 100 ft s_{ui} = undrained shear strength for a cohesive soil layer (not to exceed 5.0 ksf in the above expression) d_c = total thickness of cohesive soil layers in the top 100 ft

Note: When using Method C, if the site class resulting from $\bar{N}_{ch}$ and $\bar{s}_u$ differ, select the site class that gives the highest site factors and design spectral response in the period range of interest. For example, if $\bar{N}_{ch}$ was equal to 20 blows/ft and $\bar{s}_u$ was equal to 0.8 ksf, the site would classify as D or E in accordance with Method C and the site class definitions of Table 3.10.3.1-1. In this example, for relatively low response spectral acceleration and for long-period motions, Table 3.10.3.2-3 indicates that the site factors are highest for Site Class E. However, for relatively high short-period spectral acceleration (Ss > 0.75), short period site factors, Fa, are higher for Site Class D.

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3.10.3.2—Site Factors

Site Factors F_{pga}, F_a, and F_v specified in Tables 3.10.3.2-1, 3.10.3.2-2, and 3.10.3.2-3 shall be used in the zero-period, short-period range, and long-period range, respectively. These factors shall be determined using the Site Class given in Table 3.10.3.1-1 and the mapped values of the coefficients *PGA*, S_S, and S_1 in Figures 3.10.2.1-1 to 3.10.2.1-21.

C3.10.3.2

Site Class B (soft rock) is taken to be the reference site category for the USGS and NEHRP MCE ground shaking maps. Site class B rock is therefore the site condition for which the site factor is 1.0. Site classes A, C, D, and E have separate sets of site factors for zero-period (F_{pga}), the short-period range (F_a) and long-period range (F_v), as indicated in Tables 3.10.3.2-1, 3.10.3.2-2, and 3.10.3.2-3. These site factors generally increase as the soil profile becomes softer (in going from site class A to E). Except for site class A (hard rock), the factors also decrease as the ground motion level increases, due to the strongly nonlinear behavior of the soil. For a given site class, C, D, or E, these nonlinear site factors increase the ground motion more in areas having lower rock ground motions than in areas having higher rock ground motions.

Table 3.10.3.2-1—Values of Site Factor, F_{pga}, at Zero-Period on Acceleration Spectrum

Site Class	Peak Ground Acceleration Coefficient (*PGA*)[1]				
	PGA < 0.10	*PGA* = 0.20	*PGA* = 0.30	*PGA* = 0.40	*PGA* > 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F[2]	*	*	*	*	*

Notes:

[1]Use straight-line interpolation for intermediate values of *PGA*.

[2]Site-specific geotechnical investigation and dynamic site response analysis should be performed for all sites in Site Class F.

Table 3.10.3.2-2—Values of Site Factor, F_a, for Short-Period Range of Acceleration Spectrum

Site Class	Spectral Acceleration Coefficient at Period 0.2 sec (S_S)[1]				
	S_S < 0.25	S_S = 0.50	S_S = 0.75	S_S = 1.00	S_S > 1.25
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F[2]	*	*	*	*	*

Notes:

[1]Use straight-line interpolation for intermediate values of S_s.

[2]Site-specific geotechnical investigation and dynamic site response analysis should be performed for all sites in Site Class F.

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Table 3.10.3.2-3—Values of Site Factor, F_v, for Long-Period Range of Acceleration Spectrum

Site Class	Spectral Acceleration Coefficient at Period 1.0 sec (S_1)[1]				
	$S_1 <$ 0.1	$S_1 =$ 0.2	$S_1 =$ 0.3	$S_1 =$ 0.4	$S_1 >$ 0.5
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F[2]	*	*	*	*	*

Notes:

[1]Use straight-line interpolation for intermediate values of S_1.

[2]Site-specific geotechnical investigation and dynamic site response analysis should be performed for all sites in Site Class F.

3.10.4—Seismic Hazard Characterization

3.10.4.1—Design Response Spectrum

The five-percent-damped-design response spectrum shall be taken as specified in Figure 3.10.4.1-1. This spectrum shall be calculated using the mapped peak ground acceleration coefficients and the spectral acceleration coefficients from Figures 3.10.2.1-1 to 3.10.2.1-21, scaled by the zero-, short-, and long-period site factors, F_{pga}, F_a, and F_v, respectively.

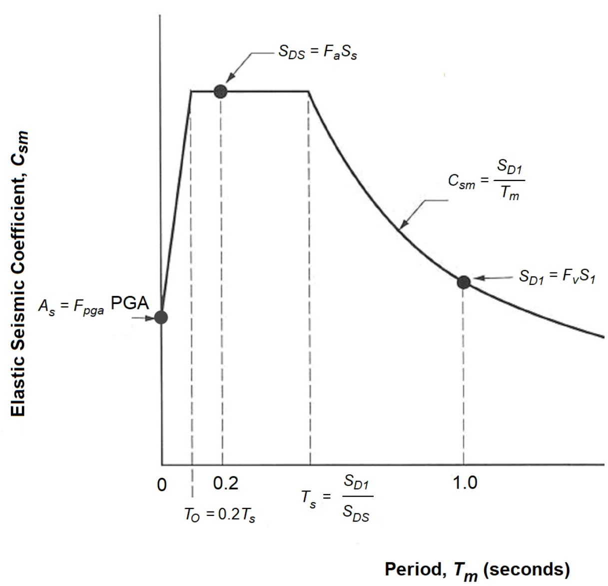

Figure 3.10.4.1-1—Design Response Spectrum

C3.10.4.1

The long-period portion of the response spectrum in Figure 3.10.4.1-1 is inversely proportional to the period, T. In the previous edition of these Specifications, this portion of the spectrum was inversely proportional to $T^{2/3}$. The consequence of this change is that spectral accelerations at periods greater than 1.0 s are smaller than previously specified (for the same ground acceleration and soil type), and greater than previously specified for periods less than 1.0 s (but greater than T_S). This change is consistent with the observed characteristics of response spectra calculated from recorded ground motions. This revised shape is recommended in recent publications by NCHRP (2002, 2006), MCEER/ATC (2003), and FHWA (2006).

For periods exceeding about 3 s, it has been observed that in certain seismic environments spectral displacements tend to a constant value which implies that the acceleration spectrum becomes inversely proportional to T^2 at these periods. As a consequence, the spectrum in Figure 3.10.4.1-1 (and Eq. 3.10.4.2-5) may give conservative results for long period bridges (greater than about 3 s).

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3.10.4.2—Elastic Seismic Response Coefficient

For periods less than or equal to T_0, the elastic seismic coefficient for the *m*th move of vibration, C_{sm}, shall be taken as:

$$C_{sm} = A_S + (S_{DS} - A_S)\,(T_m / T_0) \qquad (3.10.4.2\text{-}1)$$

in which:

$$A_S = F_{pga}\,PGA \qquad (3.10.4.2\text{-}2)$$

$$S_{DS} = F_a\,S_S \qquad (3.10.4.2\text{-}3)$$

where:

PGA = peak ground acceleration coefficient on rock (Site Class B)
S_S = horizontal response spectral acceleration coefficient at 0.2-sec period on rock (Site Class B)
T_m = period of vibration of *m*th mode (s)
T_0 = reference period used to define spectral shape = $0.2\,T_S$ (s)
T_S = corner period at which spectrum changes from being independent of period to being inversely proportional to period = S_{D1}/S_{DS} (s)

For periods greater than or equal to T_0 and less than or equal to T_S, the elastic seismic response coefficient shall be taken as:

$$C_{sm} = S_{DS} \qquad (3.10.4.2\text{-}4)$$

For periods greater than T_S, the elastic seismic response coefficient shall be taken as:

$$C_{sm} = S_{D1} / T_m \qquad (3.10.4.2\text{-}5)$$

in which:

$$S_{D1} = F_v\,S_1 \qquad (3.10.4.2\text{-}6)$$

where:

S_1 = horizontal response spectral acceleration coefficient at 1.0 sec period on rock (Site Class B)

C3.10.4.2

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 illustrates the relationship 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 sets of seismic forces, each corresponding to the period and mode shape of one of the modes of vibration, and the results are combined using acceptable methods, such as the Complete Quadratic Combination method as required in Article 4.7.4.3.3. C_{sm} applies to weight, not mass.

3.10.5—Operational Classification

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

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

C3.10.5

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 1,000-yr 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-yr return period

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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.

event. These bridges should be regarded as critical structures.

3.10.6—Seismic Performance Zones

Each bridge shall be assigned to one of the four seismic zones in accordance with Table 3.10.6-1 using the value of S_{D1} given by Eq. 3.10.4.2-6.

C3.10.6

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.

Table 3.10.6-1—Seismic Zones

Acceleration Coefficient, S_{D1}	Seismic Zone
$S_{D1} \leq 0.15$	1
$0.15 < S_{D1} \leq 0.30$	2
$0.30 < S_{D1} \leq 0.50$	3
$0.50 < S_{D1}$	4

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 and 5.11.

Except as noted herein, seismic design force effects for substructures and the connections between parts of structures, listed in Table 3.10.7.1-2, shall be determined by dividing the force effects resulting from elastic analysis by the appropriate response modification factor, *R*, as specified in Tables 3.10.7.1-1 and 3.10.7.1-2, respectively.

As an alternative to the use of the R-factors, specified in Table 3.10.7.1-2 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 3.10.7.1-2, resulting in a more economical design.

Table 3.10.7.1-1—Response Modification Factors—Substructures

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

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Table 3.10.7.1-2 Response Modification Factors—Connections

Connection	All Operational 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.

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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 acceleration coefficient, A_S, specified in Eq. 3.10.4.2-2, and the tributary permanent load.

Minimum support lengths 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 in Zone 1 where the acceleration coefficient, A_S, as specified in Eq. 3.10.4.2-2, is less than 0.05, the horizontal design connection force in the restrained directions shall not be less than 0.15 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 less than 0.25 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.

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. The minimum connection design forces of this Article are used in lieu of determining such forces through rigorous analysis. The division of Zone 1 at an acceleration coefficient, A_S, of 0.05 recognizes that, in parts of the country with very low seismicity, seismic forces on connections are relatively small. However as outlined below, the intent of this Article is to prevent connections from becoming unintended weak links in the seismic lateral load path. Accordingly, the minimum connection forces specified in this Article are intended to be sufficiently conservative to prevent premature failure and are not intended to precisely reflect the expected dynamic seismic forces. See Article C3.10.7.1 for a description of typical elements considered to be connections, and note that a connection, as considered in this Article, may be an element that simply restrains a member and may not physically connect to that member, such as transverse shear keys. Additionally, anchorage detailing for connections should be extended far enough into the adjacent member to ensure that premature or unintentional local failure is prevented. Similarly, the design of a girder support pedestal should consider the connection forces specified in this Article, since failure of a pedestal located above the pier cap could potentially lead to loss of span support.

In Zone 1, the prevention of superstructure collapse due to unseating of spans is the primary objective behind the provisions for minimum connection forces in restrained directions, as covered by this Article, and for minimum support lengths for unrestrained directions (e.g. expansion bearings), as covered by Article 4.7.4.4. The minimum connection forces specified in this Article are not intended to be minimum design forces for the bridge, because the main elements of the bridge in Zone 1 should generally be capable of resisting the expected lateral seismic forces, by virtue of satisfying the nonseismic design requirements. However, this presumed structural resistance is predicated on providing sufficient integrity and connectivity within the structure to mobilize the lateral resistance of the main

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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.

structural elements (e.g. columns, pier caps, superstructure, abutments, and foundations).

Accordingly, the design forces for connections need only be considered for those elements that directly prevent loss of span support or prevent system instability. Connections that fall into this category include, but are not limited to, those elements restraining the superstructure at in-span hinges and at substructure support locations. Other connections in this category include connections between substructure elements if failure of such connections could lead to loss of span support. For example, failure of the connections between steel piles and a precast concrete bent cap could lead to loss of support for both the cap and superstructure and, therefore, such a connection should meet the requirements of this Article.

If the minimum connection forces are deemed unreasonably large, the design may be completed using the requirements of a higher seismic zone. The minimum requirements of this Article require adequate connection strength for restrained directions and adequate support length in unrestrained directions. In many cases, it is feasible, conservative, and economical to provide both sufficient connection force capacity and support length, and both should be considered. In situations where load sharing of connections may be uncertain, adequate support length, in addition to the required connection force capacity, should be considered. An example is the case of bearings that may not take up load equally, thus leading to the possibility of "unzipping" of the lateral restraint elements. In cases where support length is needed in the transverse direction, the designer is cautioned that the minimum support length equations for *N* were developed empirically considering longitudinal response. Thus, adequate support in the transverse direction should be based on engineering judgment to prevent loss of superstructure support.

If each bearing supporting a continuous segment or simply supported span is an elastomeric bearing, there may be no fully restrained directions due to the flexibility of the bearings. However, the forces transmitted through these bearings should be determined in accordance with this Article and Article 14.6.3. If positive connection capable of transferring the minimum force is not provided, then the minimum support length requirements for expansion bearings of Article 4.7.4.4 should be followed. For this Article, friction is not considered a positive connection due to uncertainty resulting from vertical effects.

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.

The designer is cautioned that in some geographic locations for certain site conditions, spectral accelerations may exceed the minimum connection forces of this Article. Typically, such a condition may occur for structures with fundamental vibration periods at or near the short-period plateau of the response spectra (e.g. stiff structures, such as

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those with wall piers). When this occurs, the designer should consider the effects due to potential connection failure and should consider providing the minimum support lengths of Article 4.7.4.4.

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.
- 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

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.

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.
- 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 ten percent of the value previously determined, use this maximum bent shear force and return to Step 3.

C3.10.9.4.3c

See Article C3.10.9.4.3b.

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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.
- 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.

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.

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

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

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.

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, A_S, as specified in Eq. 3.10.4.2-2, 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.

In lieu of restrainers, STUs may be used and designed for either the elastic force calculated in Article 4.7 or the maximum force effects generated by inelastic hinging of the substructure as specified in Article 3.10.7.1.

3.10.9.6—Hold-Down Devices

For Seismic Zones 2, 3, and 4, hold-down devices shall be provided at supports and at hinges in continuous structures where the vertical seismic force due to the longitudinal seismic load opposes and exceeds 50 percent, but is less than 100 percent, of the reaction due to permanent loads. In this case, the net uplift force for the design of the hold-down device shall be taken as ten percent of the reaction due to permanent loads that would be exerted if the span were simply supported.

If the vertical seismic forces result in net uplift, the hold-down device shall be designed to resist the larger of either:

- 120 percent of the difference between the vertical seismic force and the reaction due to permanent loads, or
- Ten percent of the reaction due to permanent loads.

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3.10.10—Requirements for Temporary Bridges and Stage Construction

Any bridge or partially constructed bridge that is expected to be temporary for more than 5 years shall be designed using the requirements for permanent structures and shall not use the provisions of this Article.

The requirement that an earthquake shall not cause collapse of all or part of a bridge, as stated in Article 3.10.1, shall apply to temporary bridges expected to carry traffic. It shall also apply to those bridges that are constructed in stages and expected to carry traffic and/or pass over routes that carry traffic. The elastic seismic response coefficient and the ground acceleration coefficient given in Article 3.10.4.2 may be reduced by a factor of not more than 2 in order to calculate the component elastic forces and displacements. Response and acceleration coefficients for construction sites that are close to active faults shall be the subject of special study. The response modification factors given in Article 3.10.7 may be increased by a factor of not more than 1.5 in order to calculate the design forces. This factor shall not be applied to connections as defined in Table 3.10.7.1-2.

The minimum support length provisions of Article 4.7.4.4 shall apply to all temporary bridges and staged construction.

C3.10.10

The option to use a reduced response coefficient and a reduced ground acceleration coefficient reflects the limited exposure period for a temporary bridge.

3.11—EARTH PRESSURE: *EH*, *ES*, *LS*, AND *DD*

3.11.1—General

Earth pressure shall be considered as a function of the:

- type and unit weight of earth,
- water content,
- soil creep characteristics,
- degree of compaction,
- location of groundwater table,
- earth-structure interaction,
- amount of surcharge,
- earthquake effects,
- back slope angle, and
- wall inclination.

C3.11.1

Walls that can tolerate little or no movement should be designed for at-rest earth pressure. Walls which can move away from the soil mass should be designed for pressures between active and at-rest conditions, depending on the magnitude of the tolerable movements. Movement required to reach the minimum active pressure or the maximum passive pressure is a function of the wall height and the soil type. Some typical values of these mobilizing movements, relative to wall height, are given in Table C3.11.1-1, where:

Δ = movement of top of wall required to reach minimum active or maximum passive pressure by tilting or lateral translation (ft)

H = height of wall (ft)

Table C3.11.1-1—Approximate Values of Relative Movements Required to Reach Active or Passive Earth Pressure Conditions (Clough and Duncan, 1991)

Type of Backfill	Values of Δ/H	
	Active	Passive
Dense sand	0.001	0.01
Medium dense sand	0.002	0.02
Loose sand	0.004	0.04
Compacted silt	0.002	0.02
Compacted lean clay	0.010	0.05
Compacted fat clay	0.010	0.05

Silt and lean clay shall not be used for backfill unless suitable design procedures are followed and construction control measures are incorporated in the construction documents to account for their presence. Consideration shall be given for the development of pore water pressure within the soil mass in accordance with Article 3.11.3. Appropriate drainage provisions shall be provided to prevent hydrostatic and seepage forces from developing behind the wall in accordance with the provisions in Section 11. In no case shall highly plastic clay be used for backfill.

The evaluation of the stress induced by cohesive soils is highly uncertain due to their sensitivity to shrink-swell, wet-dry and degree of saturation. Tension cracks can form, which considerably alter the assumptions for the estimation of stress. Extreme caution is advised in the determination of lateral earth pressures assuming the most unfavorable conditions. If possible, cohesive or other fine-grained soils should be avoided as backfill.

For walls retaining cohesive materials, the effects of soil creep should be taken into consideration in estimating the design earth pressures. Evaluation of soil creep is complex and requires duplication in the laboratory of the stress conditions in the field as discussed by Mitchell (1976).

Under stress conditions close to the minimum active or maximum passive earth pressures, cohesive soils indicated in Table C3.11.1-1 creep continually, and the movements shown produce active or passive pressures only temporarily. If there is no further movement, active pressures will increase with time, approaching the at-rest pressure, and passive pressures will decrease with time, approaching values on the order of 40 percent of the maximum short-term value. A conservative assumption to account for unknowns would be to use the at-rest pressure based on the residual strength of the soil.

3.11.2—Compaction

Where activity by mechanical compaction equipment is anticipated within a distance of one-half the height of the wall, taken as the difference in elevation between the point where finished grade intersects the back of the wall and the base of the wall, the effect of additional earth pressure that may be induced by compaction shall be taken into account.

C3.11.2

Compaction-induced earth pressures may be estimated using the procedures described by Clough and Duncan (1991). The heavier the equipment used to compact the backfill, and the closer it operates to the wall, the larger are the compaction-induced pressures. The magnitude of the earth pressures exerted on a wall by compacted backfill can be minimized by using only small rollers or hand compactors within a distance of one-half wall height from the back of the wall. For MSE structures, compaction stresses are already included in the design model and specified compaction procedures.

3.11.3—Presence of Water

If the retained earth is not allowed to drain, the effect of hydrostatic water pressure shall be added to that of earth pressure.

In cases where water is expected to pond behind a wall, the wall shall be designed to withstand the hydrostatic water pressure plus the earth pressure.

Submerged unit weights of the soil shall be used to determine the lateral earth pressure below the groundwater table.

C3.11.3

The effect of additional pressure caused by groundwater is shown in Figure C3.11.3-1.

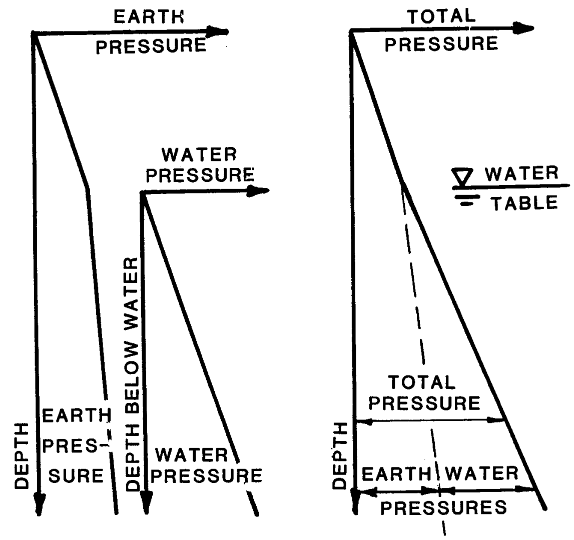

Figure C3.11.3-1—Effect of Groundwater Table

The development of hydrostatic water pressure on walls should be eliminated through use of crushed rock, pipe drains, gravel drains, perforated drains or geosynthetic drains.

If the groundwater levels differ on opposite sides of the wall, the effects of seepage on wall stability and the potential for piping shall be considered. Pore water pressures shall be added to the effective horizontal stresses in determining total lateral earth pressures on the wall.

Pore water pressures behind the wall may be approximated by flow net procedures or various analytical methods.

3.11.4—Effect of Earthquake

The effects of wall inertia and probable amplification of active earth pressure and/or mobilization of passive earth masses by earthquake shall be considered.

C3.11.4

The Mononobe-Okabe method for determining equivalent static fluid pressures for seismic loads on gravity and semigravity retaining walls is presented in Appendix A11.

The Mononobe-Okabe analysis is based, in part, on the assumption that the backfill soils are unsaturated and thus, not susceptible to liquefaction.

Where soils are subject to both saturation and seismic or other cyclic/instantaneous loads, special consideration should be given to address the possibility of soil liquefaction.

3.11.5—Earth Pressure: *EH*

3.11.5.1—Lateral Earth Pressure

C3.11.5.1

Lateral earth pressure shall be assumed to be linearly proportional to the depth of earth and taken as:

$$p = k\gamma_s z \qquad (3.11.5.1\text{-}1)$$

where:

p = lateral earth pressure (ksf)
k = coefficient of lateral earth pressure taken as k_o, specified in Article 3.11.5.2, for walls that do not

deflect or move; k_a, specified in Articles 3.11.5.3, 3.11.5.6 and 3.11.5.7, for walls that deflect or move sufficiently to reach minimum active conditions; or k_p, specified in Article 3.11.5.4, for walls that deflect or move sufficiently to reach a passive condition

γ_s = unit weight of soil (kcf)

z = depth below the surface of earth (ft)

The resultant lateral earth load due to the weight of the backfill shall be assumed to act at a height of $H/3$ above the base of the wall, where H is the total wall height, measured from the surface of the ground at the back of the wall to the bottom of the footing or the top of the leveling pad (for MSE walls).

Although previous versions of these Specifications have required design of conventional gravity walls for a resultant earth pressure located 0.4 H above the wall base, the current specifications require design for a resultant located $H/3$ above the base. This requirement is consistent with historical practice and with calibrated resistance factors in Section 11. The resultant lateral load due to the earth pressure may act as high as 0.4 H above the base of the wall for a mass concrete gravity retaining wall, where H is the total wall height measured from the top of the backfill to the base of the footing, where the wall deflects laterally, i.e., translates, in response to lateral earth loading. For such structures, the backfill behind the wall must slide down along the back of the wall for the retained soil mass to achieve the active state of stress. Experimental results indicate that the backfill arches against the upper portion of the wall as the wall translates, causing an upward shift in the location at which the resultant of the lateral earth load is transferred to the wall (Terzaghi, 1934; Clausen and Johansen et al., 1972; Sherif et al., 1982). Such walls are not representative of typical gravity walls used in highway applications.

For most gravity walls which are representative of those used in highway construction, nongravity cantilever retaining walls or other flexible walls which tilt or deform laterally in response to lateral loading, e.g., MSE walls, as well as walls which cannot translate or tilt, e.g., integral abutment walls, significant arching of the backfill against the wall does not occur, and the resultant lateral load due to earth pressure acts at a height of $H/3$ above the base of the wall. Furthermore, where wall friction is not considered in the analysis, it is sufficiently conservative to use a resultant location of $H/3$, even if the wall can translate.

3.11.5.2—At-Rest Lateral Earth Pressure Coefficient, k_o

For normally consolidated soils, vertical wall, and level ground, the coefficient of at-rest lateral earth pressure may be taken as:

$$k_o = 1 - \sin \phi'_f \quad (3.11.5.2\text{-}1)$$

where:

ϕ'_f = effective friction angle of soil

k_o = coefficient of at-rest lateral earth pressure

C3.11.5.2

For typical cantilevered walls over 5.0 ft high with structural grade backfill, calculations indicate that the horizontal movement of the top of the wall due to a combination of structural deformation of the stem and rotation of the foundation is sufficient to develop active conditions.

In many instances, the OCR may not be known with enough accuracy to calculate k_o using Eq. 3.11.5.2-2. Based on information on this issue provided by Holtz and Kovacs (1981), in general, for lightly overconsolidated sands (OCR = 1 to 2), k_o is in the range of 0.4 to 0.6. For

For overconsolidated soils, the coefficient of at-rest lateral earth pressure may be assumed to vary as a function of the overconsolidation ratio or stress history, and may be taken as:

$$k_o = (1 - \sin\phi'_f)(OCR)^{\sin\phi'_f} \qquad (3.11.5.2\text{-}2)$$

where:

OCR = overconsolidation ratio

highly overconsolidated sand, k_o can be on the order of 1.0.

Silt and lean clay shall not be used for backfill unless suitable design procedures are followed and construction control measures are incorporated in the construction documents to account for their presence. Consideration must be given for the development of pore water pressure within the soil mass in accordance with Article 3.11.3. Appropriate drainage provisions shall be provided to prevent hydrostatic and seepage forces from developing behind the wall in accordance with the provisions of Section 11. In no case shall highly plastic clay be used for backfill.

The evaluation of the stress induced by cohesive soils is highly uncertain due to their sensitivity to shrink-swell, wet-dry and degree of saturation. Tension cracks can form, which considerably alter the assumptions for the estimation of stress. Extreme caution is advised in the determination of lateral earth pressures assuming the most unfavorable conditions. See Article C3.11.1 for additional guidance on estimating earth pressures in fine-grained soils. If possible, cohesive or other fine-grained soils should be avoided as backfill.

3.11.5.3—Active Lateral Earth Pressure Coefficient, k_a

Values for the coefficient of active lateral earth pressure may be taken as:

$$k_a = \frac{\sin^2(\theta + \phi'_f)}{\Gamma\,[\sin^2\theta\,\sin(\theta - \delta)]} \qquad (3.11.5.3\text{-}1)$$

in which:

$$\Gamma = \left[1 + \sqrt{\frac{\sin(\phi'_f + \delta)\sin(\phi'_f - \beta)}{\sin(\theta - \delta)\sin(\theta + \beta)}}\right]^2 \qquad (3.11.5.3\text{-}2)$$

where:

δ = friction angle between fill and wall (degrees)
β = angle of fill to the horizontal as shown in Figure 3.11.5.3-1 (degrees)
θ = angle of back face of wall to the horizontal as shown in Figure 3.11.5.3-1 (degrees)
ϕ'_f = effective angle of internal friction (degrees)

For conditions that deviate from those described in Figure 3.11.5.3-1, the active pressure may be calculated by using a trial procedure based on wedge theory using the Culmann method (e.g., see Terzaghi et al., 1996).

