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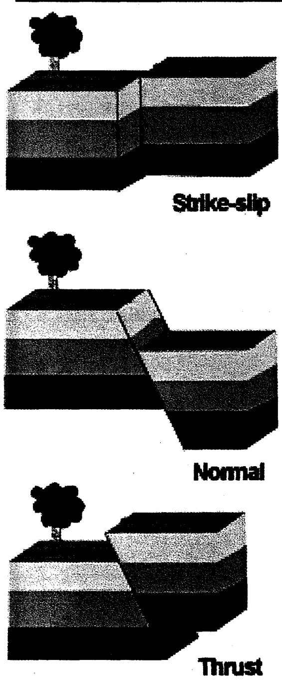
Seismic for California Civil Engineering License

2007 California Building Code (CBC)
2006 International Building Code (IBC)/ASCE 7-05



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Glossary & Definitions

Acceleration: The rate of change of velocity of a reference point. Commonly expressed as a fraction or percentage of the acceleration due to gravity (g), where g = 386.4 in/s^2 or 980 cm/s^2.

Accelerogram: The record from an accelerometer (or accelerograph) showing acceleration as a function of time

Accelerograph: A compact, rugged, and relatively inexpensive instrument that records the signal from an accelerometer. Film is the most common recording medium.

Accelerometer: A sensor whose output is almost directly proportional to ground acceleration. The conventional strong-motion accelerometer is a simple, nearly critically damped oscillator having a natural frequency of about 20 Hz.

Acceptable risk: The level and probability of physical damage and deaths judged by appropriate authorities to represent a basis for design requirements in engineered structures.

Active Fault: A fault determined to be active by the authority having jurisdiction from properly substantiated data (e.g., most recent mapping of active faults by the United States Geological Survey).

Actual liquefaction: The condition when the pore pressures in a saturated sand undergoing large shear strains or continued cyclic loading rise to a level such that the sand tends to flow like a heavy fluid. Note that the term is qualitative as there is yet no general agreement on its quantitative definition.

Addition: An increase in building area, aggregate floor area, height, or number of stories of a structure.

Aftershocks: Secondary tremors that may follow the largest shock of an earthquake sequence. Such tremors can extend over a period of weeks, months, or years.

Allowable Stress Design (ASD): is a method of proportioning structural elements