C3.11.5.3

The values of k_a by Eq. 3.11.5.3-1 are based on the Coulomb earth pressure theories. The Coulomb theory is necessary for design of retaining walls for which the back face of the wall interferes with the development of the full sliding surfaces in the backfill soil assumed in Rankine theory (Figure C3.11.5.3-1 and Article C3.11.5.3). Either Coulomb or Rankine wedge theory may be used for long heeled cantilever walls shown in Figure C3.11.5.3-1a. In general, Coulomb wedge theory applies for gravity, semigravity, prefabricated modular, and MSE walls, and concrete cantilever walls with short heels.

Table C3.11.5.3-1 provides typical values for the nominal interface friction angle δ between the back of the wall and backfill for a range of materials and soils. These values are presumptive in nature and are therefore likely to be conservative. Alternatively, long-term practice has been to use $\delta = 0.67\phi'_f$ at the interface between the back of the wall and the retained soil to calculate k_a for soil against concrete as well as soil against soil. For soil against steel, $\delta = 0.33\ \phi'_f$ has typically been used. This is usually conservative for noncohesive soils. For sliding resistance calculations, Article 10.6.3.4 indicates that $\tan\delta = 0.80\tan\phi'_f$ should be used for footing sliding resistance, if the footing is precast concrete, and $\tan\delta = \tan\phi_f$ if the footing concrete is cast directly on the foundation soil. Additional information on friction values of various materials against soil is provided in Potyondy (1961), which is a key source for these friction values. In the absence of specific test data such as measured ϕ_f from laboratory testing, ϕ_f determined through correlation to in-situ measured SPT, or cone resistance values, the values in Table C3.11.5.3-1 or

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$\delta = 0.67\phi'_f$ may be used in computations that include effects of wall friction. To estimate sliding resistance along the base of a wall or footing foundation, Table C3.11.5.3-1, $\tan \delta = 0.80 \tan \phi'_f$ if the footing is precast concrete, or $\tan \delta = \tan \phi_f$ if the footing concrete is cast directly on the foundation soil may be used. Based on the work by Potyondy (1961), all these values are likely to be conservative.

If the wall friction acting on the back of the wall is for soil against soil, such as occurs for semigravity and MSE walls, theoretically δ could be as high as the soil friction angle. However, even for semigravity cantilever walls in which most of the wall friction surface assumed is soil on soil, the wall friction is usually limited to $0.67\phi'_f$ of the reinforced or retained soil, whichever is lower, for design. For sliding resistance, the reduction in friction angle at the interface with the wall base or structure footing is typically applied to $\tan \phi'_f$ rather than directly to ϕ'_f (e.g., see Article 10.6.3.4 and commentary with regard to sliding frictional resistance). However, for wall friction used to determine the k_a value, δ is used directly in the rather complex k_a equation, and it is more practical to simply reduce ϕ'_f.

For the cantilever wall in Figure C3.11.5.3-1b, the earth pressure is applied to a plane extending vertically up from the heel of the wall base, and the weight of soil to the left of the vertical plane is considered as part of the wall weight.

The differences between the Coulomb theory currently specified, and the Rankine theory specified in the past is illustrated in Figure C3.11.5.3-1. The Rankine theory is the basis of the equivalent fluid method of Article 3.11.5.5.

Silt and lean clay should not be used for backfill where free-draining granular materials are available. When using poorly draining silts or cohesive soils, extreme caution is advised in the determination of lateral earth pressures assuming the most unfavorable conditions. Consideration must be given for the development of pore water pressure within the soil mass in accordance with Article 3.11.3. Appropriate drainage provisions should be provided to prevent hydrostatic and seepage forces from developing behind the wall in accordance with the provisions in Section 11. In no case should highly plastic clay be used for backfill.

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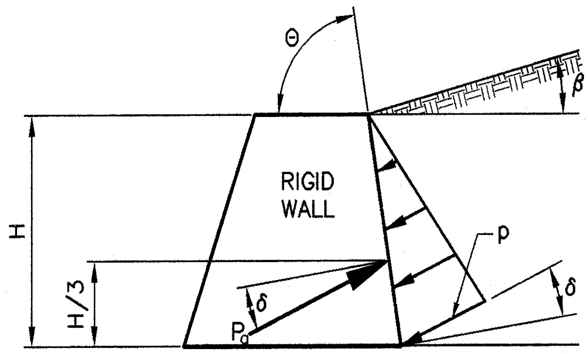

Figure 3.11.5.3-1—Notation for Coulomb Active Earth Pressure

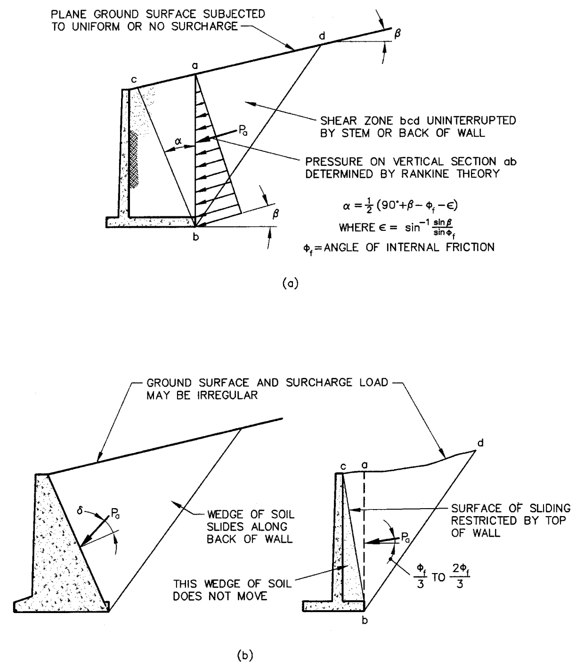

Figure C3.11.5.3-1—Application of (a) Rankine and (b) Coulomb Earth Pressure Theories in Retaining Wall Design

Table C3.11.5.3-1—Friction Angle for Dissimilar Materials (U.S. Department of the Navy, 1982a)

Interface Materials	Friction Angle, δ (degrees)	Coefficient of Friction, tan δ (dim.)
Mass concrete on the following foundation materials:		
• Clean sound rock	35	0.70
• Clean gravel, gravel-sand mixtures, coarse sand	29 to 31	0.55 to 0.60
• Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	24 to 29	0.45 to 0.55
• Clean fine sand, silty or clayey fine to medium sand	19 to 24	0.34 to 0.45
• Fine sandy silt, nonplastic silt	17 to 19	0.31 to 0.34
• Very stiff and hard residual or preconsolidated clay	22 to 26	0.40 to 0.49
• Medium stiff and stiff clay and silty clay	17 to 19	0.31 to 0.34
Masonry on foundation materials has same friction factors.		
Steel sheet piles against the following soils:		
• Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	22	0.40
• Clean sand, silty sand-gravel mixture, single-size hard rock fill	17	0.31
• Silty sand, gravel or sand mixed with silt or clay	14	0.25
• Fine sandy silt, nonplastic silt	11	0.19
Formed or precast concrete or concrete sheet piling against the following soils:		
• Clean gravel, gravel-sand mixture, well-graded rock fill with spalls	22 to 26	0.40 to 0.49
• Clean sand, silty sand-gravel mixture, single-size hard rock fill	17 to 22	0.31 to 0.40
• Silty sand, gravel or sand mixed with silt or clay	17	0.31
• Fine sandy silt, nonplastic silt	14	0.25
Various structural materials:		
• Masonry on masonry, igneous and metamorphic rocks:		
○ dressed soft rock on dressed soft rock	35	0.70
○ dressed hard rock on dressed soft rock	33	0.65
○ dressed hard rock on dressed hard rock	29	0.55
• Masonry on wood in direction of cross grain	26	0.49
• Steel on steel at sheet pile interlocks	17	0.31

3.11.5.4—Passive Lateral Earth Pressure Coefficient, k_p

For noncohesive soils, values of the coefficient of passive lateral earth pressure may be taken from Figure 3.11.5.4-1 for the case of a sloping or vertical wall with a horizontal backfill or from Figure 3.11.5.4-2 for the case of a vertical wall and sloping backfill. For conditions that deviate from those described in Figures 3.11.5.4-1 and 3.11.5.4-2, the passive pressure may be calculated by using a trial procedure based on wedge theory, e.g., see Terzaghi et al. (1996). When wedge theory is used, the limiting value of the wall friction angle should not be taken larger than one-half the angle of internal friction, ϕ_f.

For cohesive soils, passive pressures may be estimated by:

C3.11.5.4

The movement required to mobilize passive pressure is approximately 10.0 times as large as the movement needed to induce earth pressure to the active values. The movement required to mobilize full passive pressure in loose sand is approximately five percent of the height of the face on which the passive pressure acts. For dense sand, the movement required to mobilize full passive pressure is smaller than five percent of the height of the face on which the passive pressure acts, and five percent represents a conservative estimate of the movement required to mobilize the full passive pressure. For poorly compacted cohesive soils, the movement required to mobilize full passive pressure is larger than five percent of the height of the face on which the pressure acts.

$$p_p = k_p \gamma_s z + 2c\sqrt{k_p} \qquad (3.11.5.4\text{-}1)$$

where:

p_p = passive lateral earth pressure (ksf)
γ_s = unit weight of soil (kcf)
z = depth below surface of soil (ft)
c = soil cohesion (ksf)
k_p = coefficient of passive lateral earth pressure specified in Figures 3.11.5.4-1 and 3.11.5.4-2, as appropriate

$$k_p = R\,k'_p \qquad (3.11.5.4\text{-}2)$$

where:

k_p = coefficient of passive lateral earth pressure
R = reduction factor for coefficient of passive lateral earth pressure for various ratios of $-\delta/\phi_f$
k'_p = intermediate coefficient of passive lateral earth pressure determined from Figures 3.11.5.4-1 and 3.11.5.4-2

Wedge solutions are inaccurate and unconservative for larger values of wall friction angle.

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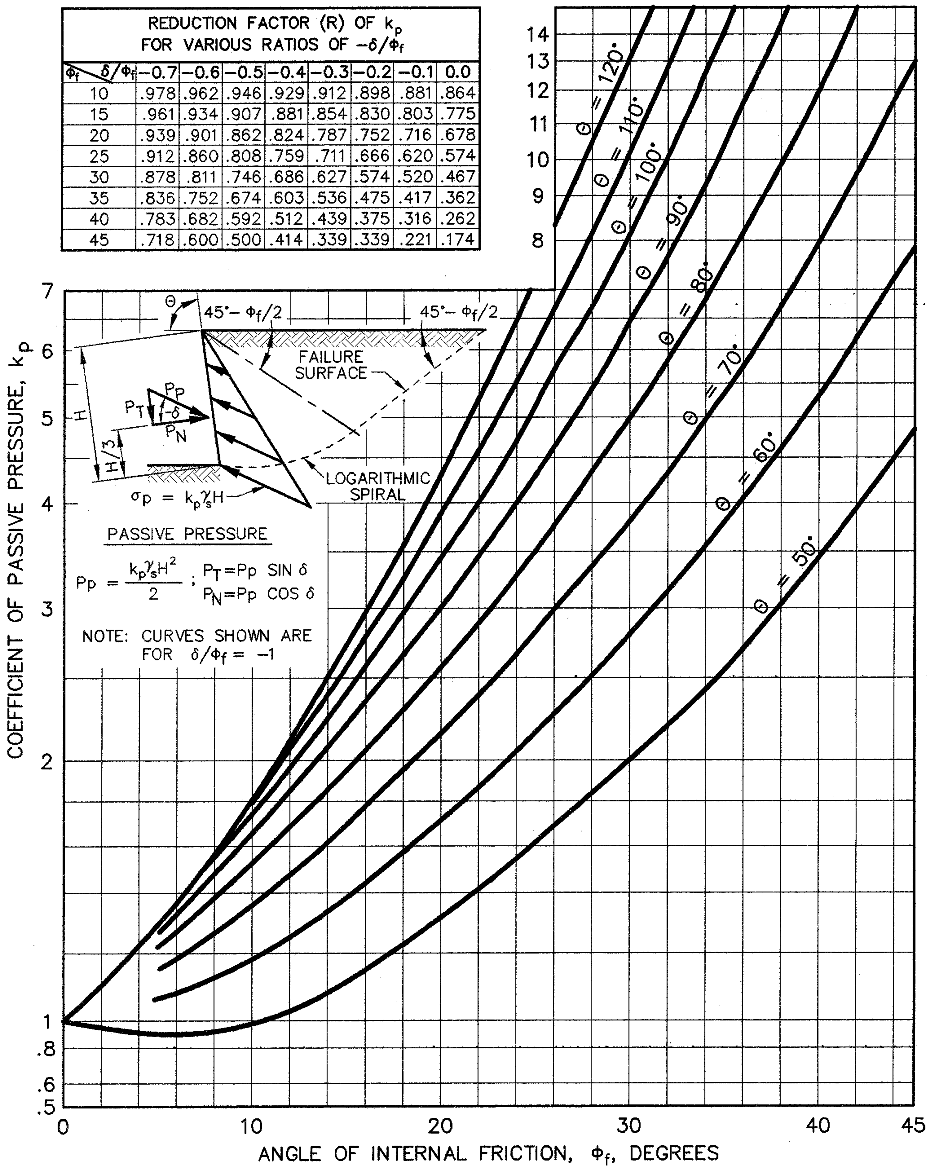

REDUCTION FACTOR (R) OF k_p FOR VARIOUS RATIOS OF $-\delta/\phi_f$

ϕ_f \ δ/ϕ_f	−0.7	−0.6	−0.5	−0.4	−0.3	−0.2	−0.1	0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.339	.221	.174

Figure 3.11.5.4-1—Computational Procedures for Passive Earth Pressures for Vertical and Sloping Walls with Horizontal Backfill (U.S. Department of the Navy, 1982a)

REDUCTION FACTOR (R) OF k_p FOR VARIOUS RATIOS OF $-\delta/\phi_f$

ϕ_f \ δ/ϕ_f	−0.7	−0.6	−0.5	−0.4	−0.3	−0.2	−0.1	0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
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25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.339	.221	.174

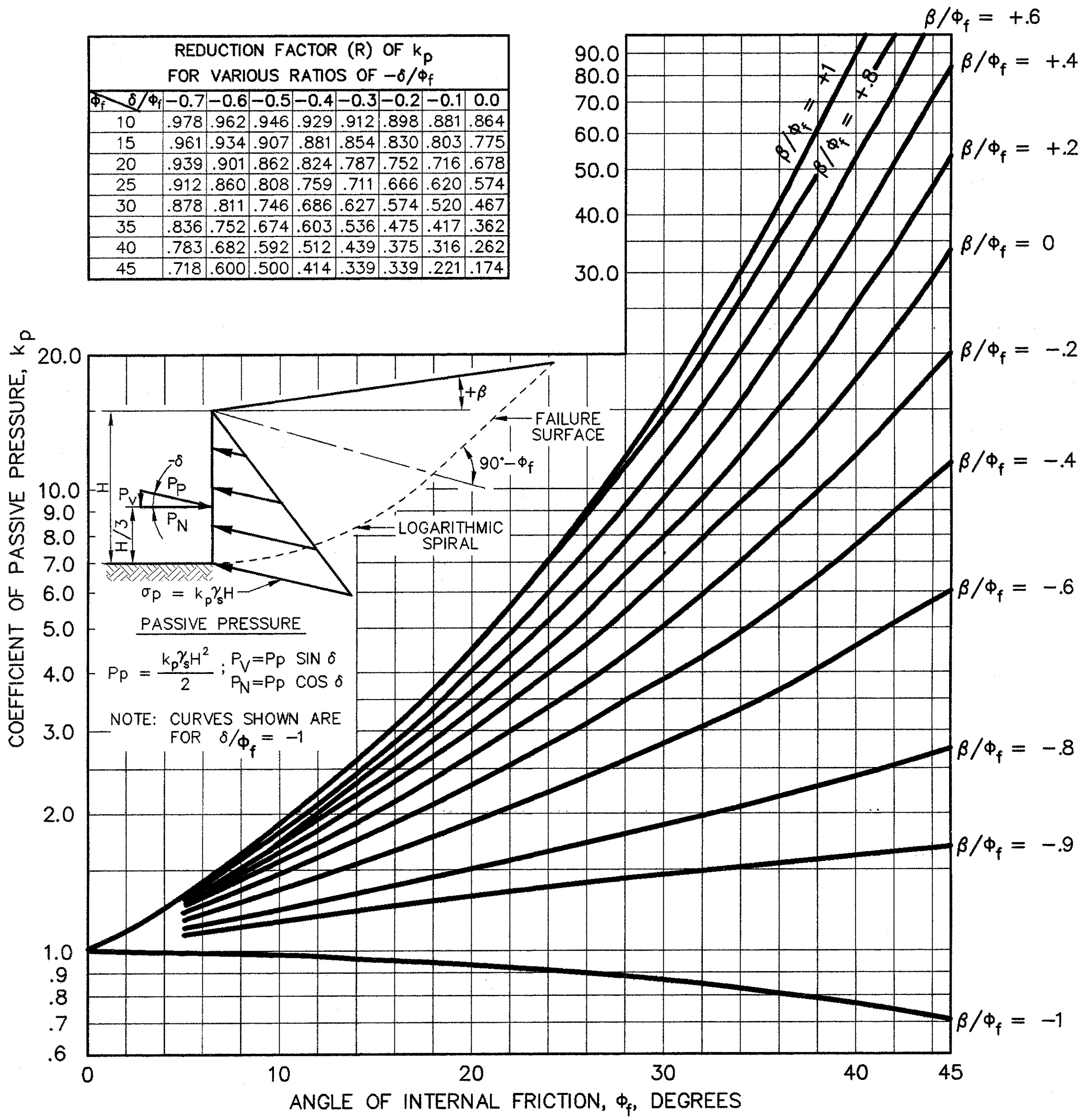

Figure 3.11.5.4-2—Computational Procedures for Passive Earth Pressures for Vertical Wall with Sloping Backfill (U.S. Department of the Navy, 1982a)

3.11.5.5—Equivalent-fluid Method of Estimating Rankine Lateral Earth Pressures

The equivalent-fluid method may be used where Rankine earth pressure theory is applicable.

The equivalent-fluid method shall only be used where the backfill is free-draining. If this criterion cannot be satisfied, the provisions of Articles 3.11.3, 3.11.5.1, and 3.11.5.3 shall be used to determine horizontal earth pressure.

C3.11.5.5

Applicability of Rankine theory is discussed in Article C3.11.5.3.

Values of the unit weights of equivalent fluids are given for walls that can tolerate very little or no movement as well as for walls that can move as much as 1.0 in. in 20.0 ft. The concepts of equivalent fluid unit weights have taken into account the effect of soil creep on walls.

Where the equivalent-fluid method is used, the basic earth pressure, p (ksf), may be taken as:

$$p = \gamma_{eq} z \quad (3.11.5.5\text{-}1)$$

where:

γ_{eq} = equivalent fluid unit weight of soil, not less than 0.030 (kcf)
z = depth below surface of soil (ft)

The resultant lateral earth load due to the weight of the backfill shall be assumed to act at a height of $H/3$ above the base of the wall, where H is the total wall height, measured from the surface of the ground to the bottom of the footing.

Typical values for equivalent fluid unit weights for design of a wall of height not exceeding 20.0 ft may be taken from Table 3.11.5.5-1, where:

Δ = movement of top of wall required to reach minimum active or maximum passive pressure by tilting or lateral translation (ft)
H = height of wall (ft)
β = angle of fill to the horizontal (degrees)

The magnitude of the vertical component of the earth pressure resultant for the case of sloping backfill surface may be determined as:

$$P_v = P_h \tan\beta \quad (3.11.5.5\text{-}2)$$

where:

$$P_h = 0.5\gamma_{eq} H^2 \quad (3.11.5.5\text{-}3)$$

If the backfill qualifies as free-draining (i.e., granular material with less than 5 percent passing a No. 200 sieve), water is prevented from creating hydrostatic pressure.

For discussion on the location of the resultant of the lateral earth force see Article C3.11.5.1.

The values of equivalent fluid unit weight presented in Table 3.11.5.5-1 for Δ/H = 1/240 represent the horizontal component of active earth pressure based on Rankine earth pressure theory. This horizontal earth pressure is applicable for cantilever retaining walls for which the wall stem does not interfere with the sliding surface defining the Rankine failure wedge within the wall backfill (Figure C3.11.5.3-1). The horizontal pressure is applied to a vertical plane extending up from the heel of the wall base, and the weight of soil to the left of the vertical plane is included as part of the wall weight.

For the case of a sloping backfill surface in Table 3.11.5.5-1, a vertical component of earth pressure also acts on the vertical plane extending up from the heel of the wall.

Table 3.11.5.5-1—Typical Values for Equivalent Fluid Unit Weights of Soils

	Level Backfill		Backfill with β = 25 degrees	
Type of Soil	At-Rest γ_{eq} (kcf)	Active Δ/H = 1/240 γ_{eq} (kcf)	At-Rest γ_{eq} (kcf)	Active Δ/H = 1/240 γ_{eq} (kcf)
Loose sand or gravel	0.055	0.040	0.065	0.050
Medium dense sand or gravel	0.050	0.035	0.060	0.045
Dense sand or gravel	0.045	0.030	0.055	0.040

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3.11.5.6—Lateral Earth Pressures for Nongravity Cantilevered Walls

For permanent walls, the simplified lateral earth pressure distributions shown in Figures 3.11.5.6-1 through 3.11.5.6-3 may be used. If walls will support or are supported by cohesive soils for temporary applications, walls may be designed based on total stress methods of analysis and undrained shear strength parameters. For this latter case, the simplified earth pressure distributions shown in Figures 3.11.5.6-4 through 3.11.5.6-7 may be used with the following restrictions:

- The ratio of total overburden pressure to undrained shear strength, N_s (see Article 3.11.5.7.2), should be <3 at the wall base.
- The active earth pressure shall not be less than 0.25 times the effective overburden pressure at any depth, or 0.035 ksf/ft of wall height, whichever is greater.

For temporary walls with discrete vertical elements embedded in granular soil or rock, Figures 3.11.5.6-1 and 3.11.5.6-2 may be used to determine passive resistance and Figures 3.11.5.6-4 and 3.11.5.6-5 may be used to determine the active earth pressure due to the retained soil.

Where discrete vertical wall elements are used for support, the width, *b*, of each vertical element shall be assumed to equal the width of the flange or diameter of the element for driven sections and the diameter of the concrete-filled hole for sections encased in concrete.

The magnitude of the sloping surcharge above the wall for the determination of P_{a2} in Figure 3.11.5.6-4 should be based on the wedge of soil above the wall within the active wedge.

In Figure 3.11.5.6-5, a portion of negative loading at top of wall due to cohesion is ignored and hydrostatic pressure in a tension crack should be considered, but is not shown on the figure.

C3.11.5.6

Nongravity cantilevered walls temporarily supporting or supported by cohesive soils are subject to excessive lateral deformation if the undrained soil shear strength is low compared to the shear stresses. Therefore, use of these walls should be limited to soils of adequate strength as represented by the stability number N_s (see Article 3.11.5.7.2).

Base movements in the soil in front of a wall become significant for values of N_s of about 3 to 4, and a base failure can occur when N_s exceeds about 5 to 6 (Terzaghi and Peck, 1967).

In Figures 3.11.5.6-1, 3.11.5.6-2, 3.11.5.6-4, and 3.11.5.6-5, the width *b* of discrete vertical wall elements effective in mobilizing the passive resistance of the soil is based on a method of analysis by Broms (1964a, 1964b) for single vertical piles embedded in cohesive or cohesionless soil and assumes a vertical element. The effective width for passive resistance of three times the element width, 3*b*, is due to the arching action in soil and side shear on resisting rock wedges. The maximum width of 3*b* can be used when material in which the vertical element is embedded does not contain discontinuities that would affect the failure geometry. This width should be reduced if planes or zones of weakness would prevent mobilization of resistance through this entire width, or if the passive resistance zones of adjacent elements overlap. If the element is embedded in soft clay having a stability number less than three, soil arching will not occur and the actual width shall be used as the effective width for passive resistance. Where a vertical element is embedded in rock, i.e., Figure 3.11.5.6-2, the passive resistance of the rock is assumed to develop through the shear failure of a rock wedge equal in width to the vertical element, *b*, and defined by a plane extending upward from the base of the element at an angle of 45 degrees. For the active zone behind the wall below the mudline or groundline in front of the wall, the active pressure is assumed to act over one vertical element width, *b*, in all cases.

The Broms (1964a, 1964b) approach of accounting for soil arching assumed the use of Rankine theory passive pressure coefficients. Therefore, Rankine theory passive earth pressure coefficients should be used when using soil arching to increase the effective width of the vertical elements. When in the unusual case the ground slope β' is

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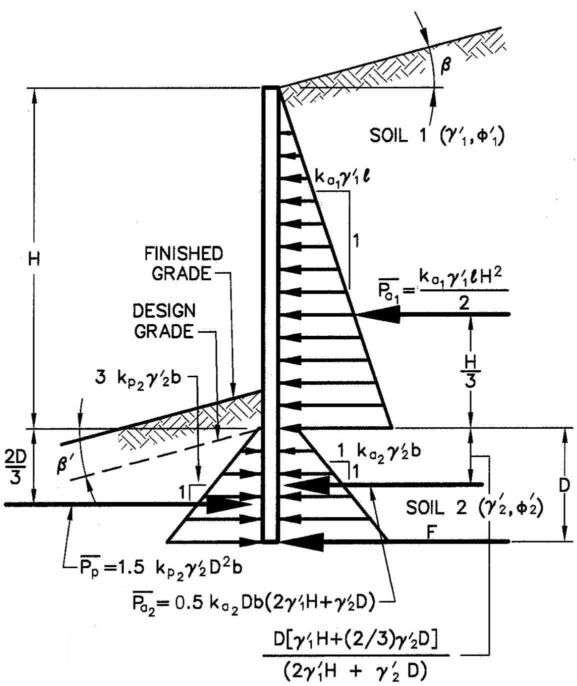


b = ACTUAL WIDTH OF EMBEDDED DISCRETE VERTICAL WALL ELEMENT BELOW DESIGN GRADE IN PLANE OF WALL (FT.).

Figure 3.11.5.6-1—Unfactored Simplified Earth Pressure Distributions for Permanent Nongravity Cantilevered Walls with Discrete Vertical Wall Elements Embedded in Granular Soil

negative (i.e., the ground slopes up away from the wall face), due to the trigonometric functions used in Eq. 3.11.5.6-1, the passive earth pressure coefficient decreases again as the ground slope becomes more negative. Hence, it is recommended that the ground surface in front of the wall be assumed to be flat when β' is negative.

The design grade is generally taken below finished grade to account for excavation during or after wall construction or other disturbance to the supporting soil during the service life of the wall.

In Figures 3.11.5.6-3, 3.11.5.6-6, and 3.11.5.6-7, the depth of embedment for the continuous vertical wall elements is shown as $D \approx 1.2\ D_o$. In these cases, a simplified method of design (see Article C11.8.4.1) is used which simplifies some computational work, but results in a small error in the calculated embedment depth, where D_o is slightly smaller than D calculated by a more rigorous calculation method. Typical practice has been to increase this depth by approximately 20 percent to accommodate the small error caused by this simplification.

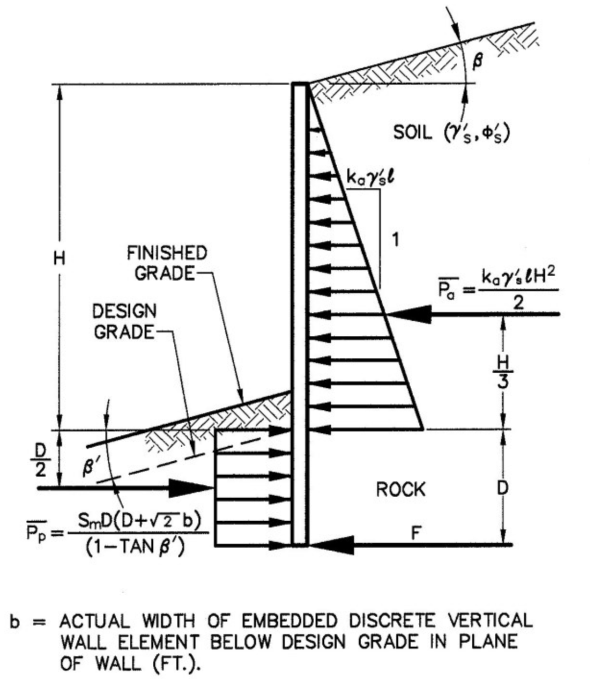


Figure 3.11.5.6-2—Unfactored Simplified Earth Pressure Distributions for Permanent Nongravity Cantilevered Walls with Discrete Vertical Wall Elements Embedded in Rock

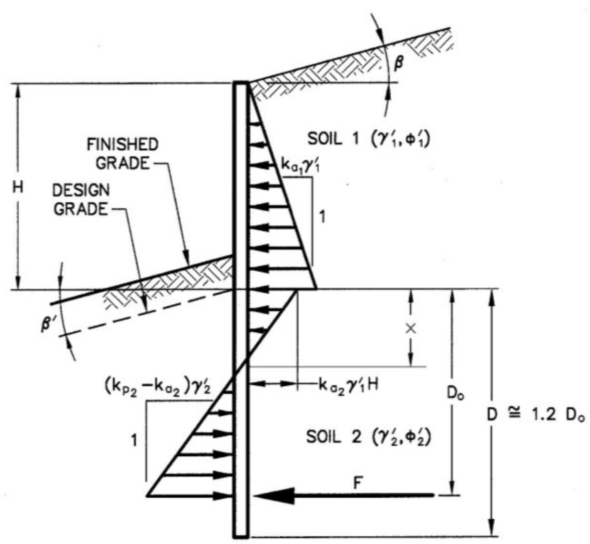


Figure 3.11.5.6-3—Unfactored Simplified Earth Pressure Distributions for Permanent Nongravity Cantilevered Walls with Continuous Vertical Wall Elements Embedded in Granular Soil Modified after Teng (1962)

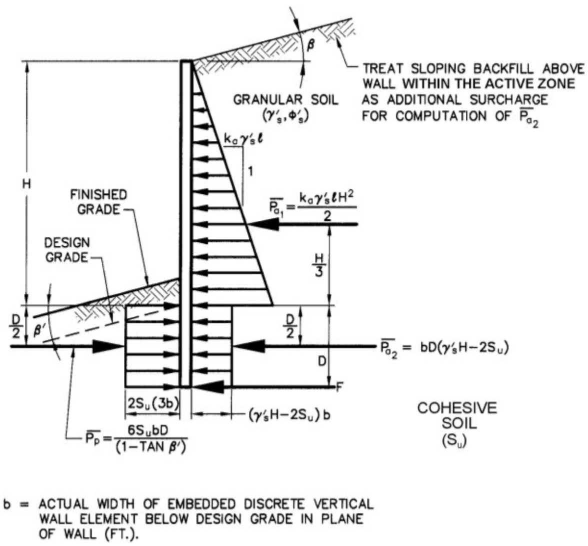


Figure 3.11.5.6-4—Unfactored Simplified Earth Pressure Distributions for Temporary Nongravity Cantilevered Walls with Discrete Vertical Wall Elements Embedded in Cohesive Soil and Retaining Granular Soil

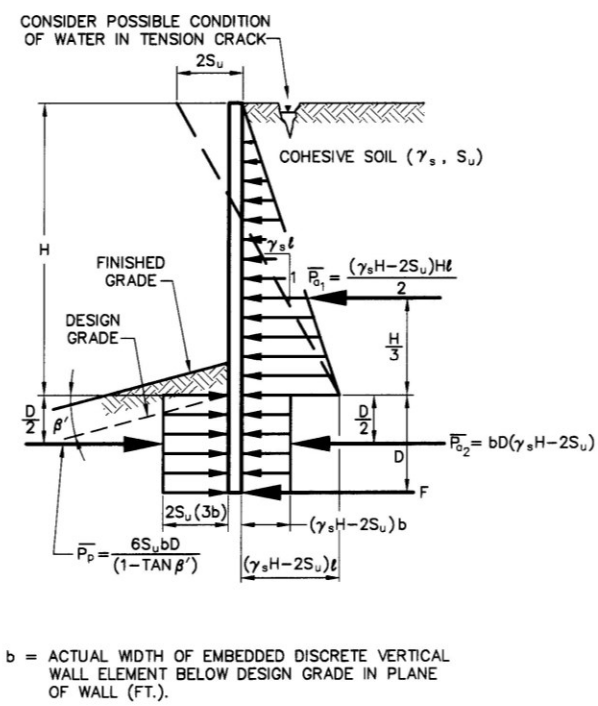
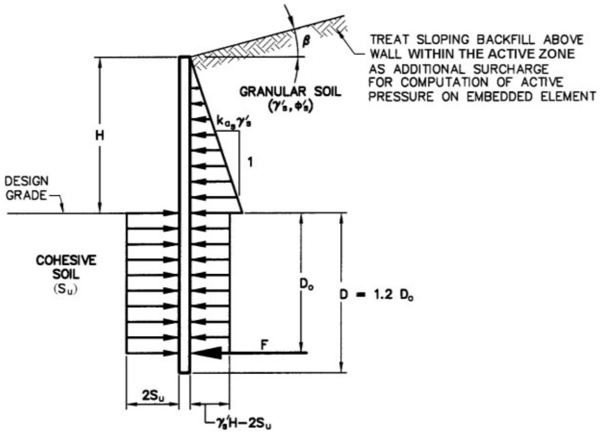


Figure 3.11.5.6-5—Unfactored Simplified Earth Pressure Distributions for Temporary Nongravity Cantilevered Walls with Discrete Vertical Wall Elements Embedded in Cohesive Soil and Retaining Cohesive Soil



Note: For walls embedded in granular soil, refer to Figure 3.11.5.6.3-3 and use Figure 3.11.5.6-7 for retained cohesive soil when appropriate.

Figure 3.11.5.6-6—Unfactored Simplified Earth Pressure Distributions for Temporary Nongravity Cantilevered Walls with Continuous Vertical Wall Elements Embedded in Cohesive Soil and Retaining Granular Soil Modified after Teng (1962)

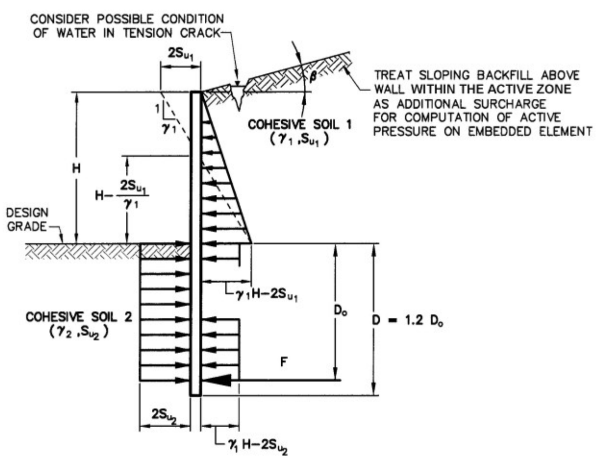


Figure 3.11.5.6-7—Unfactored Simplified Earth Pressure Distributions for Temporary Nongravity Cantilevered Walls with Continuous Vertical Wall Elements Embedded in Cohesive Soil and Retaining Cohesive Soil Modified after Teng (1962)

If the effective width of the discrete vertical elements used to calculate passive resistance is greater than the actual width of the vertical element b due to soil arching, Rankine theory shall be used to calculate the passive lateral earth pressure coefficient if the ground in front of the wall is level or sloping downhill away from the wall. For sloping ground in granular soils, the following Rankine theory based equation shall be used to calculate K_p in this case:

$$K_p = \frac{\cos\beta' + \sqrt{\left(\cos^2\beta' - \cos^2\phi_p\right)}}{\cos\beta' - \sqrt{\left(\cos^2\beta' - \cos^2\phi_p\right)}} \qquad (3.11.5.6\text{-}1)$$

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where:

β' = slope angle for finished grade, defined as positive as shown in Figure 3.11.5.6-1 (degrees)
ϕ_p = soil friction angle for soil within passive resistance zone (degrees)

If, in the unusual case, β' is negative (i.e., sloping up away from the wall in the passive zone), the ground in front of the wall should be assumed to be flat.

3.11.5.7—Apparent Earth Pressure (*AEP*) for Anchored Walls

For anchored walls constructed from the top down, the earth pressure may be estimated in accordance with Articles 3.11.5.7.1 or 3.11.5.7.2.

In developing the design pressure for an anchored wall, consideration shall be given to wall displacements that may affect adjacent structures and/or underground utilities.

C3.11.5.7

In the development of lateral earth pressures, the method and sequence of construction, the rigidity of the wall/anchor system, the physical characteristics and stability of the ground mass to be supported, allowable wall deflections, anchor spacing and prestress and the potential for anchor yield should be considered.

Several suitable apparent earth pressure distribution diagrams are available and in common use for the design of anchored walls, Sabatini et al. (1999); Cheney (1988); and U. S. Department of the Navy (1982a). Some of the apparent earth pressure diagrams, such as those described in Articles 3.11.5.7.1 and 3.11.5.7.2, are based on the results of measurements on anchored walls, Sabatini et al. (1999). Others are based on the results of measurements on strutted excavations, Terzaghi and Peck (1967), the results of analytical and scale model studies, Clough and Tsui (1974); Hanna and Matallana (1970), and observations of anchored wall installations (Nicholson et al., 1981); Schnabel (1982). While the results of these efforts provide somewhat different and occasionally conflicting results, they all tend to confirm the presence of higher lateral pressures near the top of the wall than would be predicted by classical earth pressure theories, due to the constraint provided by the upper level of anchors, and a generally uniform pressure distribution with depth.

3.11.5.7.1—Cohesionless Soils

The earth pressure on temporary or permanent anchored walls constructed in cohesionless soils may be determined using Figure 3.11.5.7.1-1, for which the maximum ordinate, p_a, of the pressure diagram is computed as follows:

For walls with one anchor level:

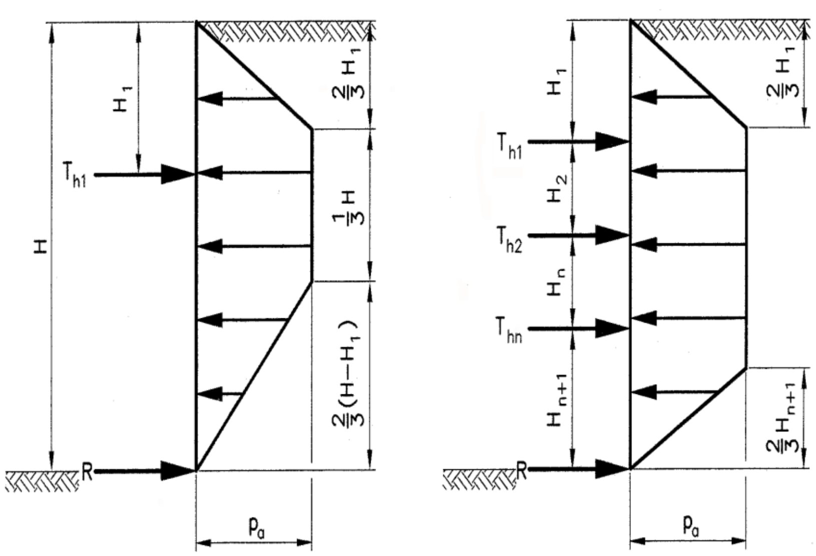
$$p_a = k_a \gamma'_s H \tag{3.11.5.7.1-1}$$

For walls with multiple anchor levels:

$$p_a = \frac{k_a \gamma'_s H^2}{1.5H - 0.5H_1 - 0.5H_{n+1}} \tag{3.11.5.7.1-2}$$

where:

p_a = maximum ordinate of pressure diagram (ksf)
k_a = active earth pressure coefficient
= $\tan^2$ (45 degrees – $\phi_f/2$) (dim.) for $\beta = 0$
use Eq. 3.11.5.3-1 for $\beta \neq 0$
γ'_s = effective unit weight of soil (kcf)
H = total excavation depth (ft)
H_1 = distance from ground surface to uppermost ground anchor (ft)
H_{n+1}= distance from base of excavation to lowermost ground anchor (ft)
T_{hi} = horizontal load in ground anchor i (kip/ft)
R = reaction force to be resisted by subgrade (i.e., below base of excavation) (kip/ft)



(a) Wall with one level of ground anchors

(b) Walls with multiple levels of ground anchors

Figure 3.11.5.7.1-1—Apparent Earth Pressure Distributions for Anchored Walls Constructed from the Top Down in Cohesionless Soils

3.11.5.7.2—Cohesive Soils

The apparent earth pressure distribution for cohesive soils is related to the stability number, N_s, which is defined as:

$$N_s = \frac{\gamma_s H}{S_u} \qquad (3.11.5.7.2\text{-}1)$$

where:

γ_s = total unit weight of soil (kcf)
H = total excavation depth (ft)
S_u = average undrained shear strength of soil (ksf)

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3.11.5.7.2a—Stiff to Hard

For temporary anchored walls in stiff to hard cohesive soils ($N_s \leq 4$), the earth pressure may be determined using Figure 3.11.5.7.1-1, with the maximum ordinate, p_a, of the pressure diagram computed as:

$$p_a = 0.2\gamma_s H \text{ to } 0.4\gamma_s H \qquad (3.11.5.7.2a\text{-}1)$$

where:

p_a = maximum ordinate of pressure diagram (ksf)
γ_s = total unit weight of soil (kcf)
H = total excavation depth (ft)

For permanent anchored walls in stiff to hard cohesive soils, the apparent earth pressure distributions described in Article 3.11.5.7.1 may be used with k_a based on the drained friction angle of the cohesive soil. For permanent walls, the distribution, permanent or temporary, resulting in the maximum total force shall be used for design.

C3.11.5.7.2a

The determination of earth pressures in cohesive soils described in this Article and Article 3.11.5.7.2b are based on the results of measurements on anchored walls, Sabatini et al. (1999). In the absence of specific experience in a particular deposit, $p_a = 0.3\gamma_s H$ should be used for the maximum pressure ordinate when ground anchors are locked off at 75 percent of the unfactored design load or less. Where anchors are to be locked off at 100 percent of the unfactored design load or greater, a maximum pressure ordinate of $p_a = 0.4\,\gamma_s\,H$ should be used.

For temporary walls, the apparent earth pressure distribution in Figure 3.11.5.7.1-1 should only be used for excavations of controlled short duration, where the soil is not fissured and where there is no available free water.

Temporary loading may control design of permanent walls and should be evaluated in addition to permanent loading.

3.11.5.7.2b—Soft to Medium Stiff

The earth pressure on temporary or permanent walls in soft to medium stiff cohesive soils ($N_s \geq 6$) may be determined using Figure 3.11.5.7.2b-1, for which the maximum ordinate, p_a, of the pressure diagram is computed as:

$$p_a = k_a \gamma_s H \qquad (3.11.5.7.2b\text{-}1)$$

where:

p_a = maximum ordinate of pressure diagram (ksf)
k_a = active earth pressure coefficient from Eq. 3.11.5.7.2b-2
γ_s = total unit weight of soil (kcf)
H = total excavation depth (ft)

The active earth pressure coefficient, k_a, may be determined by:

$$k_a = 1 - \frac{4S_u}{\gamma_s H} + 2\sqrt{2}\,\frac{d}{H}\left(1 - \frac{5.14 S_{ub}}{\gamma_s H}\right) \geq 0.22 \qquad (3.11.5.7.2b\text{-}2)$$

where:

S_u = undrained strength of retained soil (ksf)
S_{ub} = undrained strength of soil below excavation base (ksf)
γ_s = total unit weight of retained soil (kcf)
H = total excavation depth (ft)

C3.11.5.7.2b

For soils with $4 < N_s < 6$, use the larger p_a from Eq. 3.11.5.7.2a-1 and Eq. 3.11.5.7.2b-1.

d = depth of potential base failure surface below base of excavation (ft)

The value of d is taken as the thickness of soft to medium stiff cohesive soil below the excavation base up to a maximum value of $B_e/\sqrt{2}$, where B_e is the excavation width.

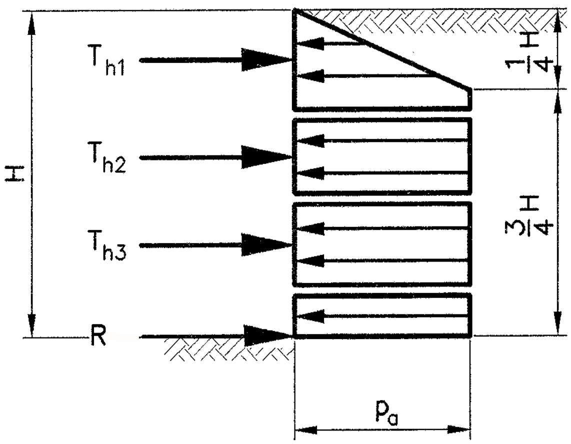

Figure 3.11.5.7.2b-1—Apparent Earth Pressure Distribution for Anchored Walls Constructed from the Top Down in Soft to Medium Stiff Cohesive Soils

3.11.5.8—Lateral Earth Pressures for Mechanically Stabilized Earth Walls

3.11.5.8.1—General

The resultant force per unit width behind an MSE wall, shown in Figures 3.11.5.8.1-1, C3.11.5.8.1-1, and 3.11.5.8.1-2 as acting at a height of h/3 above the base of the wall, shall be taken as:

$$P_a = 0.5k_a\gamma_s h^2 \qquad (3.11.5.8.1\text{-}1)$$

where:

P_a = force resultant per unit width (kip/ft)
γ_s = total unit weight of backfill (kcf)
h = height of horizontal earth pressure diagram taken as shown in Figures 3.11.5.8.1-1, C3.11.5.8.1-1, and 3.11.5.8.1-2 (ft)
k_a = active earth pressure coefficient specified in Article 3.11.5.3, with the angle of backfill slope taken as β, and the friction angle between soil zones taken as δ as specified in Figures 3.11.5.8.1-1, C3.11.5.8.1-1, and 3.11.5.8.1-2. δ should be no greater than $0.67\phi'_f$ of the reinforced or retained soil zone, whichever is lower.

For "broken back" soil surcharge conditions, a generalized limit equilibrium (GLE) analysis, as presented in Article A11.3.3, or a Coulomb trial wedge analysis should be used.

C3.11.5.8.1

MSE wall Figures 3.11.5.8.1-1 and 3.11.5.8.1-2 are based on a Coulomb external load model consistent with the analysis of all other gravity walls. For MSE walls, the back of the reinforced soil zone is assumed to define the plane upon which wall friction acts. See Article C3.11.5.3 for guidance and background on the determination of wall friction angles for calculating earth pressure coefficients. Rankine theory may be used to calculate k_a for external stability evaluation using δ = β of the finished slope. Rankine theory will typically produce more conservative results. However, the Rankine δ = β should only be used in combination with Rankine theory, and δ = the angle of friction between the back of the wall and the soil behind the wall should only be used with Coulomb theory.

For "broken back" soil surcharge conditions, the Coulomb load model also permits a direct solution using a trial wedge analysis. A simplified alternative for broken back surcharge conditions is also provided in Figure C3.11.5.8.1-1 and may be used, since long-term practice has been to use this simplified approach. This simplified approach may be used with Rankine theory as well, but with δ replaced with β'.

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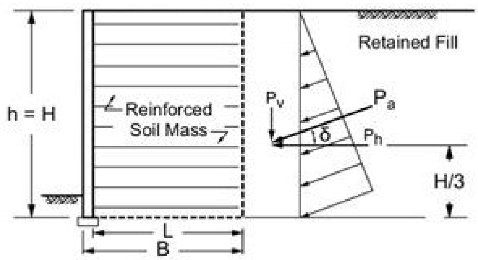

Figure 3.11.5.8.1-1—Earth Pressure Distribution for MSE Wall with Level Backfill Surface

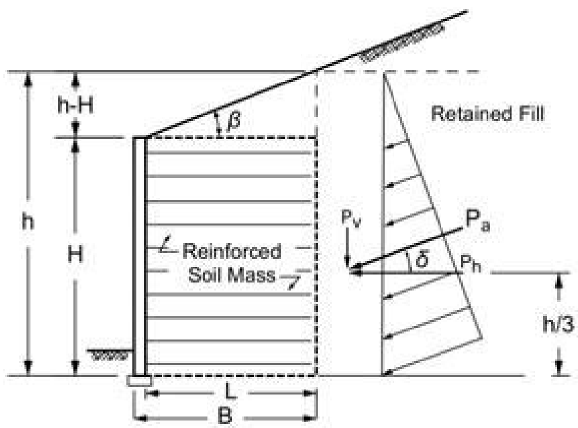

Figure 3.11.5.8.1-2—Earth Pressure Distribution for MSE Wall with Sloping Backfill Surface

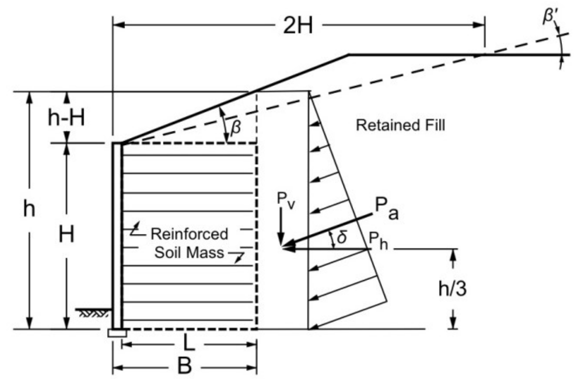

Figure C3.11.5.8.1-1—Earth Pressure Distribution for MSE Wall with Broken Back Earth Surcharge

3.11.5.8.2—Internal Stability

The load factor γ_p to be applied to the maximum load carried by the reinforcement T_{max} for reinforcement strength, connection strength, and pullout calculations (see Article 11.10.6.2) shall be $\gamma_{p\text{-}EV}$, for vertical earth pressure.

For MSE walls, η_i shall be taken as 1.

C3.11.5.8.2

Loads carried by the soil reinforcement in mechanically stabilized earth walls are the result of vertical and lateral earth pressures which exist within the reinforced soil mass, reinforcement extensibility, facing stiffness, wall toe restraint, and the stiffness and strength of the soil backfill within the reinforced soil mass. The calculation method for T_{max} is empirically derived, based on reinforcement strain measurements, converted to load based on the reinforcement stiffness, from full scale walls during or at end of wall construction. The primary source of these reinforcement loads is the gravity forces acting on the reinforced soil mass plus any soil surcharge present. Hence, the load factor $\gamma_{p\text{-EV}}$ as specified in Table 3.4.1-2 is used for the soil loads acting on the reinforcements. If there are concentrated surcharge loads acting on the reinforced soil mass (e.g., structure footings – see Article 3.11.6.2), such surcharge loads should be added to the gravity loads acting on the reinforcement by superposition, and the appropriate load factor applied to the concentrated surcharge load from Table 3.4.1-2.

For MSE wall internal stability using the Stiffness Method, when addressing the soil failure, the load factor is designated as $\gamma_{p\text{-}EVsf}$. This load factor was developed for the Service Limit (see Article 11.10.4.3).

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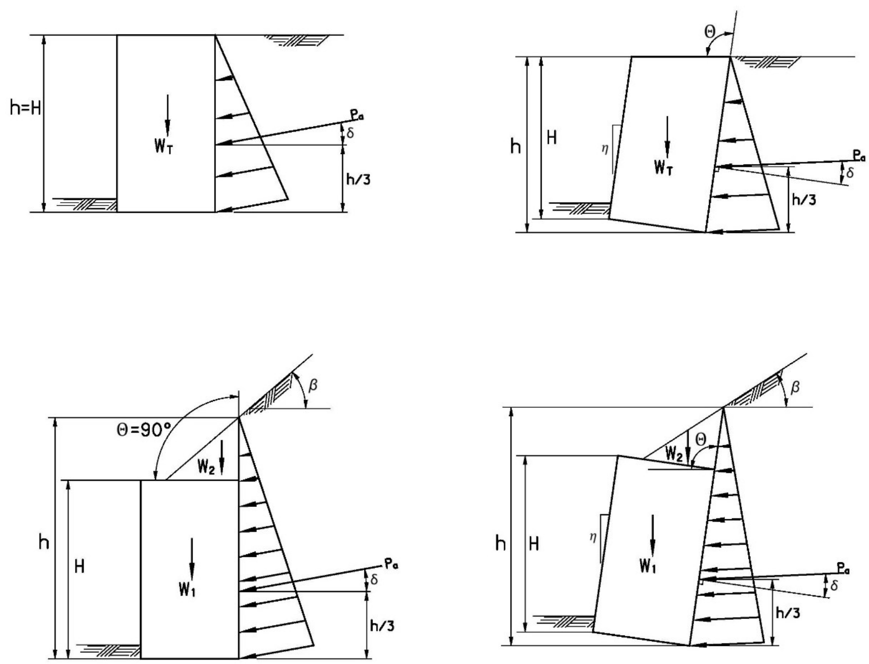
3.11.5.9—Lateral Earth Pressures for Prefabricated Modular Walls

The magnitude and location of resultant loads and resisting forces for prefabricated modular walls may be determined using the earth pressure distributions presented in Figures 3.11.5.9-1 and 3.11.5.9-2. Where the back of the prefabricated modules forms an irregular, stepped surface, the earth pressure shall be computed on a plane surface drawn from the upper back corner of the top module to the lower back heel of the bottom module using Coulomb earth pressure theory.

C3.11.5.9

Prefabricated modular walls are gravity walls constructed of prefabricated concrete elements that are in-filled with soil. They differ from modular block MSE structures in that they contain no soil reinforcing elements.



$$P_a @ \frac{h}{3}$$

$$P_a = \frac{1}{2}\gamma_s' h^2 k_a$$

Figure 3.11.5.9-1—Earth Pressure Distributions for Prefabricated Modular Walls with Continuous Pressure Surfaces

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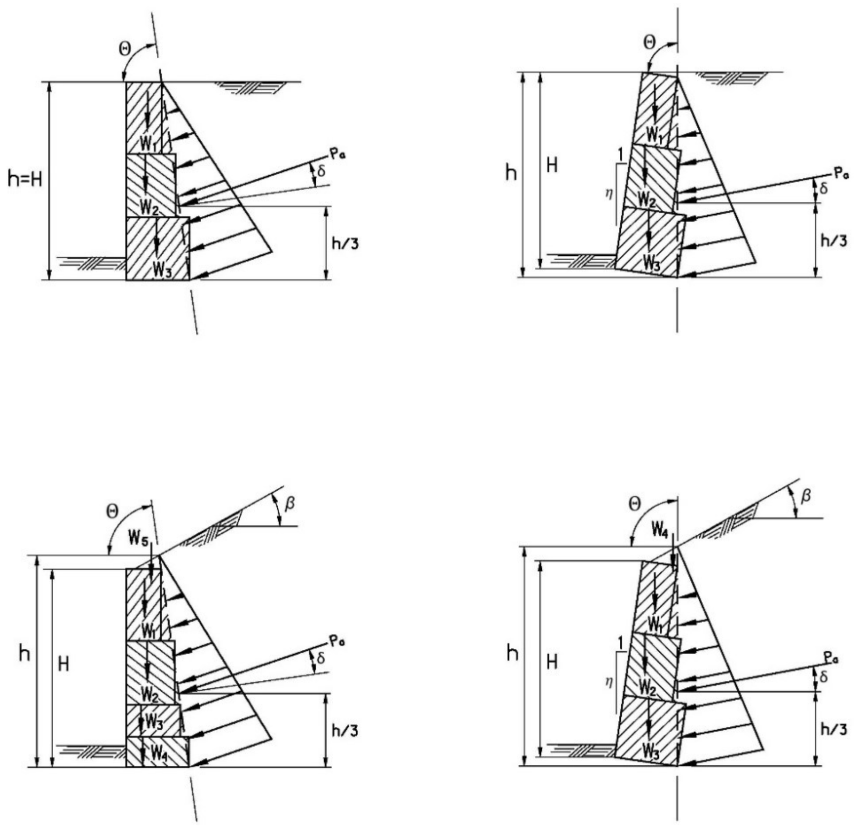

Figure 3.11.5.9-2—Earth Pressure Distributions for Prefabricated Modular Walls with Irregular Pressure Surfaces

The value of k_a used to compute lateral thrust resulting from retained backfill and other loads behind the wall shall be computed based on the friction angle of the backfill behind the modules. In the absence of specific data, if granular backfill is used behind the prefabricated modules within a zone of at least $1V{:}1H$ from the heel of the wall, a value of 34 degrees may be used for ϕ_f. Otherwise, without specific data, a maximum friction angle of 30 degrees shall be used.

The wall friction angle, δ, is a function of the direction and magnitude of possible movements, and the properties of the backfill. When the structure settles more than the backfill, the wall friction angle is negative.

As a maximum, the wall friction angles, given in Table C3.11.5.9-1, should be used to compute k_a, unless more exact coefficients are demonstrated:

Table C3.11.5.9-1—Maximum Wall Friction Angles, δ

Case	Wall Friction Angle (δ)
Modules settle more than backfill	0
Continuous pressure surface of precast concrete (uniform width modules)	$0.50\ \phi_f$
Average pressure surface (stepped modules)	$0.75\ \phi_f$

3.11.5.10—Lateral Earth Pressures for Sound Barriers Supported on Discrete and Continuous Vertical Embedded Elements

For sound barriers supported on discrete vertical wall elements embedded in granular soil, rock, or cohesive soil,

C3.11.5.10

Earth pressure on foundations of sound barriers is similar to that on nongravity retaining walls discussed in the simplified lateral earth pressure distributions shown in Figures 3.11.5.10-1, 3.11.5.10-2, and 3.11.5.10-3, respectively, may be used. For sound barriers supported on continuous vertical elements embedded in granular soil or cohesive soil, the simplified earth pressure distributions shown in Figures 3.11.5.10-4 and 3.11.5.10-5, respectively, may be used. For sound barriers supported on retaining walls, the applicable provisions of Section 11 shall apply.

Where discrete vertical elements are used for support, the width, *b*, of each vertical element shall be assumed to equal the width of the flange or diameter of the element for driven sections and the diameter of the concrete-filled hole for sections encased in concrete.

The reversal in the direction of applied lateral forces on sound barriers shall be considered in the design.

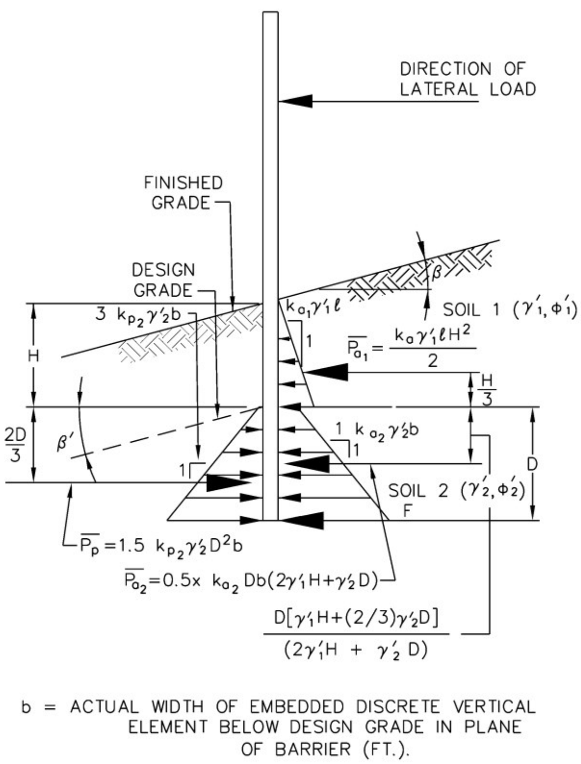

Figure 3.11.5.10-1—Unfactored Simplified Earth Pressure Distributions for Discrete Vertical Wall Elements Embedded in Granular Soil

Article 3.11.5.6 except that the soil elevation on both sides of the wall is often the same or, if there is a difference, does not reach the top of the wall on one side. The provisions of this Article are applicable to the foundations of any wall that is not primarily intended to retain earth, i.e. there is no or little difference in the elevation of fill on either side of the wall.

In Figures 3.11.5.10-1 and 3.11.5.10-3, the width, *b,* of discrete vertical elements effective in mobilizing the passive resistance of the soil is based on a method of analysis by Broms (1964a, *1964b*) for single vertical piles embedded in cohesive or granular soil. Additional information on the background of the earth pressure on discrete vertical elements is presented in Article C3.11.5.6.

The main applied lateral forces on sound barriers are wind and seismic forces; both of them are reversible. When the ground surface in front of or behind the sound barrier, or both, is not flat or the ground surface is not at the same elevation on both sides of the sound barrier, the design should be checked assuming that the lateral force is applied in either direction. The effect of the direction of ground surface slope, i.e. toward the barrier or away from the barrier, should be considered in earth pressure calculations for both directions of lateral loads. The earth pressure diagrams shown in Figures 3.11.5.10-1 through 3.11.5.10-5 correspond to the lateral load direction shown in these figures.

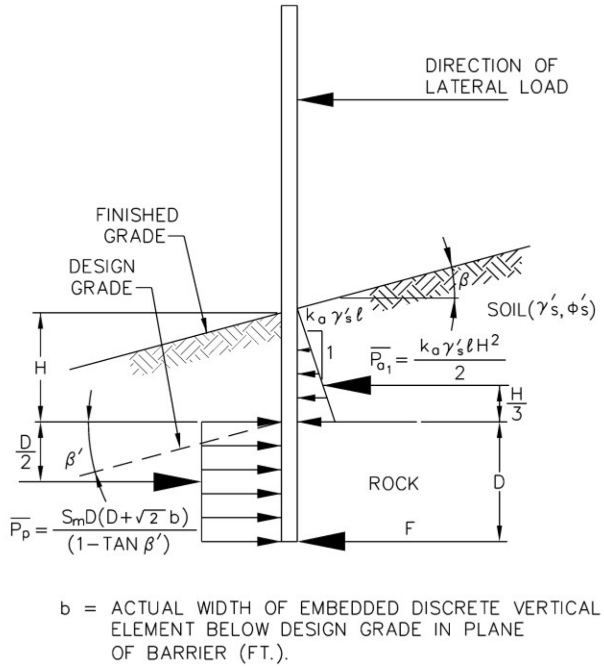

Figure 3.11.5.10-2—Unfactored Simplified Earth Pressure Distributions for Discrete Vertical Wall Elements Embedded in Rock

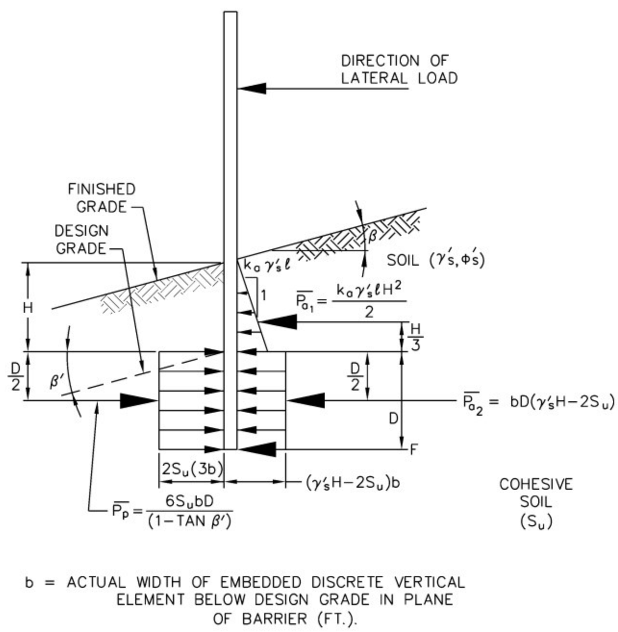

Figure 3.11.5.10-3—Unfactored Simplified Earth Pressure Distributions for Discrete Vertical Wall Elements Embedded in Cohesive Soil

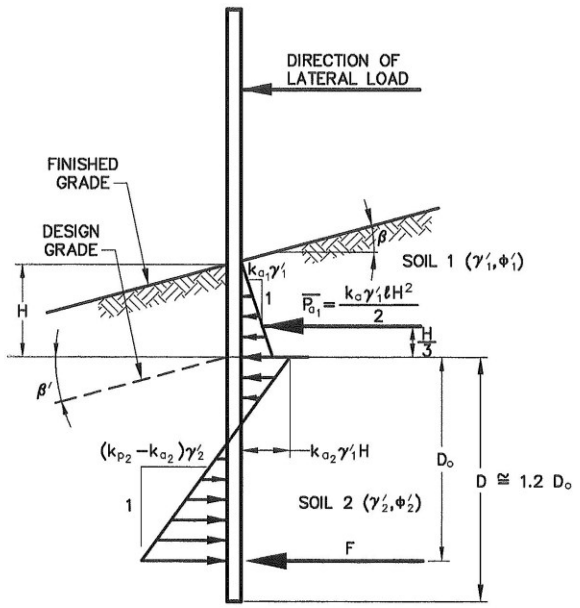

Figure 3.11.5.10-4—Unfactored Simplified Earth Pressure Distributions for Continuous Vertical Elements Embedded in Granular Soil Modified after Teng (1962)

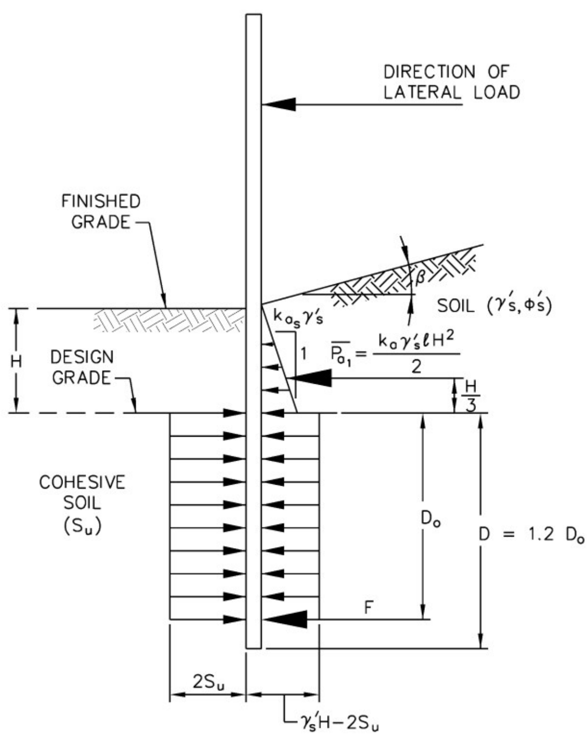

Figure 3.11.5.10-5—Unfactored Simplified Earth Pressure Distributions for Continuous Vertical Wall Elements Embedded in Cohesive Soil Modified after Teng (1962)

3.11.6—Surcharge Loads: *ES* and *LS*

The factored soil stress increase behind or within the wall caused by concentrated surcharge loads or stresses shall be the greater of (1) the unfactored surcharge loads or stresses multiplied by the specified load factor, *ES*, or (2) the factored loads for the structure as applied to the

C3.11.6

Concentrated surcharge loads induced by foundations are typically the result of dead load, live load, wind load, and possibly other loads that are associated with load factors other than *ES*. However, the controlling uncertainty in load prediction for surcharges is the structural element causing the surcharge load, setting *ES* to 1.0. The load applied to the wall due to the structural element above the wall shall not be double factored.

transmission of the surcharge load through the soil to the wall or other structure below the surcharge. Hence, *ES* should be applied to the unfactored concentrated surcharge loads, unless the combined effect of the factored loads applicable to the foundation unit transmitting load to the top of the wall is more conservative. In this latter case, *ES* should be set equal to 1.0 and the factored footing loads used as the concentrated surcharge load in the wall design.

3.11.6.1—Uniform Surcharge Loads (*ES*)

Where a uniform surcharge is present, a constant horizontal earth pressure shall be added to the basic earth pressure. This constant earth pressure may be taken as:

$$\Delta_p = k_s q_s \tag{3.11.6.1-1}$$

where:

Δ_p = constant horizontal earth pressure due to uniform surcharge (ksf)
k_s = coefficient of earth pressure due to surcharge
q_s = uniform surcharge applied to the upper surface of the active earth wedge (ksf)

For active earth pressure conditions, k_s shall be taken as k_a, and for at-rest conditions, k_s shall be taken as k_o. Otherwise, intermediate values appropriate for the type of backfill and amount of wall movement may be used.

C3.11.6.1

When the uniform surcharge is produced by an earth loading on the upper surface, the load factor for both vertical and horizontal components shall be taken as specified in Table 3.4.1-2 for earth surcharge.

Wall movement needed to mobilize extreme active and passive pressures for various types of backfill can be found in Table C3.11.1-1.

3.11.6.2—Point, Line, and Strip Loads (*ES*): Walls Restrained from Movement

The horizontal pressure, Δ_{ph} in ksf, on a wall resulting from a uniformly loaded strip parallel to the wall may be taken as:

$$\Delta_{ph} = \frac{2p}{\pi}\,[\delta - \sin\delta\cos(\delta + 2\alpha)] \tag{3.11.6.2-1}$$

where:

p = uniform load intensity on strip parallel to wall (ksf)
α = angle specified in Figure 3.11.6.2-1 (rad)
δ = angle specified in Figure 3.11.6.2-1 (rad)

C3.11.6.2

Eqs. 3.11.6.2-1, 3.11.6.2-2, 3.11.6.2-3, and 3.11.6.2-4 are based on the assumption that the wall does not move, i.e., walls which have a high degree of structural rigidity or restrained at the top combined with an inability to slide in response to applied loads. For flexible walls, this assumption can be very conservative. Additional guidance regarding the ability of walls to move is provided in Article C3.11.1.

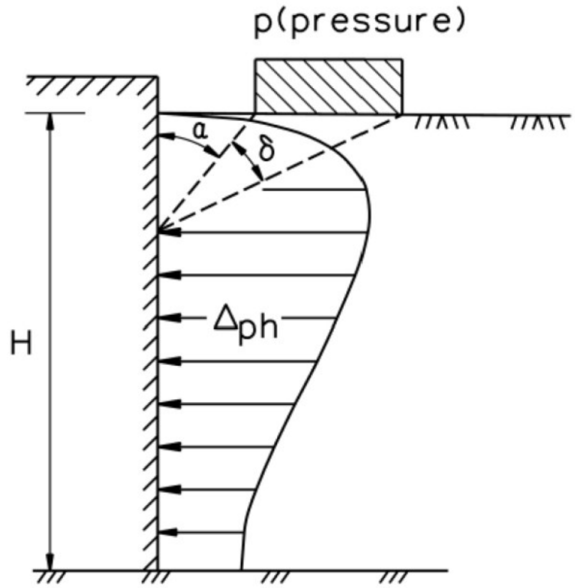

Figure 3.11.6.2-1—Horizontal Pressure on Wall Caused by a Uniformly Loaded Strip

The horizontal pressure, Δ_{ph} in ksf, on a wall resulting from a point load may be taken as:

$$\Delta_{ph} = \frac{P}{\pi R^2}\left[\frac{3ZX^2}{R^3} - \frac{R(1-2\nu)}{R+Z}\right] \tag{3.11.6.2-2}$$

where:

P = point load (kip)
R = radial distance from point of load application to a point on the wall as specified in Figure 3.11.6.2-2 where $R = (x^2 + y^2 + z^2)^{0.5}$ (ft)
X = horizontal distance from back of wall to point of load application (ft)
Y = horizontal distance from point on the wall under consideration to a plane, which is perpendicular to the wall and passes through the point of load application measured along the wall (ft)
Z = vertical distance from point of load application to the elevation of a point on the wall under consideration (ft)
ν = Poisson's ratio (dim.)

The point on the wall does not have to lie in a plane which is perpendicular to the wall and passes through the point of load application.

Poisson's ratio for soils varies from about 0.25 to 0.49, with lower values more typical for granular and stiff cohesive soils and higher values more typical for soft cohesive soils.

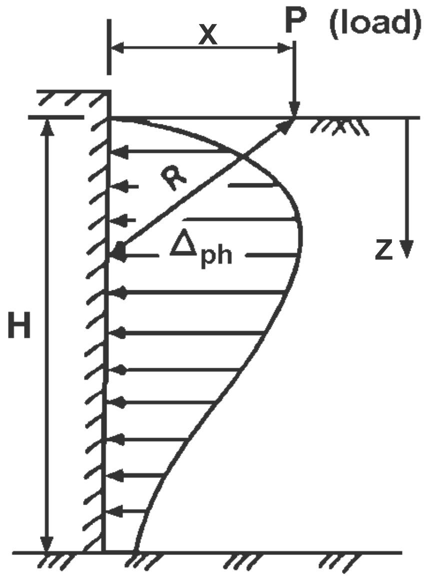

Figure 3.11.6.2-2—Horizontal Pressure on a Wall Caused by a Point Load

The horizontal pressure, Δ_{ph} in ksf, resulting from an infinitely long line load parallel to a wall may be taken as:

$$\Delta_{ph} = \frac{4Q}{\pi} \frac{X^2 Z}{R^4} \qquad (3.11.6.2\text{-}3)$$

where:

Q = load intensity in kip/ft

and all other notation is as defined above and shown in Figure 3.11.6.2-3.

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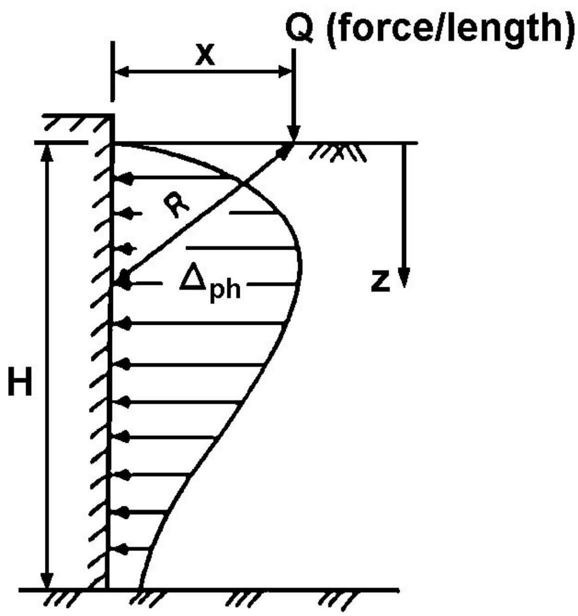


Figure 3.11.6.2-3—Horizontal Pressure on a Wall Caused by an Infinitely Long Line Load Parallel to the Wall

The horizontal pressure distribution, Δ_{ph} in ksf, on a wall resulting from a finite line load perpendicular to a wall may be taken as:

$$\Delta_{ph} = \frac{Q}{\pi Z}\left(\frac{1}{A^3} - \frac{1-2\nu}{A + \dfrac{Z}{X_2}} - \frac{1}{B^3} + \frac{1-2\nu}{B + \dfrac{Z}{X_1}}\right) \quad (3.11.6.2\text{-}4)$$

in which:

$$A = \sqrt{1 + \left(\frac{Z}{X_2}\right)^2} \quad (3.11.6.2\text{-}5)$$

$$B = \sqrt{1 + \left(\frac{Z}{X_1}\right)^2} \quad (3.11.6.2\text{-}6)$$

where:

X_1 = distance from the back of the wall to the start of the line load as specified in Figure 3.11.6.2-4 (ft)
X_2 = distance between the back of wall and the far end of the finite line load as specified in Figure 3.11.6.2-4 (ft)
Z = depth from the ground surface to a point on the wall under consideration (ft)
ν = Poisson's Ratio (dim.)
Q = load intensity (kip/ft)

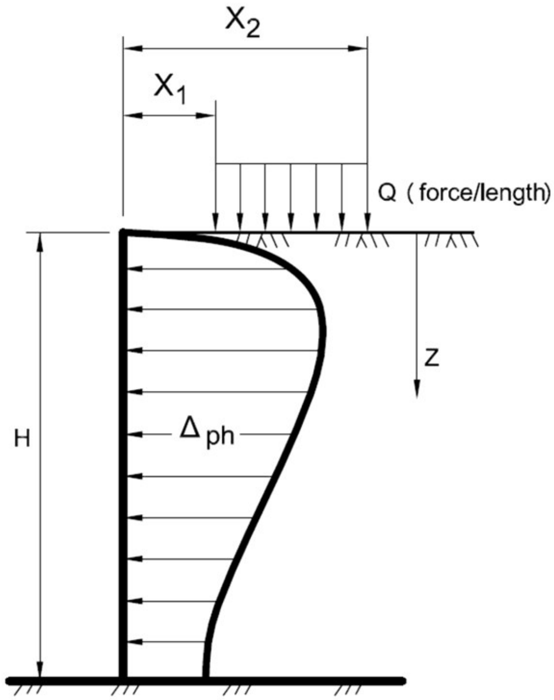


Figure 3.11.6.2-4—Horizontal Pressure on a Wall Caused by a Finite Line Load Perpendicular to the Wall

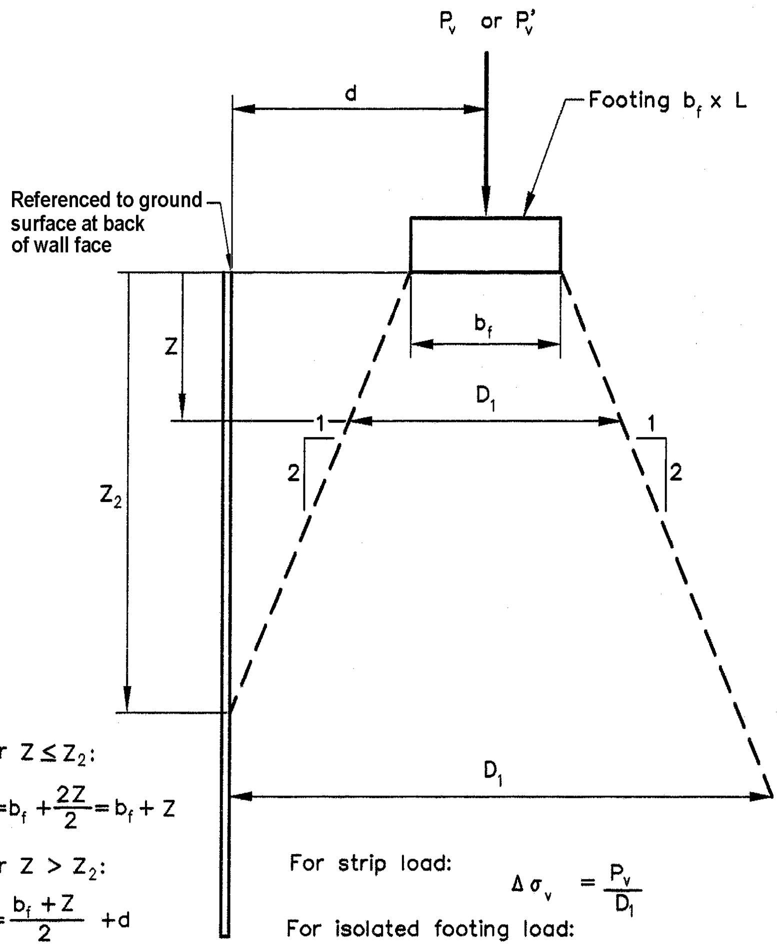
3.11.6.3—Strip Loads (*ES*): Flexible Walls

Concentrated dead loads shall be incorporated into the internal and external stability design by using a simplified uniform vertical distribution of 2 vertical to 1 horizontal to determine the vertical component of stress with depth within the reinforced soil mass as specified in Figure 3.11.6.3-1. Concentrated horizontal loads at the top of the wall shall be distributed within the reinforced soil mass as specified in Figure 3.11.6.3-2. If concentrated dead loads are located behind the reinforced soil mass, they shall be distributed in the same way as would be done within the reinforced soil mass.

The vertical stress distributed behind the reinforced zone shall be multiplied by k_a when determining the effect of this surcharge load on external stability. The concentrated horizontal stress distributed behind the wall as specified in Figure 3.11.6.3-2 shall not be multiplied by k_a.

C3.11.6.3

Figures 3.11.6.3-1 and 3.11.6.3-2 are based on the assumption that the wall is relatively free to move laterally (e.g., MSE walls).



For $Z \leq Z_2$:

$$D_1 = b_f + \frac{2Z}{2} = b_f + Z$$

For $Z > Z_2$:

$$D_1 = \frac{b_f + Z}{2} + d$$

For strip load:

$$\Delta\sigma_v = \frac{P_v}{D_1}$$

For isolated footing load:

$$\Delta\sigma_v = \frac{P'_v}{D_1(L+Z)}$$

For point load:

$$\Delta\sigma_v = \frac{P'_v}{D_1^2} \text{ with } b_f = 0$$

Where: D_1 = Effective width of applied load at any depth, calculated as shown above

b_f = Width of applied load. For footings which are eccentrically loaded (e.g., bridge abutment footings), set b_f equal to the equivalent footing width B' by reducing it by 2e', where e' is the eccentricity of the footing load (i.e., b_f −2e').

L = Length of footing

P_v = Load per linear foot of strip footing

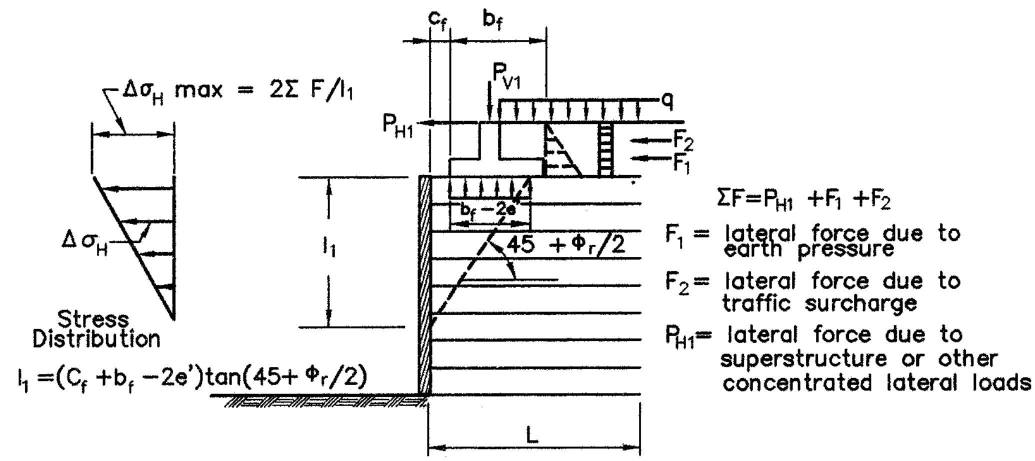
P'_v = Load on isolated rectangular footing or point load

Z_2 = depth where effective width intersects back of wall face = $2d - b_f$

d = distance between the centroid of the concentrated vertical load and the back of the wall face.

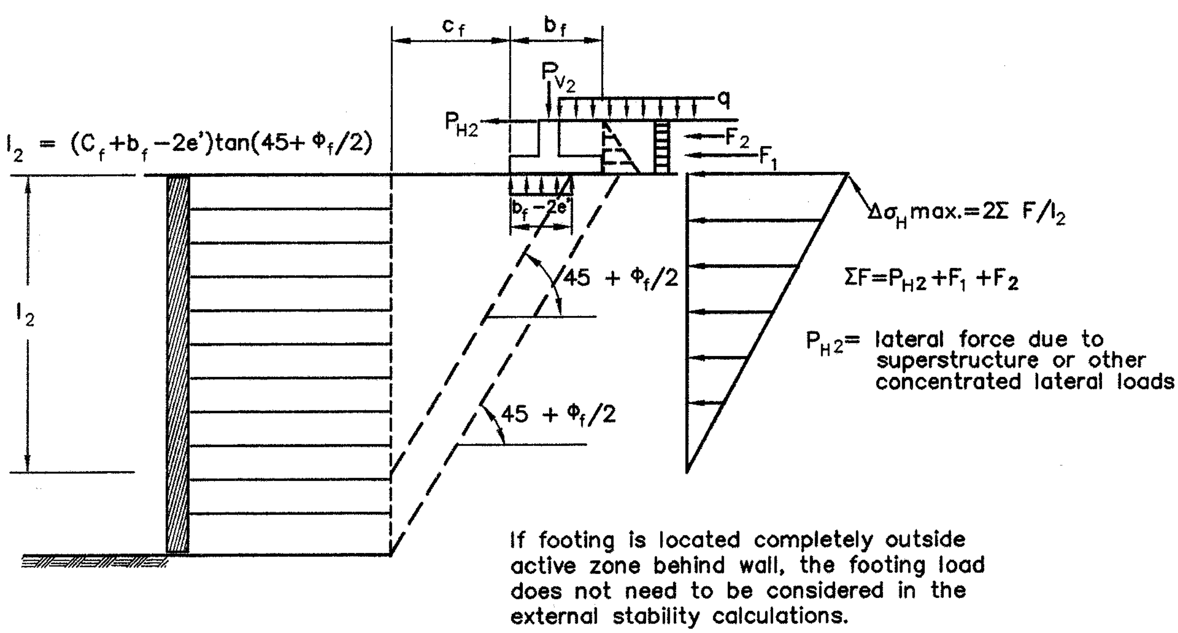
Assume the increased vertical stress due to the surcharge load has no influence on stresses used to evaluate internal stability if the surcharge load is located behind the reinforced soil mass. For external stability, assume the surcharge has no influence if it is located outside the active zone behind the wall.

Figure 3.11.6.3-1—Distribution of Stress from Concentrated Vertical Load *P_v* for Internal and External Stability Calculations



e' = eccentricity of load on footing (see Figure 11.10.10.1-1 for example of how to calculate this)

a—Distribution of Stress for Internal Stability Calculations



b—Distribution of Stress for External Stability Calculations

Figure 3.11.6.3-2—Distribution of Stress from Concentrated Horizontal Loads

3.11.6.4—Live Load Surcharge (*LS*)

A live load surcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the wall height behind the back face of the wall. If the surcharge is for a highway, the intensity of the load shall be consistent with the provisions of Article 3.6.1.2. If the surcharge is for other than a highway, the Owner shall specify and/or approve appropriate surcharge loads.

The increase in horizontal pressure due to live load surcharge may be estimated as:

$$\Delta_p = k\gamma_s h_{eq} \quad (3.11.6.4\text{-}1)$$

where:

Δ_p = constant horizontal earth pressure due to live load surcharge (ksf)
γ_s = total unit weight of soil (kcf)
k = coefficient of lateral earth pressure
h_{eq} = equivalent height of soil for vehicular load (ft)

Equivalent heights of soil, h_{eq}, for highway loadings on abutments and retaining walls may be taken from Tables 3.11.6.4-1 and 3.11.6.4-2. Linear interpolation shall be used for intermediate wall heights.

The wall height shall be taken as the distance between the surface of the backfill and the bottom of the footing along the pressure surface being considered.

Table 3.11.6.4-1—Equivalent Height of Soil for Vehicular Loading on Abutments Perpendicular to Traffic

Abutment Height (ft)	h_{eq} (ft)
5.0	4.0
10.0	3.0
≥20.0	2.0

Table 3.11.6.4-2—Equivalent Height of Soil for Vehicular Loading on Retaining Walls Parallel to Traffic

Retaining Wall Height (ft)	h_{eq} (ft) Distance from wall backface to edge of traffic	
	0.0 ft	1.0 ft or Further
5.0	5.0	2.0
10.0	3.5	2.0
≥20.0	2.0	2.0

The load factor for both vertical and horizontal components of live load surcharge shall be taken as specified in Table 3.4.1-1 for live load surcharge.

C3.11.6.4

The tabulated values for h_{eq} were determined by evaluating the horizontal force against an abutment or wall from the pressure distribution produced by the vehicular live load of Article 3.6.1.2. The pressure distributions were developed from elastic half-space solutions using the following assumptions:

- Vehicle loads are distributed through a two-layer system consisting of pavement and soil subgrade.
- Poisson's ratio for the pavement and subgrade materials are 0.2 and 0.4, respectively.
- Wheel loads were modeled as a finite number of point loads distributed across the tire area to produce an equivalent tire contact stress.
- The process for equating wall moments resulting from the elastic solution with the equivalent surcharge method used a wall height increment of 0.25 ft.

The value of the coefficient of lateral earth pressure k is taken as k_o, specified in Article 3.11.5.2, for walls that do not deflect or move, or k_a, specified in Articles 3.11.5.3, 3.11.5.6 and 3.11.5.7, for walls that deflect or move sufficiently to reach minimum active conditions.

The analyses used to develop Tables 3.11.6.4-1 and 3.11.6.4-2 are presented in Kim and Barker (1998).

The values for h_{eq} given in Tables 3.11.6.4-1 and 3.11.6.4-2 are generally greater than the traditional 2.0 ft of earth load historically used in the AASHTO specifications, but less than those prescribed in previous editions (i.e., before 1998) of this specification. The traditional value corresponds to a 20.0-kip single unit truck formerly known as an H10 truck, Peck et al. (1974). This partially explains the increase in h_{eq} in previous editions of this specification. Subsequent analyses, i.e., Kim and Barker (1998), show the importance of the direction of traffic, i.e., parallel for a wall and perpendicular for an abutment on the magnitude of h_{eq}. The magnitude of h_{eq} is greater for an abutment than for a wall due to the proximity and closer spacing of wheel loads to the back of an abutment compared to a wall.

The backface of the wall should be taken as the pressure surface being considered. Refer to Article C11.5.5 for application of surcharge pressures on retaining walls.

3.11.6.5—Reduction of Surcharge

If the vehicular loading is transmitted through a structural slab, which is also supported by means other than earth, a corresponding reduction in the surcharge loads may be permitted.

C3.11.6.5

This Article relates primarily to approach slabs which are supported at one edge by the backwall of an abutment, thus transmitting load directly thereto.

3.11.7—Reduction Due to Earth Pressure

For culverts and bridges and their components where earth pressure may reduce effects caused by other loads and forces, such reduction shall be limited to the extent earth pressure can be expected to be permanently present. In lieu of more precise information, a 50 percent reduction may be used, but need not be combined with the minimum load factor specified in Table 3.4.1-2.

C3.11.7

This provision is intended to refine the traditional approach in which the earth pressure is reduced by 50 percent in order to obtain maximum positive moment in top slab of culverts and frames. It permits obtaining more precise estimates of force effects where earth pressures are present.

3.11.8—Downdrag

Possible development of downdrag on piles or shafts shall be evaluated where:

- Sites are underlain by compressible material such as clays, silts or organic soils;
- Fill will be or has recently been placed adjacent to the piles or shafts, such as is frequently the case for bridge approach fills;
- The groundwater is substantially lowered; or
- Liquefaction of loose sandy soil can occur.

When the potential exists for downdrag to act on a pile or shaft due to downward movement of the soil relative to the pile or shaft, and the potential for downdrag is not eliminated by preloading the soil to reduce downward movements or other mitigating measure, the pile or shaft shall be designed to resist the induced downdrag.

Consideration shall be given to eliminating the potential for downdrag loads through the use of embankment surcharge loads, ground improvement techniques, and/or vertical drainage and settlement monitoring measurements.

For Extreme Event I limit state, downdrag induced by liquefaction settlement shall be applied to the pile or shaft in combination with the other loads included within that load group. Liquefaction-induced downdrag shall not be combined with downdrag induced by consolidation settlements.

For downdrag load applied to pile or shaft groups, group effects shall be evaluated.

C3.11.8

Downdrag, also known as negative skin friction, can be caused by soil settlement due to loads applied after the piles were driven, such as an approach embankment as shown in Figure C3.11.8-1. Consolidation can also occur due to recent lowering of the groundwater level as shown in Figure C3.11.8-2.

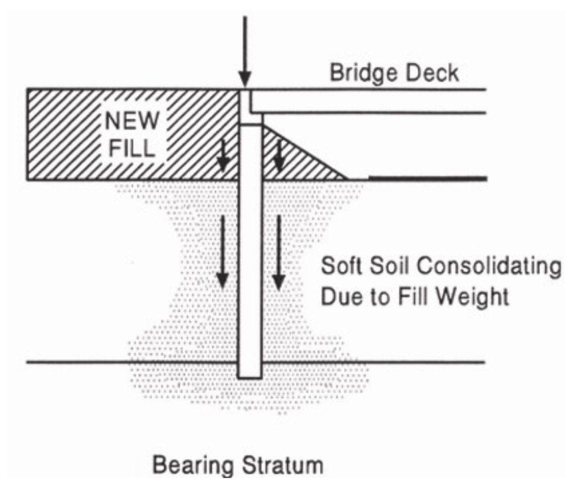


Figure C3.11.8-1—Common Downdrag Situation Due to Fill Weight (Hannigan, et al., 2005)

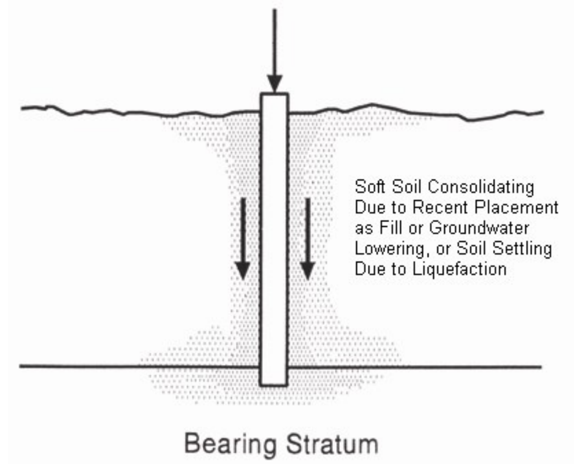


Figure C3.11.8-2—Common Downdrag Situation Due to Causes Other than Recent Fill Placement

Regarding the load factors for downdrag in Table 3.4.1-2, use the maximum load factor when investigating maximum downward pile loads. The minimum load factor shall only be utilized when investigating possible uplift loads.

For some downdrag estimation methods, the magnitude of the load factor is dependent on the magnitude of the downdrag load relative to the dead load. The downdrag load factors were developed considering that downdrag loads equal to or greater than the magnitude of the dead load become somewhat impractical for design. See Allen (2005) for additional background and guidance on the effect of downdrag load magnitude.

Methods for eliminating static downdrag potential include preloading. The procedure for designing a preload is presented in Cheney and Chassie (2000).

Post-liquefaction settlement can also cause downdrag. Methods for mitigating liquefaction-induced downdrag are presented in Kavazanjian, et al. (1997).

The application of downdrag to pile or shaft groups can be complex. If the pile or shaft cap is near or below the fill material causing consolidation settlement of the underlying soft soil, the cap will prevent transfer of stresses adequate to produce settlement of the soil inside the pile or shaft group. The downdrag applied in this case is the frictional force around the exterior of the pile or shaft group and along the sides of the pile or shaft cap (if any). If the cap is located well up in the fill causing consolidation stresses or if the piles or shafts are used as individual columns to support the structure above ground, the downdrag on each individual pile or shaft will control the magnitude of the load. If group effects are likely, the downdrag calculated using the group perimeter shear force should be determined in addition to the sum of the downdrag forces for each individual pile or shaft. The greater of the two calculations should be used for design.

The skin friction used to estimate downdrag due to liquefaction settlement should be conservatively assumed to be equal to the residual soil strength in the liquefiable zone,

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If transient loads act to reduce the magnitude of downdrag loads and this reduction is considered in the design of the pile or shaft, the reduction shall not exceed that portion of transient load equal to the downdrag force effect.

Force effects due to downdrag on piles or drilled shafts should be determined as follows:

Step 1—Establish soil profile and soil properties for computing settlement using the procedures in Article 10.4.

Step 2—Perform settlement computations for the soil layers along the length of the pile or shaft using the procedures in Article 10.6.2.4.3.

Step 3—Determine the length of pile or shaft that will be subject to downdrag. If the settlement in the soil layer is 0.4 in. or greater relative to the pile or shaft, downdrag can be assumed to fully develop.

Step 4—Determine the magnitude of the downdrag, *DD*, by computing the negative skin resistance using any of the static analysis procedures in Article 10.7.3.8.6 for piles in all soils and Article 10.8.3.4 for shafts if the zone subject to downdrag is characterized as a cohesive soil. If the downdrag zone is characterized as a cohesionless soil, the procedures provided in Article 10.8.3.4 should be used to estimate the downdrag for shafts. Sum the negative skin resistance for all layers contributing to downdrag from the lowest layer to the bottom of the pile cap or ground surface.

The neutral plane method may also be used to determine downdrag.

and nonliquefied skin friction in nonliquefiable layers above the zone of liquefaction.

Transient loads can act to reduce the downdrag because they cause a downward movement of the pile resulting in a temporary reduction or elimination of the downdrag load. It is conservative to include the transient loads together with downdrag.

The step-by-step procedure for determining downdrag is presented in detail in Hannigan, et al. (2005).

The stress increases in each soil layer due to embankment load can be estimated using the procedures in Hannigan et al. (2005) or Cheney and Chassie (2000).

If the settlement is due to liquefaction, the Tokimatsu and Seed (1987) or the Ishihara and Yoshimine (1992) procedures can be used to estimate settlement.

The methods used to estimate downdrag are the same as those used to estimate skin friction, as described in Articles 10.7 and 10.8. The distinction between the two is that downdrag acts downward on the sides of the piles or shafts and loads the foundation, whereas skin friction acts upward on the sides of piles or shafts and, thus, supports the foundation loads.

Downdrag can be estimated for piles using the α or λ methods for cohesive soils. An alternative approach would be to use the β method where the long-term conditions after consolidation should be considered. Cohesionless soil layers overlying the consolidating layers will also contribute to downdrag, and the negative skin resistance in these layers should be estimated using an effective stress method.

Downdrag loads for shafts may be estimated using the α method for cohesive soils and the β method for granular soils, as specified in Article 10.8, for calculating negative shaft resistance. As with positive shaft resistance, the top 5.0 ft and a bottom length taken as one shaft diameter do not contribute to downdrag loads. When using the α method, an allowance should be made for a possible increase in the undrained shear strength as consolidation occurs.

The neutral plane method is described and discussed in NCHRP 393 (Briaud and Tucker, 1993).

3.12—FORCE EFFECTS DUE TO SUPERIMPOSED DEFORMATIONS: *TU*, *TG*, *SH*, *CR*, *SE, PS*

3.12.1—General

Internal force effects in a component due to creep and shrinkage shall be considered. The effect of a temperature gradient should be included where appropriate. Force effects resulting from resisting component deformation, displacement of points of load application, and support movements shall be included in the analysis.

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3.12.2—Uniform Temperature

The design thermal movement associated with a uniform temperature change may be calculated using Procedure A or Procedure B below. Either Procedure A or Procedure B may be employed for concrete deck bridges having concrete or steel girders. Procedure A shall be employed for all other bridge types.

3.12.2.1—Temperature Range for Procedure A

The ranges of temperature shall be as specified in Table 3.12.2.1-1. The difference between the extended lower or upper boundary and the base construction temperature assumed in the design shall be used to calculate thermal deformation effects.

The minimum and maximum temperatures specified in Table 3.12.2.1-1 shall be taken as $T_{MinDesign}$ and $T_{MaxDesign}$, respectively, in Eq. 3.12.2.3-1.

C3.12.2.1

Procedure A is the historic method that has been used for bridge design.

For these Specifications, a moderate climate may be determined by the number of freezing days per yr. If the number of freezing days is less than 14, the climate is considered to be moderate. Freezing days are days when the average temperature is less than 32°F.

Although temperature changes in a bridge do not occur uniformly, bridges generally are designed for an assumed uniform temperature change. The orientation of bearing guides and the freedom of bearing movement is important. Sharp curvature and sharply skewed supports can cause excessive lateral thermal forces at supports if only tangential movement is permitted. Wide bridges are particularly prone to large lateral thermal forces because the bridge expands radially as well as longitudinally.

Table 3.12.2.1-1—Procedure A Temperature Ranges

Climate	Steel or Aluminum	Concrete	Wood
Moderate	0° to 120°F	10° to 80°F	10° to 75°F
Cold	−30° to 120°F	0° to 80°F	0° to 75°F

3.12.2.2—Temperature Range for Procedure B

The temperature range shall be defined as the difference between the maximum design temperature, $T_{MaxDesign}$, and the minimum design temperature, $T_{MinDesign}$. For all concrete girder bridges with concrete decks, $T_{MaxDesign}$ shall be determined from the contours of Figure 3.12.2.2-1 and $T_{MinDesign}$ shall be determined from the contours of Figure 3.12.2.2-2. For steel girder bridges with concrete decks, $T_{MaxDesign}$ shall be determined from the contours of Figure 3.12.2.2-3 and $T_{MinDesign}$ shall be determined from the contours of Figure 3.12.2.2-4.

C3.12.2.2

The Procedure B design was developed on the basis of the report *Thermal Movement Design Procedure for Steel and Concrete Bridges (*Roeder, 2002*)*.

Procedure B is a calibrated procedure and does not cover all bridge types. The temperatures provided in the maps of Figures 3.12.2.2-1 to 3.12.2.2-4 are extreme bridge design temperatures for an average history of 70 yr with a minimum of 60 yr of data for locations throughout the U.S.

The design values for locations between contours should be determined by linear interpolation. As an alternative method, the largest adjacent contour may be used to define $T_{MaxDesign}$ and the smallest adjacent contour may be used to define $T_{MinDesign}$. Both the minimum and maximum design temperatures should be noted on the drawings for the girders, expansion joints, and bearings.

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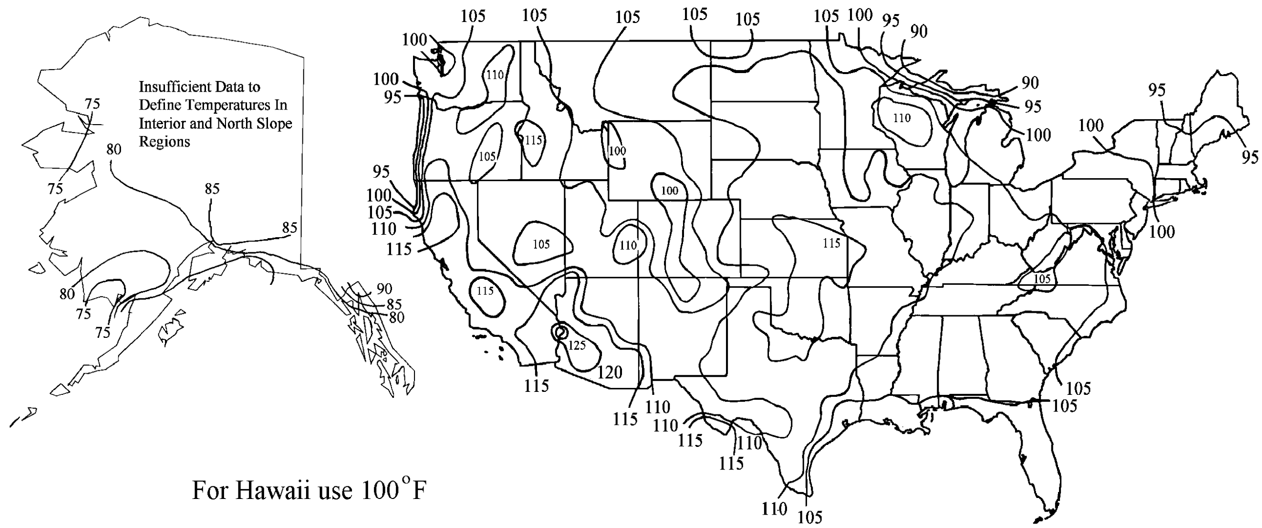

Figure 3.12.2.2-1—Contour Maps for *T_{MaxDesign}* for Concrete Girder Bridges with Concrete Decks

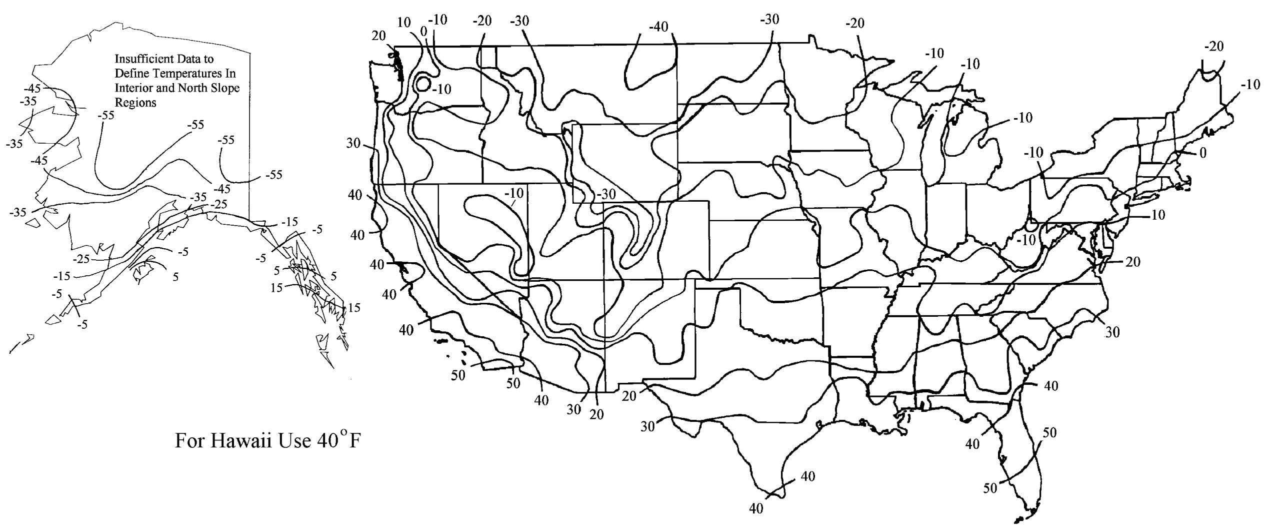

Figure 3.12.2.2-2—Contour Maps for *T_{MinDesign}* for Concrete Girder Bridges with Concrete Decks

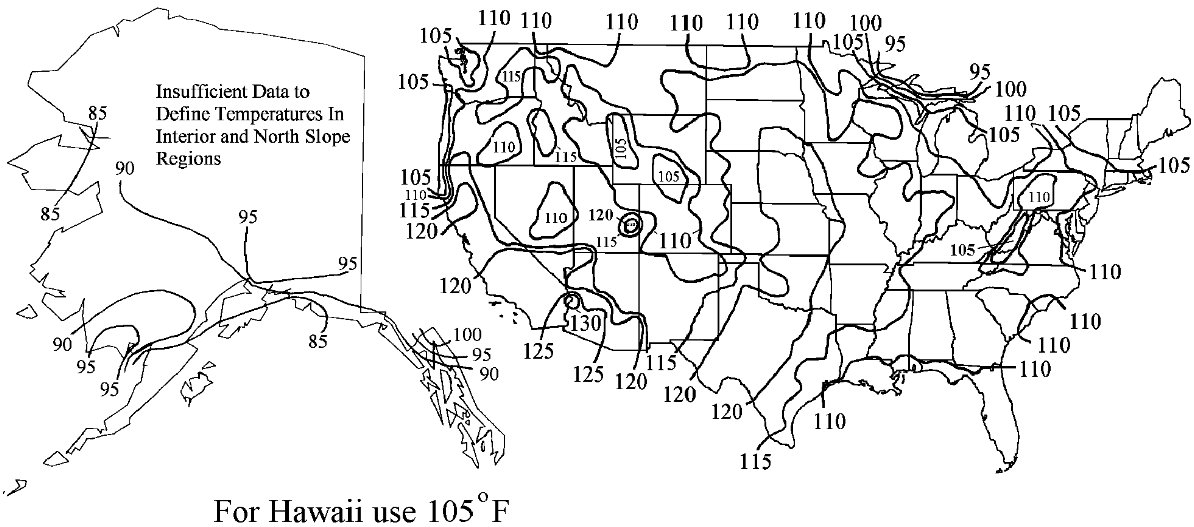

Figure 3.12.2.2-3—Contour Maps for $T_{MaxDesign}$ for Steel Girder Bridges with Concrete Decks

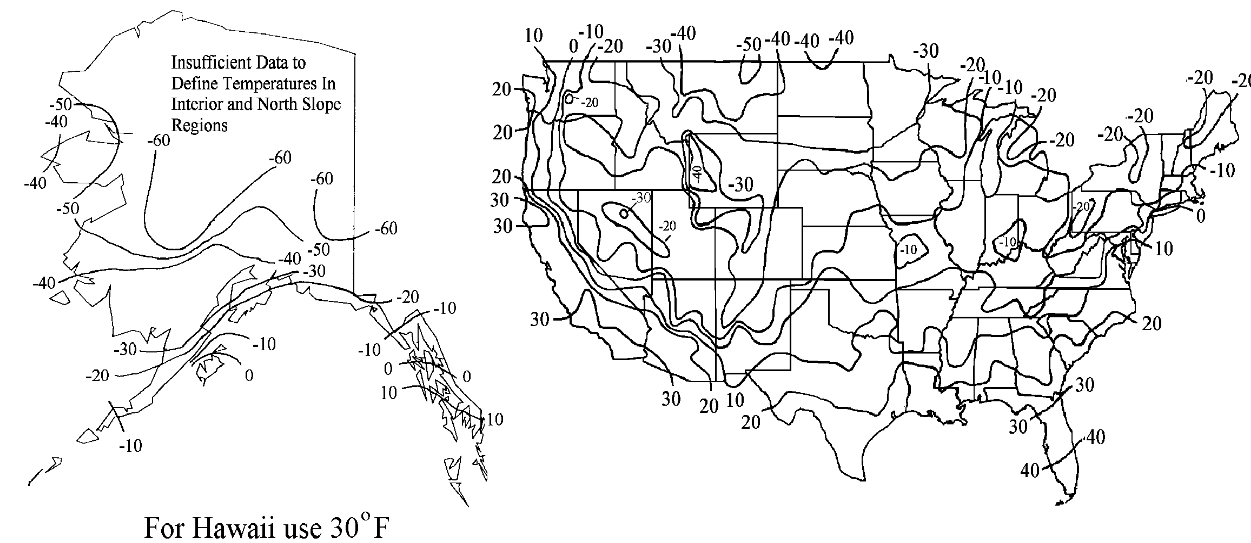

Figure 3.12.2.2-4—Contour Maps for $T_{MinDesign}$ for Steel Girder Bridges with Concrete Decks

3.12.2.3—Design Thermal Movements

The design thermal movement range, Δ_T, shall depend upon the extreme bridge design temperatures defined in Article 3.12.2.1 or 3.12.2.2, and be determined as:

$$\Delta_T = \alpha\, L\, (T_{MaxDesign} - T_{MinDesign}) \qquad (3.12.2.3\text{-}1)$$

where:

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

3.12.3—Temperature Gradient

For the purpose of this Article, the country shall be subdivided into zones as indicated in Figure 3.12.3-1. Positive temperature values for the zones shall be taken as specified for various deck surface conditions in Table 3.12.3-1. Negative temperature values shall be obtained by multiplying the values specified in Table 3.12.3-1 by −0.30 for plain concrete decks and −0.20 for decks with an asphalt overlay.

The vertical temperature gradient in concrete and steel superstructures with concrete decks may be taken as shown in Figure 3.12.3-2.

Dimension *A* in Figure 3.12.3-2 shall be taken as:

- For concrete superstructures that are 16.0 in. or more in depth—12.0 in.
- For concrete sections shallower than 16.0 in.—4.0 in. less than the actual depth
- For steel superstructures—12.0 in. and the distance *t* shall be taken as the depth of the concrete deck

Temperature value T_3 shall be taken as 0.0 degrees F, unless a site-specific study is made to determine an appropriate value, but it shall not exceed 5 degrees F.

Where temperature gradient is considered, internal stresses and structure deformations due to both positive and negative temperature gradients may be determined in accordance with the provisions of Article 4.6.6.

Table 3.12.3-1—Basis for Temperature Gradients

Zone	T_1 (°F)	T_2 (°F)
1	54	14
2	46	12
3	41	11
4	38	9

C3.12.3

Temperature gradient is included in various load combinations in Table 3.4.1-1. This does not mean that it need be investigated for all types of structures. If experience has shown that neglecting temperature gradient in the design of a given type of structure has not lead to structural distress, the Owner may choose to exclude temperature gradient. Multibeam bridges are an example of a type of structure for which judgment and past experience should be considered.

Redistribution of reactive loads, both longitudinally and transversely, should also be calculated and considered in the design of the bearings and substructures.

The temperature gradient given herein is a modification of that proposed in Imbsen et al. (1985), which was based on studies of concrete superstructures. The addition for steel superstructures is patterned after the temperature gradient for that type of bridge in the Australian bridge specifications (AUSTROADS, 1992).

The data in Table 3.12.3-1 does not make a distinction regarding the presence or lack of an asphaltic overlay on decks. Field measurements have yielded apparently different indications concerning the effect of asphalt as an insulator or as a contributor (Spring, 1997). Therefore, any possible insulating qualities have been ignored herein.

The temperatures given in Table 3.12.3-1 form the basis for calculating the change in temperature with depth in the cross-section, not absolute temperature.

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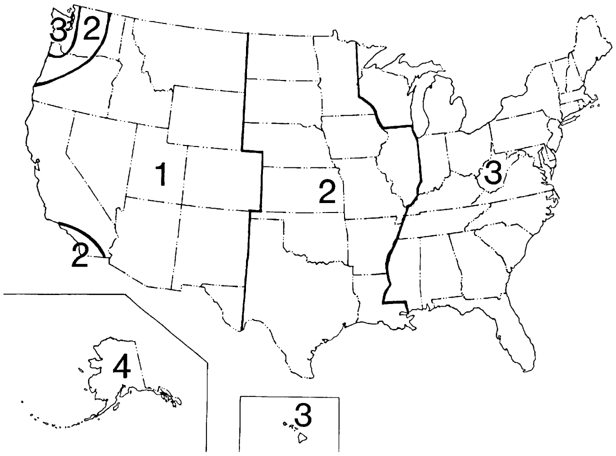

Figure 3.12.3-1—Solar Radiation Zones for the United States

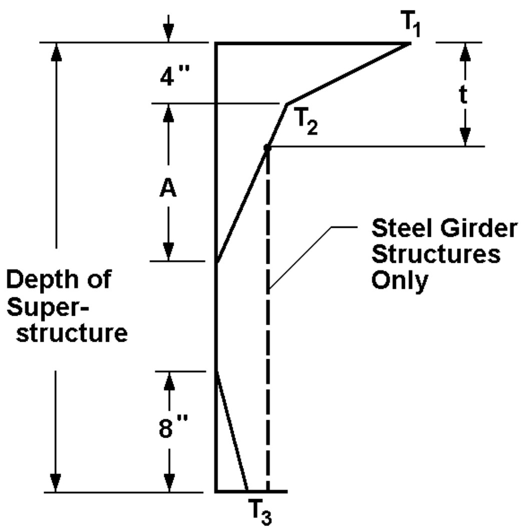

Figure 3.12.3-2—Positive Vertical Temperature Gradient in Concrete and Steel Superstructures

3.12.4—Differential Shrinkage

Where appropriate, differential shrinkage strains between concretes of different age and composition, and between concrete and steel or wood, shall be determined in accordance with the provisions of Section 5.

C3.12.4

The Designer may specify timing and sequence of construction in order to minimize stresses due to differential shrinkage between components. The load factor may be reduced to 1.0 if physical testing is performed to establish material properties and upper bound values are used in the analysis.

3.12.5—Creep

Creep strains for concrete and wood shall be in accordance with the provisions of Section 5 and Section 8, respectively. In determining force effects and deformations due to creep, dependence on time and changes in compressive stresses shall be taken into account.

C3.12.5

Traditionally, only creep of concrete is considered. Creep of wood is addressed only because it applies to prestressed wood decks. The load factor may be reduced to 1.0 if physical testing is performed to establish material properties and upper bound values are used in the analysis.

3.12.6—Settlement

Force effects due to extreme values of differential settlements among substructures and within individual substructure units shall be considered. Estimates of settlement for individual substructure units may be made in accordance with the provisions in Article 10.7.2.3.

C3.12.6

Force effects due to settlement may be reduced by considering creep. Analysis for the load combinations in Tables 3.4.1-1 and 3.4.1-2 which include settlement should be repeated for settlement of each possible substructure unit settling individually, as well as combinations of substructure units settling, that could create critical force effects in the structure.

3.12.7—Secondary Forces from Post-Tensioning, *PS*

The application of post-tensioning forces on a continuous structure produces reactions at the supports and internal forces that are collectively called secondary forces, which shall be considered where applicable.

C3.12.7

In frame analysis software, secondary forces are generally obtained by subtracting the primary prestress forces from the total prestresssing.

3.13—FRICTION FORCES: *FR*

Forces due to friction shall be established on the basis of extreme values of the friction coefficient between the sliding surfaces. Where appropriate, the effect of moisture and possible degradation or contamination of sliding or rotating surfaces upon the friction coefficient shall be considered.

C3.13

Low and high friction coefficients may be obtained from standard textbooks. If so warranted, the values may be determined by physical tests, especially if the surfaces are expected to be roughened in service.

3.14—VESSEL COLLISION: *CV*

3.14.1—General

The provisions of this Article apply to the accidental collision between a vessel and a bridge. These provisions may be revised as stated in Article 3.14.16 to account for intentional collisions.

All bridge components in a navigable waterway crossing, located in design water depths not less than 2.0 ft, shall be designed for vessel impact.

The minimum design impact load for substructure design shall be determined using an empty hopper barge drifting at a velocity equal to the yearly mean current for the waterway location. The design barge shall be a single 35.0-ft × 195-ft barge, with an empty displacement of 200 tons, unless approved otherwise by the Owner.

Where bridges span deep draft waterways and are not sufficiently high to preclude contact with the vessel, the minimum superstructure design impact may be taken to be the mast collision impact load specified in Article 3.14.10.3.

C3.14.1

Intentional collision between a vessel and a bridge may be considered when conducting security studies.

The determination of the navigability of a waterway is usually made by the U.S. Coast Guard.

The requirements herein have been adapted from the AASHTO *Guide Specifications and Commentary for Vessel Collision Design of Highway Bridges* (*1991*) using the Method II risk acceptance alternative, and modified for the second edition (2009). The 1991 Guide Specifications required the use of a single vessel length overall (*LOA*) selected in accordance with the Method I criteria for use in estimating the geometric probability and impact speed to represent all vessel classifications. This was a conservative simplification applied to reduce the amount of effort required in the analysis. With the introduction of personal computers and programming, the simplification can be lifted and *AF* can be quickly obtained for each design vessel, which was originally envisioned. The end result is a more accurate model for the vessel collision study as well as more informative conclusions about the vessel fleet and associated probabilities of collision.

Another source of information has been the proceedings of an international colloquium, Ship Collisions with Bridges and Offshore Structures (IABSE, 1983).

Barges are categorized by ton = 2,000 lbs. and ships by tonne = 2,205 lbs.

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In navigable waterways where vessel collision is anticipated, structures shall be:

- designed to resist vessel collision forces, and/or
- adequately protected by fenders, dolphins, berms, islands, or other sacrifice-able devices.

In determining vessel collision loads, consideration shall be given to the relationship of the bridge to:

- waterway geometry,
- size, type, loading condition, and frequency of vessels using the waterway,
- available water depth,
- vessel speed and direction, and
- the structural response of the bridge to collision.

The deadweight tonnage (DWT) of a ship is the weight of the cargo, fuel, water, and stores. The DWT is only a portion of the total vessel weight, but it gives a general estimation of the ship size.

A minimum impact requirement from an empty barge drifting in all waterways and the mast impact of a drifting ship in deep draft waterways is specified because of the high frequency of occurrences of such collision accidents in United States waterways.

The intent of the vessel collision provisions is to minimize the risk of catastrophic failure of bridges crossing navigable waterways due to collisions by aberrant vessels. The collision impact forces represent a probabilistically based, worst-case, head-on collision, with the vessel moving in a forward direction at a relatively high velocity. The requirements are applicable to steel-hulled merchant ships larger than 1,000 DWT and to inland waterway barges.

The channel layout and geometry can affect the navigation conditions, the largest vessel size that can use the waterway and the loading condition and the speed of vessels approaching a bridge. The presence of bends, intersections with other waterways, and the presence of other bridge crossings near the bridge increase the probability of accidents. The vessel transit paths in the waterway in relation to the navigation channel and the bridge piers can affect the risk of aberrant vessels hitting the piers and the exposed portions of the superstructure.

The water level and the loading conditions of vessels influence the location on the pier where vessel impact loads are applied, and the susceptibility of the superstructure to vessel hits. The water depth plays a critical role in the accessibility of vessels to piers and spans outside the navigation channel. The water depth at the pier should not include short-term scour. In addition, the water depth should not just be evaluated at the specific pier location itself, but also at locations upstream and downstream of the pier—which may be shallower and would potentially block certain deeper draft vessels from hitting the pier. In waterways with large water stage fluctuations, the water level used can have a significant effect on the structural requirements for the pier and/or pier protection design.

The maneuverability of ships is reduced by the low underkeel clearance typical in inland waterways. Shallow underkeel clearance can also affect the hydrodynamic forces during a collision increasing the collision energy, especially in the transverse direction. In addition, ships riding in ballast can be greatly affected by winds and currents. When under ballast, vessels are susceptible to wind gusts that could push them into the bridge.

It is very difficult to control and steer barge tows, especially near bends and in waterways with high stream velocities and cross currents. In maneuvering a bend, tows experience a sliding effect in a direction opposite to the direction of the turn, due to inertia forces which are often coupled with the current flow. Bridges located in a high velocity waterway and near a bend in the channel will probably be hit by barges at frequent intervals.

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Unless otherwise indicated in these Specifications, an evaluation of the following two vessel collision events combined with scour conditions are recommended:

- A drifting empty barge breaking loose from its moorings and striking the bridge. The vessel impact loads should be combined with one-half of the predicted long-term scour plus one-half of the predicted short term scour. The flow rate, water level, and short-term scour depth are those associated with the design flood for bridge scour (100-year flood event).

- A ship or barge tow striking the bridge while transiting the navigation channel under typical waterway conditions. The vessel impact loads should be combined with the effects of one-half of the long-term scour and no short-term scour. The flow rate and water level should be taken as the yearly mean conditions.

3.14.2—Owner's Responsibility

The Owner shall establish and/or approve the bridge operational classification, the vessel traffic density in the waterway, and the design velocity of vessels for the bridge. The Owner shall specify or approve the degree of damage that the bridge components, including protective systems, are allowed to sustain.

C3.14.2

Pier protection systems may also be warranted for bridges over navigable channels transversed only by pleasure boats or small commercial vessels. For such locations, dolphins and fender systems are commonly used to protect the pier and to minimize the hazards of passage under the bridge for the vessels using the waterway.

3.14.3—Operational Classification

For the purpose of Article 3.14, an operational classification, either "critical or essential" or "typical," shall be determined for all bridges located in navigable waterways. Critical bridges shall continue to function after an impact, the probability of which is smaller than regular bridges.

C3.14.3

This Article implies that a critical or essential bridge may be damaged to an extent acceptable to the Owner, as specified in Article 3.14.2, but should not collapse and should remain serviceable, even though repairs are needed.

3.14.4—Design Vessel

A design vessel for each pier or span component shall be selected, such that the estimated annual frequency of collapse computed in accordance with Article 3.14.5, due to vessels not smaller than the design vessel, is less than the acceptance criterion for the component.

The design vessels shall be selected on the basis of the bridge operational classification and the vessel, bridge, and waterway characteristics.

C3.14.4

An analysis of the annual frequency of collapse is performed for each pier or span component exposed to collision. From this analysis, a design vessel and its associated collision loads can be determined for each pier or span component. The design vessel size and impact loads can vary greatly among the components of the same structure, depending upon the waterway geometry, available water depth, bridge geometry, and vessel traffic characteristics.

The design vessel is selected using a probability-based analysis procedure in which the predicted annual frequency of bridge collapse, AF, is compared to an acceptance criterion. The analysis procedure is an iterative process in which a trial design vessel is selected for a bridge component and a resulting AF is computed using the characteristics of waterway, bridge, and vessel fleet. This AF is compared to the acceptance criterion, and revisions to the analysis variables

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are made as necessary to achieve compliance. The primary variables that the Designer can usually alter include the:

- location of the bridge in the waterway;
- location and clearances of bridge pier and span components;
- resistance of piers and superstructures; and
- use of protective systems to either reduce or eliminate the collision forces.

3.14.5—Annual Frequency of Collapse

The annual frequency of a bridge component collapse shall be taken as:

$$AF = (N)(PA)(PG)(PC)(PF) \quad (3.14.5\text{-}1)$$

where:

AF = annual frequency of bridge component collapse due to vessel collision

N = the annual number of vessels, classified by type, size, and loading condition, that utilize the channel

PA = the probability of vessel aberrancy

PG = the geometric probability of a collision between an aberrant vessel and a bridge pier or span

PC = the probability of bridge collapse due to a collision with an aberrant vessel

PF = adjustment factor to account for potential protection of the piers from vessel collision due to upstream or downstream land masses or other structures that block the vessel

AF shall be computed for each bridge component and vessel classification. The annual frequency of collapse for the total bridge shall be taken as the sum of all component *AF*s.

For critical or essential bridges, the maximum annual frequency of collapse, *AF*, for the whole bridge, shall be taken as 0.0001.

For typical bridges, the maximum annual frequency of collapse, *AF*, for the total bridge, shall be taken as 0.001.

For waterways with widths less than 6.0 times the length overall of the design vessel, *LOA*, the acceptance criterion for the annual frequency of collapse for each pier and superstructure component shall be determined by distributing the total bridge acceptance criterion, *AF*, over the number of pier and span components located in the waterway.

For wide waterways with widths greater than 6.0 times *LOA*, the acceptance criterion for the annual frequency of collapse for each pier and span component shall be determined by distributing the total bridge acceptance criterion over the number of pier and superstructure

C3.14.5

Various types of risk assessment models have been developed for vessel collision with bridges by researchers worldwide (IABSE, 1983; Modjeski and Masters, 1984; Prucz, 1987; Larsen, 1993). Practically all of these models are based on a form similar to Eq. 3.14.5-1, which is used to compute the annual frequency of bridge collapse, *AF*, associated with a particular bridge component.

The inverse of the annual frequency of collapse, 1/*AF*, is equal to the return period in yr. The summation of *AF*s computed over all of the vessel classification intervals for a specific component equals the annual frequency of collapse of the component.

Risk can be defined as the potential realization of unwanted consequences of an event. Both a probability of occurrence of an event and the magnitude of its consequences are involved. Defining an acceptable level of risk is a value-oriented process and is by nature subjective (Rowe, 1977).

Based on historical collision data, the primary area of concern for vessel impact is the central portion of the bridge near the navigation channel. The limits of this area extend to a distance of 3.0 times *LOA* on each side of the inbound and outbound vessel transit path centerlines. For most bridges, these vessel transit path centerlines coincide with the centerline of the navigable channel. Where two-way vessel traffic exists under the bridge, the vessel transit path centerline of the inbound and outbound vessels should be taken as the centerline of each half of the channel, respectively.

components located within the distance 3.0 times *LOA* on each side of the inbound and outbound vessel transit centerline paths.

The distribution of the *AF* acceptance criterion among the exposed pier and span components is based on the Designer's judgment. One method is to equally spread the acceptable risk among all the components. This method is usually not desirable because it fails to take into account the importance and higher cost of most main span components. The preferred method is to apportion the risk to each pier and span component on the basis of its percentage value to the replacement cost of the structure in the central analysis area.

3.14.5.1—Vessel Frequency Distribution

The number of vessels, *N*, based on size, type, and loading condition and available water depth shall be developed for each pier and span component to be evaluated. Depending on waterway conditions, a differentiation between the number and loading condition of vessels transiting inbound and outbound shall be considered.

C3.14.5.1

In developing the design vessel distribution, the Designer should first establish the number and characteristics of the vessels using the navigable waterway or channel under the bridge. Because the water depth limits the size of vessel that could strike a bridge component, the navigable channel vessel frequency data can be modified, as required, on the basis of the water depth at each bridge component to determine the number and characteristics of the vessels that could strike the pier or span component being analyzed. Thus, each component could have a different value of *N*.

Vessel characteristics necessary to conduct the analysis include:

- Type, i.e., ship or barge;
- Size based on the vessel's deadweight tonnage, *DWT*;
- Inbound and outbound operating characteristics;
- Loading condition, i.e., loaded, partly loaded, ballasted, or empty;
- Length overall, *LOA*;
- Width or beam, B_M;
- Draft associated with each loading condition;
- Bow depth, D_B;
- Bow shape;
- Displacement tonnage, *W*;
- Vertical clearances; and
- Number of transits under the bridge each year.

Sources for the vessel data and typical ship and barge characteristics are included in the AASHTO Guide Specifications and Commentary for Vessel Collision Design of Highway Bridges (2009).

The Designer should use judgment in developing a distribution of the vessel frequency data based on discrete groupings or categories of vessel size by *DWT*. It is recommended that the *DWT* intervals used in developing the vessel distribution not exceed 20,000 *DWT* for vessels smaller than 100,000 *DWT*, and not exceeding 50,000 *DWT* for ships larger than 100,000 *DWT*.

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3.14.5.2—Probability of Aberrancy

3.14.5.2.1—General

The probability of vessel aberrancy, PA, may be determined by the statistical or the approximate method.

C3.14.5.2.1

The probability of aberrancy is mainly related to the navigation conditions at the bridge site. Vessel traffic regulations, vessel traffic management systems and aids to navigation can improve the navigation conditions and reduce the probability of aberrancy.

The probability of aberrancy, PA, sometimes referred to as the causation probability, is a measure of the risk that a vessel is in trouble as a result of pilot error, adverse environmental conditions, or mechanical failure.

An evaluation of accident statistics indicates that human error and adverse environmental conditions, not mechanical failures, are the primary reasons for accidents. In the United States, an estimated 60 percent to 85 percent of all vessel accidents have been attributed to human error.

3.14.5.2.2—Statistical Method

The probability of aberrancy may be computed on the basis of a statistical analysis of historical data on vessel collisions, rammings, and groundings in the waterway and on the number of vessels transiting the waterway during the period of accident reporting.

C3.14.5.2.2

The most accurate procedure for determining PA is to compute it using long-term vessel accident statistics in the waterway and data on the frequency of ship/barge traffic in the waterway during the same period of time (Larsen 1983). Data from ship simulation studies and radar analysis of vessel movements in the waterway have also been used to estimate PA. Based on historical data, it has been determined that the aberrancy rate for barges is usually two to three times that measured for ships in the same waterway.

3.14.5.2.3—Approximate Method

The probability of aberrancy may be taken as:

$$PA = (BR)(R_B)(R_C)(R_{XC})(R_D) \qquad (3.14.5.2.3\text{-}1)$$

where:

PA = probability of aberrancy
BR = aberrancy base rate
R_B = correction factor for bridge location
R_C = correction factor for current acting parallel to vessel transit path
R_{XC} = correction factor for cross-currents acting perpendicular to vessel transit path
R_D = correction factor for vessel traffic density

The base rate, BR, of aberrancy shall be taken as:

- For ships:

$$BR = 0.6\times10^{-4}$$

C3.14.5.2.3

Because the determination of PA based on actual accident data in the waterway is often a difficult and time-consuming process, an alternative method for estimating PA was established during the development of the AASHTO *Guide Specification and Commentary on Vessel Collision Design of Highway Bridges*. The equations in this Article are empirical relationships based on historical accident data. The predicted PA value using these equations and the values determined from accident statistics are generally in agreement, although exceptions do occur.

It should be noted that the procedure for computing PA using Eq. 3.14.5.2.3-1 should not be considered to be either rigorous or exhaustive. Several influences, such as wind, visibility conditions, navigation aids, pilotage, etc., were not directly included in the method because their effects were difficult to quantify. These influences have been indirectly included because the empirical equations were developed from accident data in which these factors had a part.

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- For barges:

$$BR = 1.2 \times 10^{-4}$$

The correction factor for bridge location, R_B, based on the relative location of the bridge in either of three waterway regions, as shown in Figure 3.14.5.2.3-1, shall be taken as:

- For straight regions:

$$R_B = 1.0 \qquad (3.14.5.2.3\text{-}2)$$

- For transition regions:

$$R_B = \left(1 + \frac{\theta}{90°}\right) \qquad (3.14.5.2.3\text{-}3)$$

- For turn/bend regions:

$$R_B = \left(1 + \frac{\theta}{45°}\right) \qquad (3.14.5.2.3\text{-}4)$$

where:

θ = angle of the turn or bend specified in Figure 3.14.5.2.3-1 (degrees)

The correction factor, R_C, for currents acting parallel to the vessel transit path in the waterway shall be taken as:

$$R_C = \left(1 + \frac{V_C}{10}\right) \qquad (3.14.5.2.3\text{-}5)$$

where:

V_C = current velocity component parallel to the vessel transit path (knots)

The correction factor, R_{XC}, for cross-currents acting perpendicular to the vessel transit path in the waterway shall be taken as:

$$R_{XC} = (1 + V_{XC}) \qquad (3.14.5.2.3\text{-}6)$$

where:

V_{XC} = current velocity component perpendicular to the vessel transit path (knots)

It is anticipated that future research will provide a better understanding of the probability of aberrancy and how to accurately estimate its value. The implementation of advanced vessel traffic control systems using automated surveillance and warning technology should significantly reduce the probability of aberrancy in navigable waterways.

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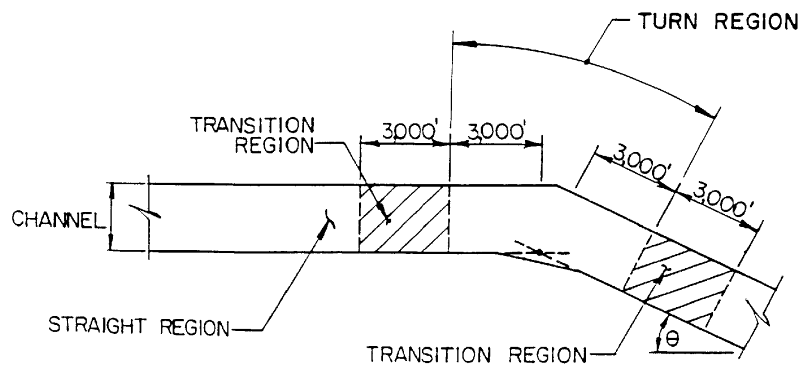
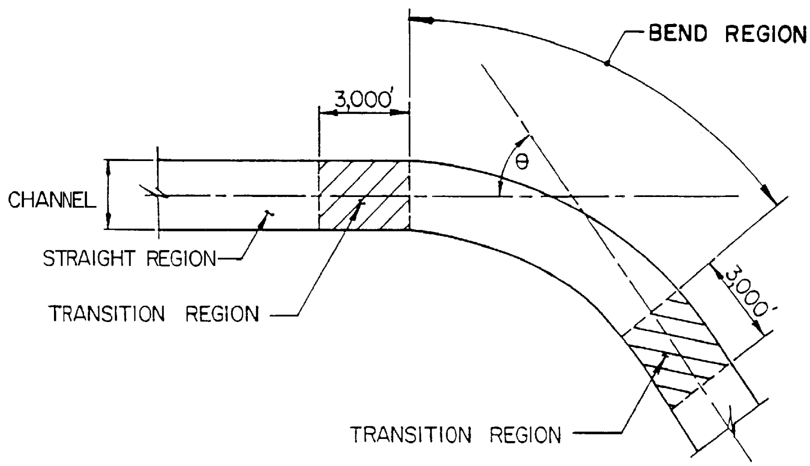


a. Turn in Channel



b. Bend in Channel

Figure 3.14.5.2.3-1—Waterway Regions for Bridge Location

The correction factor for vessel traffic density, R_D, shall be selected on the basis of the ship/barge traffic density level in the waterway in the immediate vicinity of the bridge defined as:

- Low density—vessels rarely meet, pass, or overtake each other in the immediate vicinity of the bridge:

$$R_D = 1.0 \quad (3.14.5.2.3\text{-}7)$$

- Average density—vessels occasionally meet, pass, or overtake each other in the immediate vicinity of the bridge:

$$R_D = 1.3 \quad (3.14.5.2.3\text{-}8)$$

- High density—vessels routinely meet, pass, or overtake each other in the immediate vicinity of the bridge:

$$R_D = 1.6 \quad (3.14.5.2.3\text{-}9)$$

3.14.5.3—Geometric Probability

A normal distribution may be utilized to model the sailing path of an aberrant vessel near the bridge. The geometric probability, *PG*, shall be taken as the area under the normal distribution bounded by the pier width and the width of the vessel on each side of the pier, as specified in Figure 3.14.5.3-1. The standard deviation, σ, of the normal distribution shall be assumed to be equal to the length overall, *LOA*, of the design vessel selected in accordance with Article 3.14.4.

The location of the mean of the standard distribution shall be taken at the centerline of the vessel transit path. *PG* shall be determined based on the width, B_M, of each vessel classification category, or it may be determined for all classification intervals using the B_M of the design vessel selected in accordance with Article 3.14.4.

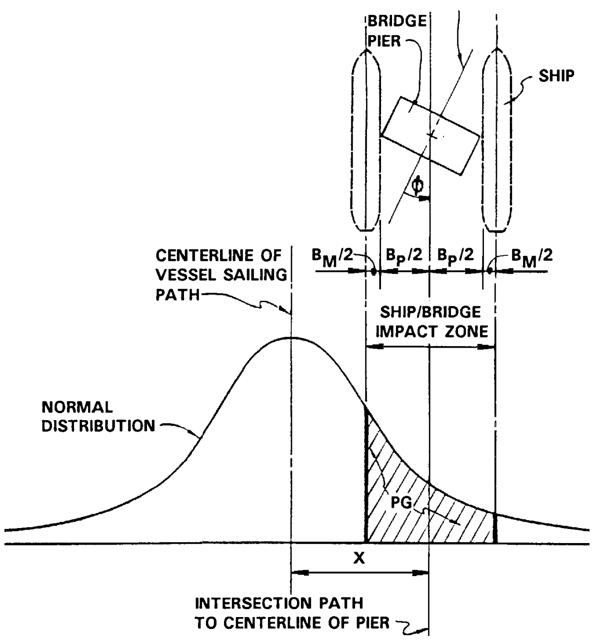


Figure 3.14.5.3-1—Geometric Probability of Pier Collision

C3.14.5.3

The geometric probability, *PG*, is defined as the conditional probability that a vessel will hit a bridge pier or superstructure component, given that it has lost control, i.e., it is aberrant, in the vicinity of the bridge. The probability of occurrence depends on the following factors:

- geometry of waterway;
- water depths of waterway;
- location of bridge piers;
- span clearances;
- sailing path of vessel;
- maneuvering characteristics of vessel;
- location, heading, and velocity of vessel;
- rudder angle at time of failure;
- environmental conditions;
- width, length, and shape of vessel; and
- vessel draft.

The horizontal clearance of the navigation span has a significant impact on the risk of vessel collision with the main piers. Analysis of past collision accidents has shown that fixed bridges with a main span less than two to three times the design vessel length or less than two times the channel width are particularly vulnerable to vessel collision.

Various geometric probability models, some based on simulation studies, have been recommended and used on different bridge projects and for the development of general design provisions. Descriptions of these models may be found in IABSE (1983), Modjeski and Masters (1984), Prucz (1987), and Larsen (1993). The method used to determine *PG* herein is similar to that proposed by Knott et al. (1985). The use of a normal distribution is based on historical ship/bridge accident data. It is recommended that σ = *LOA* of the design vessel for computing *PG*, and that bridge components located beyond 3σ from the centerline of the vessel transit path not be included in the analysis, other than the minimum impact requirement of Article 3.14.1.

The accident data used to develop the *PG* methodology primarily represents ships. Although barge accidents occur relatively frequently in United States waterways, there have been little published research findings concerning the distribution of barge accidents over a waterway. Until such data and research become available, it is recommended that the same σ = *LOA* developed for ships be applied to barges with the barge *LOA* equal to the total length of the barge tow, including the towboat.

3.14.5.4—Probability of Collapse

The probability of bridge collapse, PC, based on the ratio of the ultimate lateral resistance of the pier, H_P, and span, H_s, to the vessel impact force, P, shall be taken as:

- If $0.0 \leq H/P < 0.1$, then

$$PC = 0.1 + 9\left(0.1 - \frac{H}{P}\right) \quad (3.14.5.4\text{-}1)$$

- If $0.1 \leq H/P < 1.0$, then

$$PC = 0.111\left(1 - \frac{H}{P}\right) \quad (3.14.5.4\text{-}2)$$

- If $H/P \geq 1.0$, then

$$PC = 0.0 \quad (3.14.5.4\text{-}3)$$

where:

PC = probability of collapse
H = resistance of bridge component to a horizontal force expressed as pier resistance, H_P, or superstructure resistance, H_s (kip)
P = vessel impact force, P_S, P_{BH}, P_{DH}, or P_{MT}, specified in Articles 3.14.8, 3.14.10.1, 3.14.10.2, and 3.14.10.3, respectively (kip)

C3.14.5.4

The probability that the bridge will collapse once it has been struck by an aberrant vessel, PC, is complex and is a function of the vessel size, type, configuration, speed, direction, and mass. It is also dependent on the nature of the collision and stiffness/strength characteristic of the bridge pier and superstructure to resist the collision impact loads.

The methodology for estimating PC was developed by Cowiconsult (1987) from studies performed by Fujii and Shiobara (1978) using Japanese historical damage data on vessels colliding at sea. The damage to bridge piers is based on ship damage data because accurate damage data for collision with bridges is relatively scarce.

Figure C3.14.5.4-1 is a plot of the probability of collapse relationships. From this figure, the following results are evident:

- Where the pier or superstructure impact resistance exceeds the vessel collision impact force of the design vessel, the bridge collapse probability becomes 0.0.
- Where the pier or superstructure impact resistance is in the range 10–100 percent of the collision force of the design vessel, the bridge collapse probability varies linearly between 0.0 and 0.10.
- Where the pier or superstructure impact resistance is below ten percent of the collision force, the bridge collapse probability varies linearly between 0.10 and 1.0.

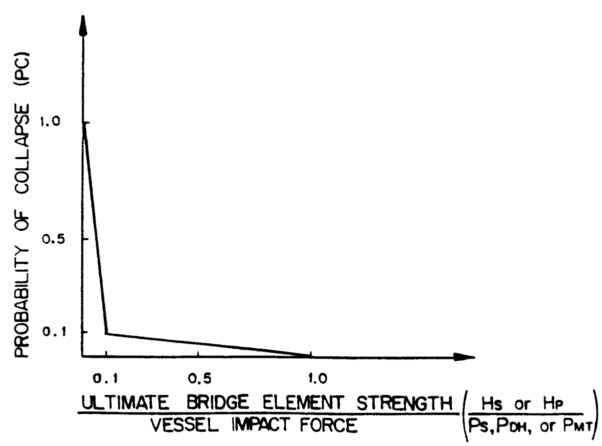


Figure C3.14.5.4-1—Probability of Collapse Distribution

3.14.5.5 Protection Factor

The protection factor, PF, shall be computed as:

$$PF = 1 - (\text{percent Protection Provided}/100) \quad (3.14.5.5\text{-}1)$$

If no protection of the pier exists, then $PF = 1.0$. If the pier is 100 percent protected, then $PF = 0.0$. If the pier protection (for example, a dolphin system) provides

C3.14.5.5

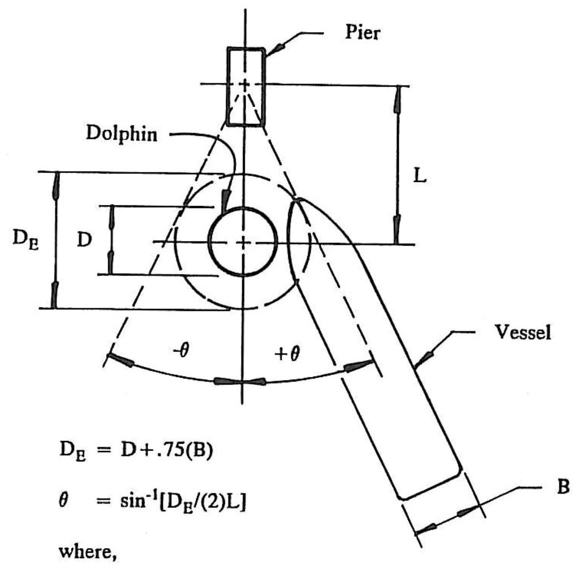
The purpose of the protection factor, PF, is to adjust the annual frequency of collapse, AF, for full or partial protection of selected bridge piers from vessel collisions such as:

- dolphins, islands, etc.,

70 percent protection, then *PF* would be equal to 0.3. Values for *PF* may vary from pier to pier and may vary depending on the direction of the vessel traffic (i.e., vessel traffic moving inbound versus traffic moving outbound).

- existing site conditions such as a parallel bridge protecting a bridge from impacts in one direction,
- a feature of the waterway (such as a peninsula extending out on one side of the bridge) that may block vessels from hitting bridge piers, or
- a wharf structure near the bridge that may block vessels from a certain direction.

The recommended procedure for estimating values for *PF* is shown in Figure C3.14.5.5-1. It illustrates a simple model developed to estimate the effectiveness of dolphin protection on a bridge pier.



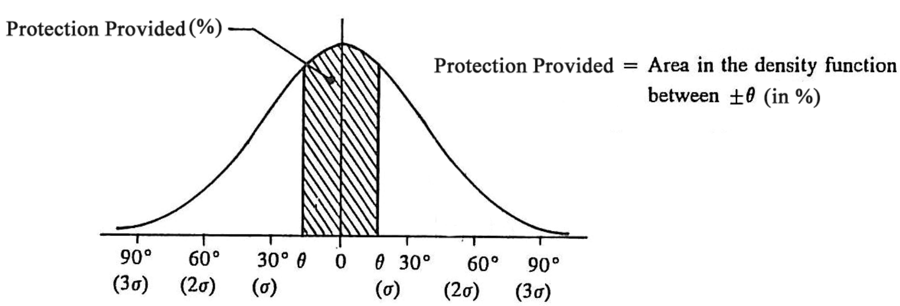
D_E = D+.75(B)

θ = $\sin^{-1}[D_E/(2)L]$

where,

θ = Protection angle provided by dolphin
D = Diameter of dolphin (ft)
B = Beam (width) of vessel (ft)
L = Distance of dolphin from pier (ft)
D_E = Effective dolphin diameter (ft)

a. Plan of Dolphin Protection.



b. Normal Distribution of Vessel Collision Trajectories Around Bridge Pier (σ assumed = 30°).

Figure C3.14.5.5-1—Illustrative Model of the Protection Factor (*PF*) of Dolphin Protection around a Bridge Pier

3.14.6—Design Collision Velocity

The design collision velocity may be determined as specified in Figure 3.14.6-1, for which:

V = design impact velocity (ft/s)
V_T = typical vessel transit velocity in the channel under normal environmental conditions but not taken to be less than V_{MIN} (ft/s)
V_{MIN} = minimum design impact velocity taken as not less than the yearly mean current velocity for the bridge location (ft/s)
X = distance to face of pier from centerline of channel (ft)
X_C = distance to edge of channel (ft)
X_L = distance equal to 3.0 times the length overall of the design vessel (ft)

The length overall, *LOA*, for barge tows shall be taken as the total length of the tow plus the length of the tug/tow boat.

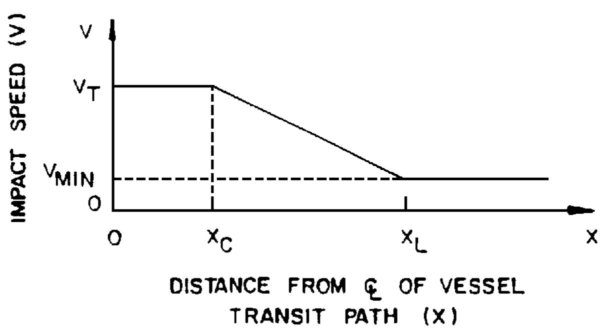


Figure 3.14.6-1—Design Collision Velocity Distribution

C3.14.6

A triangular distribution of collision impact velocity across the length of the bridge and centered on the centerline of the vessel transit path in the channel was based on historical accident data. This data indicated that aberrant ships and barges that collide with bridge piers further away from the channel are moving at reduced velocities compared with those hitting piers located closer to the navigable channel limits. Aberrant vessels located at long distances from the channel are usually drifting with the current. Aberrant vessels, located very near the channel, are moving at velocities approaching that of ships and barges in the main navigation channel.

The exact distribution of the velocity reduction is unknown. However, a triangular distribution was chosen because of its simplicity as well as its reasonableness in modeling the aberrant vessel velocity situation. The use of the distance 3.0 times *LOA* in Figure 3.14.6-1 to define the limits at which the design velocity becomes equal to that of the water current was based on the observation that very few accidents, other than with drifting vessels, have historically occurred beyond that boundary.

The selection of the design collision velocity is one of the most significant design parameters associated with the vessel collision requirements. Judgment should be exercised in determining the appropriate design velocity for a vessel transiting the waterway. The chosen velocity should reflect the "typical" transit velocity of the design vessel under "typical" conditions of wind, current, visibility, opposing traffic, waterway geometry, etc. A different vessel velocity may be required for inbound vessels than for outbound vessels given the presence of currents that may exist in the waterway.

In waterways subject to seasonal flooding, consideration should be given to flood flow velocities in determining the minimum collision velocity.

In general, the design velocity should not be based on extreme values representing extreme events, such as exceptional flooding and other extreme environmental conditions. Vessels transiting under these conditions are not representative of the "annual average" situations reflecting the typical transit conditions.

3.14.7—Vessel Collision Energy

The kinetic energy of a moving vessel to be absorbed during a noneccentric collision with a bridge pier shall be taken as:

$$KE = \frac{C_H W V^2}{29.2} \tag{3.14.7-1}$$

where:

KE = vessel collision energy (kip-ft)
W = vessel displacement tonnage (tonne)

C3.14.7

Eq. 3.14.7-1 is the standard $mV^2/2$ relationship for computing kinetic energy with conversion from mass to weight, conversion of units and incorporation of a hydrodynamic mass coefficient, C_H, to account for the influence of the surrounding water upon the moving vessel. Recommendations for estimating C_H for vessels moving in a forward direction were based on studies by Saul and Svensson (1980) and data published by PIANC (1984). It should be noted that these hydrodynamic mass coefficients are smaller than those normally used for ship berthing computations, in which a relatively large mass of water moves

C_H = hydrodynamic mass coefficient
V = vessel impact velocity (ft/s)

The vessel displacement tonnage, W, shall be based upon the loading condition of the vessel and shall include the empty weight of the vessel, plus consideration of the weight of cargo, DWT, for loaded vessels, or the weight of water ballast for vessels transiting in an empty or lightly loaded condition. The displacement tonnage for barge tows shall be the sum of the displacement of the tug/tow vessel and the combined displacement of a row of barges in the length of the tow.

The hydrodynamic mass coefficient, C_H, shall be taken as:

- If underkeel clearance exceeds 0.5 × draft:

$$C_H = 1.05 \quad (3.14.7\text{-}2)$$

- If underkeel clearance is less than 0.1 × draft:

$$C_H = 1.25 \quad (3.14.7\text{-}3)$$

Values of C_H may be interpolated from the range shown above for intermediate values of underkeel clearance. The underkeel clearance shall be taken as the distance between the bottom of the vessel and the bottom of the waterway.

with the vessel as it approaches a dock from a lateral, or broadside, direction.

3.14.8—Ship Collision Force on Pier

The head-on ship collision impact force on a pier shall be taken as:

$$P_S = 8.15\, V \sqrt{DWT} \quad (3.14.8\text{-}1)$$

where:

P_S = equivalent static vessel impact force (kip)
DWT = deadweight tonnage of vessel (tonne)
V = vessel impact velocity (ft/s)

C3.14.8

The determination of the impact load on a bridge structure during a ship collision is complex and depends on many factors as follows:

- Structural type and shape of the ship's bow,
- Degree of water ballast carried in the forepeak of the bow,
- Size and velocity of the ship,
- Geometry of the collision, and
- Geometry and strength characteristics of the pier.

Eq. 3.14.8-1 was developed from research conducted by Woisin (1976) in West Germany to generate collision data with a view to protecting the reactors of nuclear-powered ships from collisions with other ships. The ship collision data resulted from collision tests with physical ship models at scales of 1:12.0 and 1:7.5. Woisin's results have been found to be in good agreement with the results of research conducted by other ship collision investigators worldwide (IABSE, 1983).

Figure C3.14.8-1 indicates the scatter in Woisin's test data due to the various collision factors discussed herein, the triangular probability density function used to model the scatter, and the selection of a 70 percent fractile force for use as an equivalent static impact force for bridge design. Using a 70 percent fractile force for a given design vessel, the number of smaller ships with a crushing strength greater than this force would be approximately equal to the number of larger ships with a crushing strength less than this force. Figure C3.14.8-2 indicates typical ship impact forces computed with Eq. 3.14.8-1.

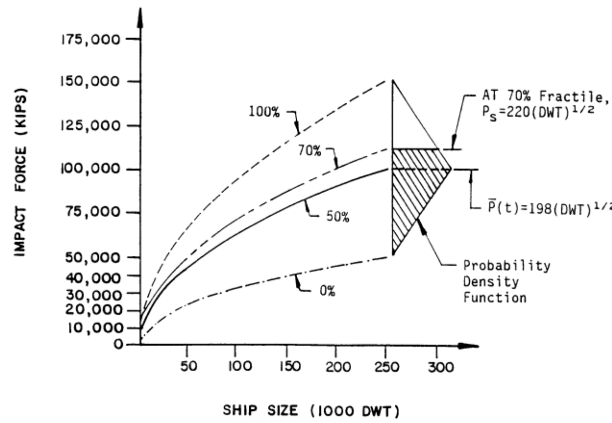


Figure C3.14.8-1—Probability Density Function of Ship Impact Force Data

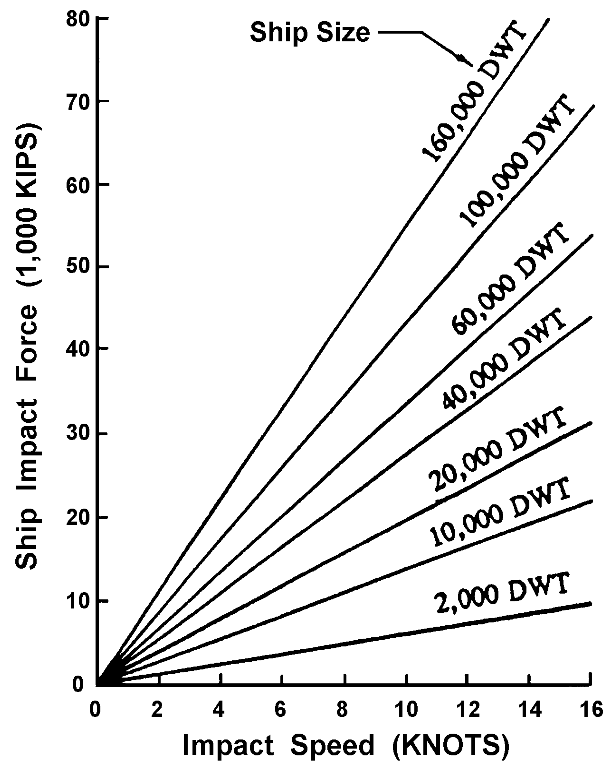


Figure C3.14.8-2—Typical Ship Impact Forces

3.14.9—Ship Bow Damage Length

The horizontal length of the ship's bow, crushed by impact with a rigid object, shall be taken as:

$$a_S = 1.54\left(\frac{KE}{P_S}\right) \qquad (3.14.9\text{-}1)$$

where:

a_s = bow damage length of ship (ft)
KE = vessel collision energy (kip-ft)
P_S = ship impact force as specified in Eq. 3.14.8-1 (kip)

C3.14.9

The average bow damage length, a, is computed based on the impact force averaged against the work path, $P(a)$, such that:

$$a = \frac{KE}{P(a)} \qquad (C3.14.9\text{-}1)$$

The 1.54 coefficient used to compute the design ship damage depth in Eq. 3.14.9-1 results from the multiplication of the following factors:

- 1.25 to account for the increase in average impact force over time versus damage length,
- 1.11 to account for the increase in average impact force to the 70 percent design fractile, and
- 1.11 to provide an increase in the damage length to provide a similar level of design safety as that used to compute P_S.

3.14.10—Ship Collision Force on Superstructure

3.14.10.1—Collision with Bow

The bow collision impact force on a superstructure shall be taken as:

$$P_{BH} = (R_{BH})(P_S) \qquad (3.14.10.1\text{-}1)$$

where:

P_{BH} = ship bow impact force on an exposed superstructure (kip)
R_{BH} = ratio of exposed superstructure depth to the total bow depth
P_S = ship impact force specified in Eq. 3.14.8-1 (kip)

For the purpose of this Article, exposure is the vertical overlap between the vessel and the bridge superstructure with the depth of the impact zone.

C3.14.10.1

Limited data exists on the collision forces between ship bows and bridge superstructure components.

3.14.10.2—Collision with Deck House

The deck house collision impact force on a superstructure shall be taken as:

$$P_{DH} = (R_{DH})(P_S) \qquad (3.14.10.2\text{-}1)$$

where:

P_{DH} = ship deck house impact force (kip)
R_{DH} = reduction factor specified herein
P_S = ship impact force as specified in Eq. 3.14.8-1 (kip)

C3.14.10.2

According to the Great Belt Bridge investigation in Denmark (Cowiconsult, Inc., 1981) forces for deck house collision with a bridge superstructure:

P_{DH} = 1,200 kip for the deck house collision of a 1,000 *DWT* freighter ship, and
P_{DH} = 6,000 kip for the deck house collision of a 100,000 *DWT* tanker ship.

Based on these values, the approximate empirical relationship of Eq. 3.14.10.2-1 was developed for selecting superstructure design impact values for deck house collision.

For ships exceeding 100,000 tonne, R_{DH} shall be taken as 0.10. For ships smaller than 100,000 tonne:

$$R_{DH} = 0.2 - \left(\frac{DWT}{100,000}\right)(0.10) \qquad (3.14.10.2\text{-}2)$$

3.14.10.3—Collision with Mast

The mast collision impact force on a superstructure shall be taken as:

$$P_{MT} = 0.10 P_{DH} \qquad (3.14.10.3\text{-}1)$$

where:

P_{MT} = ship mast impact force (kip)
P_{DH} = ship deck house impact force specified in Eq. 3.14.10.2-1 (kip)

C3.14.10.3

Eq. 3.14.10.3-1 was developed by estimating the impact forces based on bridge girder and superstructure damage from a limited number of mast impact accidents.

3.14.11—Barge Collision Force on Pier

For the purpose of Article 3.14, the standard hopper barge shall be taken as an inland river barge with:

width	=	35.0 ft
length	=	195.0 ft
depth	=	12.0 ft
empty draft	=	1.7 ft
loaded draft	=	8.7 ft
DWT	=	1,700 tons

The collision impact force on a pier for a standard hopper barge shall be taken as:

- If $a_B < 0.34$ then:

$$P_B = 4{,}112 a_B \qquad (3.14.11\text{-}1)$$

- If $a_B \geq 0.34$ then:

$$P_B = 1{,}349 + 110 a_B \qquad (3.14.11\text{-}2)$$

where:

P_B = equivalent static barge impact force (kip)
a_B = barge bow damage length specified in Eq. 3.14.12-1 (ft)

C3.14.11

There is less reported data on impact forces resulting from barge collisions than from ship collision. The barge collision impact forces determined by Eqs. 3.14.11-1 and 3.14.11-2 were developed from research conducted by Meir-Dornberg (*1983*) in West Germany. Meir-Dornberg's study included dynamic loading with a pendulum hammer on barge bottom models in scale 1:4.5, static loading on one bottom model in scale 1:6, and numerical analysis. The results for the standard European Barge, Type IIa, which has a similar bow to the standard hopper barge in the United States, are shown in Figure C3.14.11-1 for barge deformation and impact loading. No significant difference was found between the static and dynamic forces measured during the study. Typical barge tow impact forces using Eqs. 3.14.11-1 and 3.14.11-2 are shown in Figure C3.14.11-2.

where:

E_B = deformation energy (kip-ft)
$\overline{P}_B$ = average equivalent static barge impact force resulting from the study (kip)

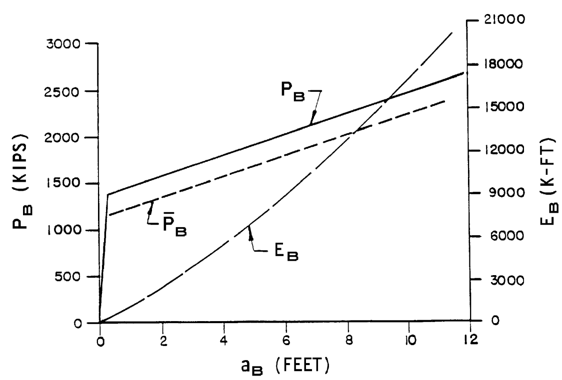


Figure C3.14.11-1—Barge Impact Force, Deformation

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Energy, and Damage Length Data

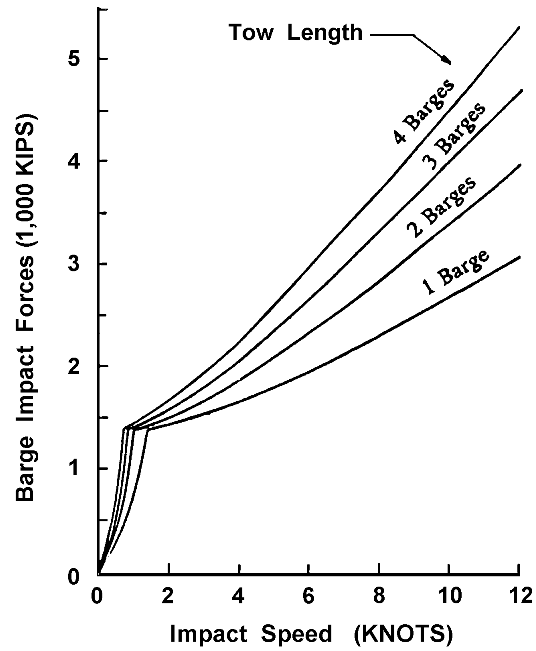

Figure C3.14.11-2—Typical Hopper Barge Impact Forces

Since the barge collision load formulation is for a standard rake head log height of 2.0 to 3.0 feet, the possibility of deeper head logs that may occur in tanker barges and special deck barges should also be considered. In lieu of better information, the barge force may be increased in proportion to the head log height compared to that of the standard hopper barge.

3.14.12—Barge Bow Damage Length

The barge bow horizontal damage length for a standard hopper barge shall be taken as:

$$a_B = 10.2\left(\sqrt{1+\frac{KE}{5{,}672}}-1\right) \quad (3.14.12\text{-}1)$$

where:

a_B = barge bow damage length (ft)
KE = vessel collision energy (kip-ft)

C3.14.12

The relationship for barge horizontal damage length, a_B, was developed from the same research conducted on barge collisions by Meir-Dornberg, as discussed in Article C3.14.11.

3.14.13—Damage at the Extreme Limit State

Inelastic behavior and redistribution of force effects is permitted in substructure and superstructure components, provided that sufficient ductility and redundancy of the remaining structure exists in the extreme event limit state to prevent catastrophic superstructure collapse.

As an alternative, pier protection may be provided for the bridge structure to eliminate or reduce the vessel collision loads applied to the bridge structure to acceptable levels.

C3.14.13

Two basic protection options are available to the Bridge Designer. The first option involves designing the bridge to withstand the impact loads in either an elastic or inelastic manner. If the response to collision is inelastic, the design must incorporate redundancy or other means to prevent collapse of the superstructure.

The second option is to provide a protective system of fenders, pile-supported structures, dolphins, islands, etc., either to reduce the magnitude of the impact

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loads to less than the strength of the bridge pier or superstructure components or to independently protect those components.

The requirements for either of these two options are general in nature because the actual design procedures that could be used vary considerably. This is particularly true for inelastic design. Because little information is available on the behavior of the inelastic deformation of materials and structures during the type of dynamic impacts associated with vessel impact, assumptions based on experience and sound engineering practice should be substituted.

3.14.14—Application of Impact Force

3.14.14.1—Substructure Design

For substructure design, equivalent static forces, parallel and normal to the centerline of the navigable channel, shall be applied separately as follows:

- 100 percent of the design impact force in a direction parallel to the alignment of the centerline of the navigable channel, or
- 50 percent of the design impact force in the direction normal to the direction of the centerline of the channel.

All components of the substructure, exposed to physical contact by any portion of the design vessel's hull or bow, shall be designed to resist the applied loads. The bow overhang, rake, or flair distance of ships and barges shall be considered in determining the portions of the substructure exposed to contact by the vessel. Crushing of the vessel's bow causing contact with any setback portion of the substructure shall also be considered.

The impact force in both design cases, specified herein, shall be applied to a substructure in accordance with the following criteria:

- For overall stability, the design impact force is applied as a concentrated force on the substructure at the mean high water level of the waterway, as shown in Figure 3.14.14.1-1, and
- For local collision forces, the design impact force is applied as a vertical line load equally distributed along the ship's bow depth, as shown in Figure 3.14.14.1-2. The ship's bow is considered to be raked forward in determining the potential contact area of the impact force on the substructure. For barge impact, the local collision force is taken as a vertical line load equally distributed on the depth of the head block, as shown in Figure 3.14.14.1-3.

C3.14.14.1

Two cases should be evaluated in designing the bridge substructure for vessel impact loadings:

- the overall stability of the substructure and foundation, assuming that the vessel impact acts as a concentrated force at the waterline, and
- the ability of each component of the substructure to withstand any local collision force resulting from a vessel impact.

The need to apply local collision forces on substructures exposed to contact by overhanging portions of a ship or barge's bow is well documented by accident case histories. The Sunshine Skyway Bridge in Tampa Bay, Florida, collapsed in 1980 as a result of the ship's bow impacting a pier column at a point 42.0 ft above the waterline. Ship and barge bow rake lengths are often large enough that they can even extend over protective fender systems and contact vulnerable bridge components, as shown in Figures C3.14.14.1-1 and C3.14.14.1-2. Bow shapes and dimensions vary widely, and the Designer may need to perform special studies to establish vessel bow geometry for a particular waterway location. Typical bow geometry data is provided in AASHTO (2009).

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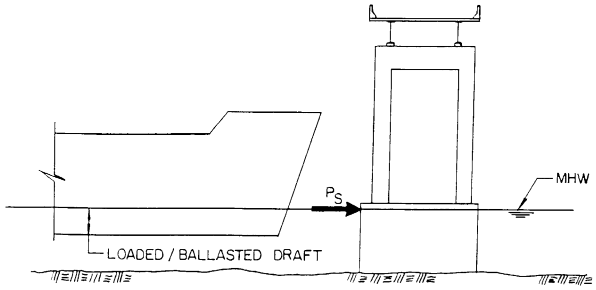

Figure 3.14.14.1-1—Ship Impact Concentrated Force on Pier

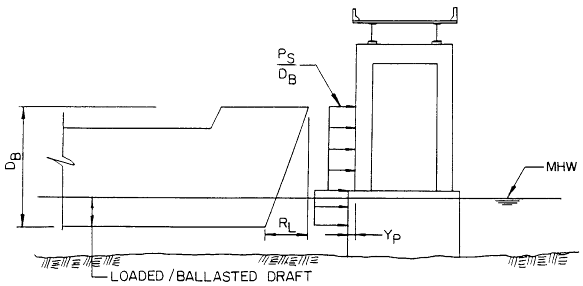

Figure 3.14.14.1-2—Ship Impact Line Load on Pier

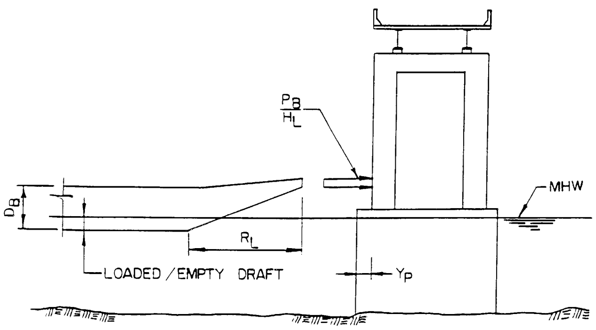

Figure 3.14.14.1-3—Barge Impact Force on Pier

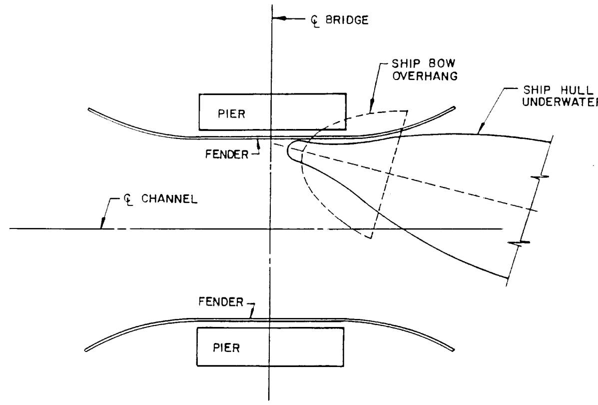

Figure C3.14.14.1-1—Plan of Ship Bow Overhang Impacting Pier

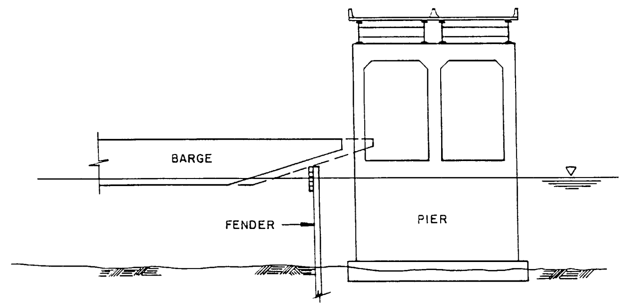

Figure C3.14.14.1-2—Elevation of Barge Bow Impacting Pier

3.14.14.2—Superstructure Design

For superstructure design, the design impact force shall be applied as an equivalent static force transverse to the superstructure component in a direction parallel to the alignment of the centerline of the navigable channel.

C3.14.14.2

The ability of various portions of a ship or barge to impact a superstructure component depends on the available vertical clearance under the structure, the water depth, vessel type and characteristics, and the loading condition of the vessel.

3.14.15—Protection of Substructures

Protection may be provided to reduce or to eliminate the exposure of bridge substructures to vessel collision by physical protection systems, including fenders, pile cluster, pile-supported structures, dolphins, islands, and combinations thereof.

Severe damage and/or collapse of the protection system may be permitted, provided that the protection system stops the vessel prior to contact with the pier or redirects the vessel away from the pier.

C3.14.15

The development of bridge protection alternatives for vessel collisions generally follows three approaches:

- Reducing the annual frequency of collision events, for example, by improving navigation aids near a bridge;
- Reducing the probability of collapse, for example, by imposing vessel speed restrictions in the waterway; or
- Reducing the disruption costs of a collision, for example, by physical protection and motorist warning systems.

Because modifications to navigation aids in the waterway and vessel operating conditions are normally beyond the Bridge Designer's ability to implement, the primary area of bridge protection to be considered by the Designer are physical protection and motorist warning systems.

The current practice in the design of protective structures is almost invariably based on energy considerations. It is assumed that the loss of kinetic energy of the vessel is transformed into an equal amount of energy absorbed by the protective structure. The kinetic impact energy is dissipated by the work done by flexure, shear, torsion, and displacement of the components of the protective system.

Design of a protective system is usually an iterative process in which a trial configuration of a protective system is initially developed. For the trial, a force versus deflection diagram is developed via analysis or physical modeling and testing. The area under the diagram is the energy capacity of the protective system. The forces and energy capacity of the protective system is then compared with the design vessel impact force and energy to see if the vessel loads have been safely resisted.

3.14.16—Security Considerations

The Owner of the bridge shall establish the size and velocity of the vessel to be used in bridge security analysis.

The vessel impact force shall be determined in accordance with Articles 3.14.8, 3.14.10.1, 3.14.10.2, or 3.14.10.3, as applicable.

The probability of bridge collapse due to intentional collision with the design vessel at the design speed shall be taken equal to *PC*, which shall be determined using the provisions of Article 3.14.5.4. The design vessel and velocity are site-specific variables that should be selected by the Owner as part of a security assessment.

C3.14.16

As the intent of intentionally ramming a vessel into a bridge is to cause the bridge to collapse, the velocity of the vessel at the moment of collision is expected to be higher than the normal travel speed. In addition to accounting for the effects of impact, consideration should also be given to the potential for vessel-delivered explosives and subsequent fire. The physical limitations on the velocity and size of the vessel should be taken into account when determining the design velocity for intentional collision as well as the likely maximum explosive size that can be delivered. For example, the velocity of a barge tow is limited by the power of the tug boats and by the geometry of the waterway in the approach to the bridge. Similarly, the factors limiting the size of the vessel should be considered when determining the design vessel.

In case of accidental collision, determining the annual probability of collapse using Eq. 3.14.5-1 involves the annual number of vessels, *N*, the probability of vessel aberrancy, *PA*, and the geometric probability of a collision, *PG*. In the case of intentional collision, the value of each of the three variables may be taken as 1.0. Therefore, the probability of collapse in case of intentional collision is taken equal to *PC*.

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3.15—BLAST LOADING: BL

3.15.1—Introduction

Where it has been determined that a bridge or a bridge component should be designed for intentional or unintentional blast force, the following should be considered:

- Size of explosive charge,
- Shape of explosive charge,
- Type of explosive,
- Stand-off distance,
- Location of the charge,
- Possible modes of delivery and associated capacities (e.g., maximum charge weight will depend upon vehicle type and can include cars, trucks, ships, etc.), and
- Fragmentation associated with vehicle-delivered explosives.

C3.15.1

The size, shape, location, and type of an explosive charge determine the intensity of the blast force produced by an explosion. For comparison purposes, all explosive charges are typically converted to their equivalent TNT charge weights.

Stand-off refers to the distance between the center of an explosive charge and a target. Due to the dispersion of blast waves in the atmosphere, increasing stand-off causes the peak pressure on a target to drop as a cubic function of the distance (i.e., for a given quantity of explosives, doubling the stand-off distance causes the peak pressure to drop by a factor of eight). The location of the charge determines the amplifying effects of the blast wave reflecting from the ground surface or from the surfaces of surrounding structural elements. The location of the charge also determines the severity of damage caused by fragments from the components closest to the blast traveling away from the blast center.

Information on the analysis of blast loads and their effects on structures may be found in J. M. Biggs (1964), W. E. Baker, et al. (1983), Department of the Army (1990), P. S. Bulson (1997), and Department of the Army (1986).

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APPENDIX A3—SEISMIC DESIGN FLOWCHARTS

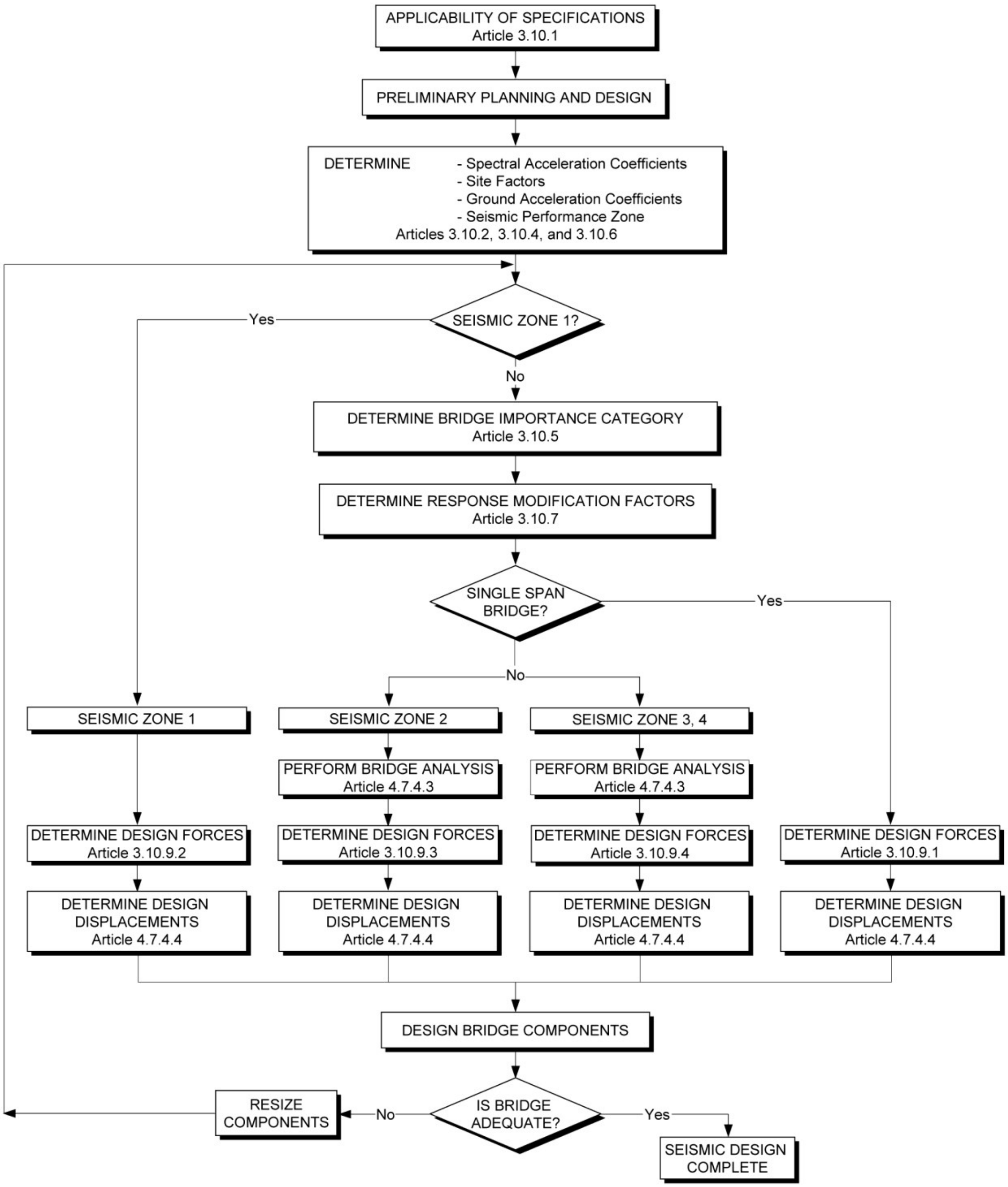

Figure A3-1—Seismic Design Procedure Flow Chart

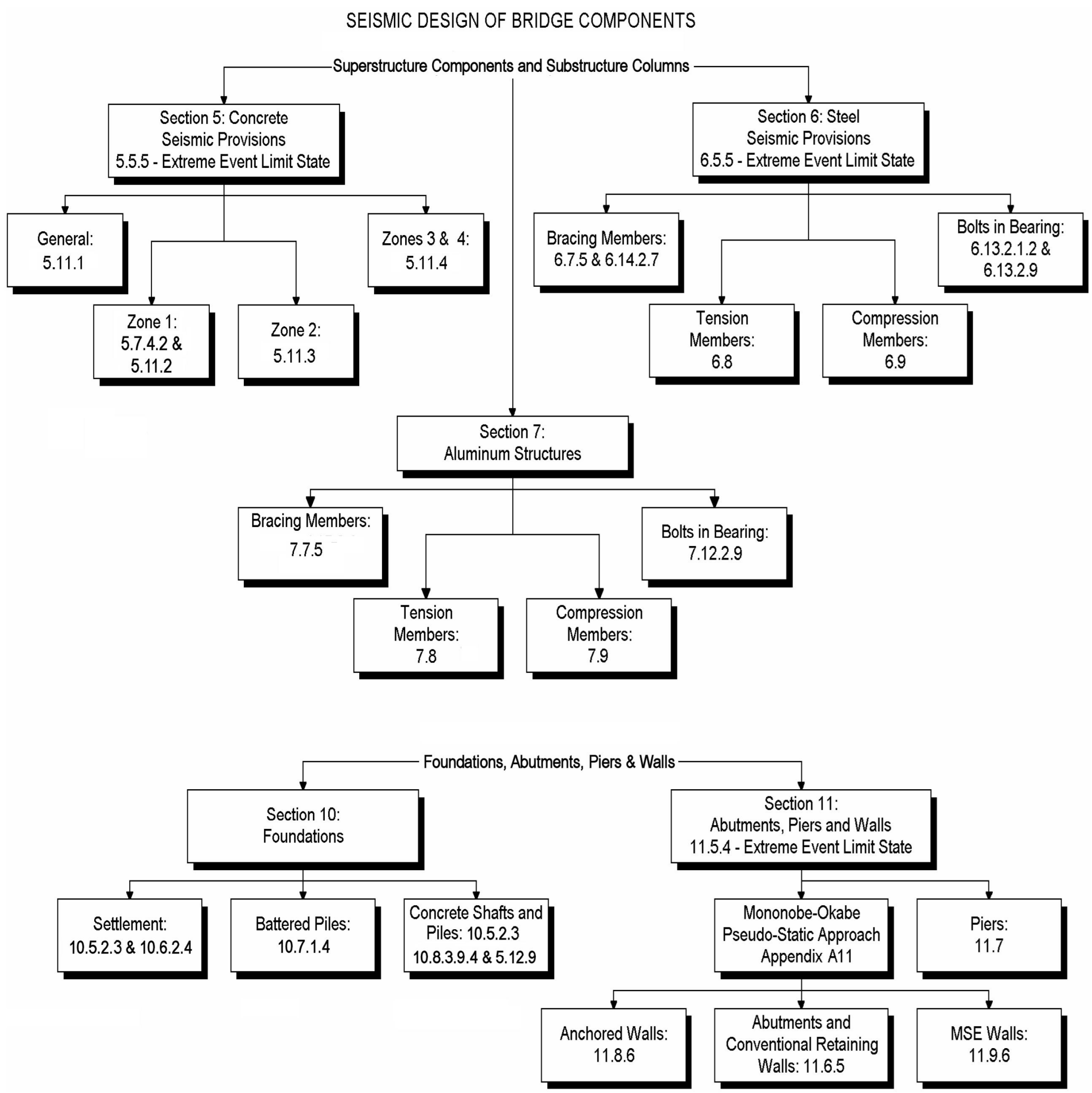

Figure A3-2—Seismic Detailing and Foundation Design Flow Chart

APPENDIX B3—OVERSTRENGTH RESISTANCE

Article 3.10.9.4.3a defines the forces resulting from plastic hinging, i.e., a column reaching its ultimate moment capacity, in the columns and presents two procedures. One is for a single column hinging about its two principal axes; this is also applicable for piers and bents acting as single columns. The other procedure is for a multiple column bent in the plane of the bent. The forces are based on the potential overstrength resistance of the materials, and to be valid the design detail requirements of this Section must be used so that plastic hinging of the columns can occur. The overstrength resistance results from actual properties being greater than the minimum specified values and is implemented by specifying resistance factors greater than unity. This fact must be accounted for when forces generated by yielding of the column are used as design forces. Generally, overstrength resistance depends on the following factors:

- The actual size of the column and the actual amount of reinforcing steel.
- The effect of an increased steel strength over the specified f_y and for strain hardening effects.
- The effect of an increased concrete strength over the specified f'_c and confinement provided by the transverse steel. Also, with time, concrete will gradually increase in strength.
- The effect of an actual concrete ultimate compressive strain above 0.003.

Column Size and Reinforcement Configuration

The design engineer should select the minimum column section size and steel reinforcement ratio when satisfying structural design requirements. As these parameters increase, the overstrength resistance increases. This may lead to an increase in the foundation size and cost. A size and reinforcement ratio which forces the design below the nose of the interaction curve is preferable, especially in high seismic areas. However, the selection of size and reinforcement must also satisfy architectural, and perhaps other requirements, which may govern the design.

Increase in Reinforcement Strength

Almost all reinforcing bars will have a yield strength larger than the minimum specified value which may be up to 30 percent higher, with an average increase of 12 percent. Combining this increase with the effect of strain hardening, it is realistic to assume an increased yield strength of 1.25 f_y, when computing the column overstrength.

Increase in Concrete Strength

Concrete strength is defined as the specified 28-day compression strength; this is a low estimate of the strength expected in the field. Typically, conservative concrete batch designs result in actual 28-day strengths of about 20–25 percent higher than specified. Concrete will also continue to gain strength with age. Tests on cores taken from older California bridges built in the 1950s and 1960s have consistently yielded compression strength in excess of 1.5 f'_c. Concrete compression strength is further enhanced by the possible confinement provided by the transverse reinforcement. Rapid loading due to seismic forces could also result in significant increase in strength, i.e., strain rate effect. In view of all the above, the actual concrete strength when a seismic event occurs is likely to significantly exceed the specified 28-day strength. Therefore, an increased concrete strength of 1.5 f'_c could be assumed in the calculation of the column overstrength resistance.

Ultimate Compressive Strain (ε_c)

Although tests on unconfined concrete show 0.003 to be a reasonable strain at first crushing, tests on confined column sections show a marked increase in this value. The use of such a low extreme fiber strain is a very conservative estimate of strains at which crushing and spalling first develop in most columns, and considerably less than the expected strain at maximum response to the design seismic event. Research has supported strains on the order of 0.01 and higher as the likely magnitude of ultimate compressive strain. Therefore, designers could assume a value of ultimate strain equal to 0.01 as a realistic value.

For calculation purposes, the thickness of clear concrete cover used to compute the section overstrength shall not be taken to be greater than 2.0 in. This reduced section shall be adequate for all applied loads associated with the plastic hinge.

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Overstrength Capacity

The derivation of the column overstrength capacity is depicted in Figure B3-1. The effect of higher material properties than specified is illustrated by comparing the actual overstrength curve, computed with realistic f'_c, f_y, and ε_c values, to the nominal strength interaction curve, P_n, M_n. It is generally satisfactory to approximate the overstrength capacity curve by multiplying the nominal moment strength by the 1.3 factor for axial loads below the nose of the interaction curve, i.e., P_n, 1.3 M_n curve. However, as shown, this curve may be in considerable error for axial loads above the nose of the interaction curve. Therefore, it is recommended that the approximate overstrength curve be obtained by multiplying both P_n and M_n by $\phi = 1.3$, i.e., 1.3 P_n, 1.3 M_n. This curve follows the general shape of the actual curve very closely at all levels of axial loads.

In the light of the above discussion, it is recommended that:

- For all bridges with axial loads below P_b, the overstrength moment capacity shall be assumed to be 1.3 times the nominal moment capacity.
- For bridges in Zones 3 and 4 with operational classification of "other", and for all bridges in Zone 2 for which plastic hinging has been invoked, the overstrength curve for axial loads greater than P_b shall be approximated by multiplying both P_n and M_n by $\phi = 1.3$.
- For bridges in Zones 3 and 4 with operational classification of "essential" or "critical," the overstrength curve for axial loads greater than P_b shall be computed using realistic values for f'_c, f_y and ε_c as recommended in Table B3-1 or from values based on actual test results. The column overstrength, thus calculated, should not be less than the value estimated by the approximate curve based on 1.3 P_n, 1.3 M_n.

Table B3-1—Recommended Increased Values of Materials Properties

Increased f_y (minimum)	1.25 f_y
Increased f'_c	1.5 f'_c
Increased ε_c	0.01

Shear Failure

The shear mode of failure in a column or pile bent will probably result in a partial or total collapse of the bridge; therefore, the design shear force must be calculated conservatively. In calculating the column or pile bent shear force, consideration must be given to the potential locations of plastic hinges. For flared columns, these may occur at the top and bottom of the flare. For multiple column bents with a partial-height wall, the plastic hinges will probably occur at the top of the wall unless the wall is structurally separated from the column. For columns with deeply embedded foundations, the plastic hinge may occur above the foundation mat or pile cap. For pile bents, the plastic hinge may occur above the calculated point of fixity. Because of the consequences of a shear failure, it is recommended that conservatism be used in locating possible plastic hinges such that the smallest potential column length be used with the plastic moments to calculate the largest potential shear force for design.

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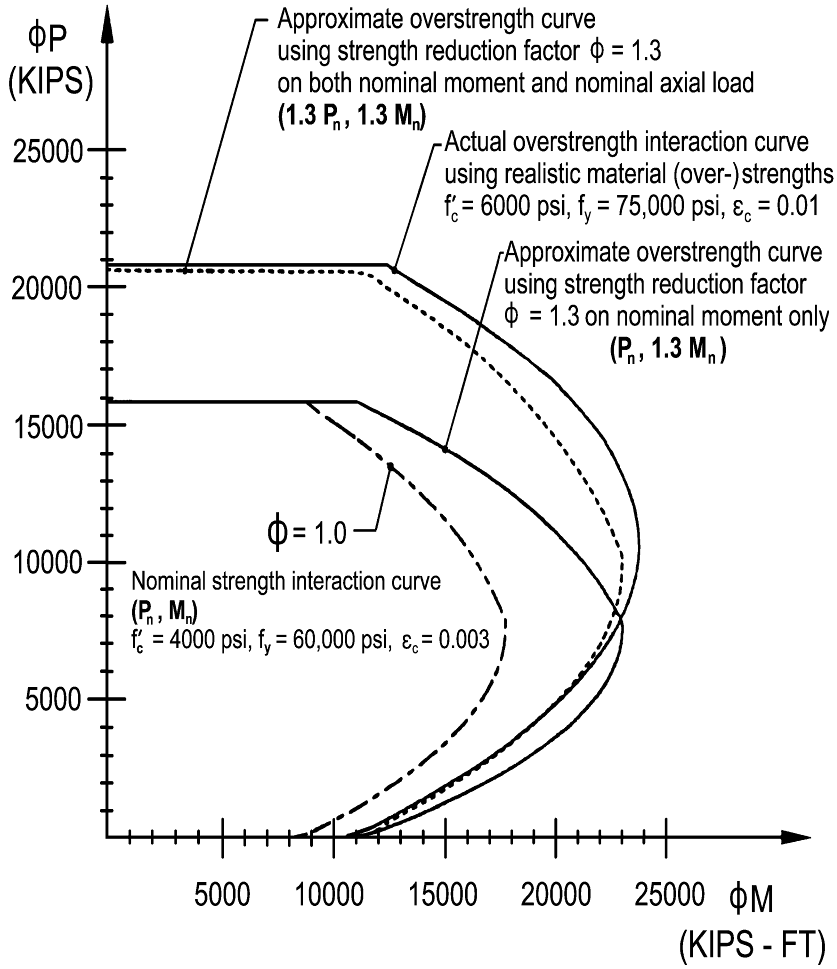

Column properties: 5' 10-3/4" x 5' 10-3/4" section
A_s = 32 #11 bars (1.05%)
f'_c = 4000 psi
f_y = 60,000 psi
ε_c = 0.003

Figure B3-1—Development of Approximate Overstrength Interaction Curves from Nominal Strength Curves after Gajer and Wagh (1994)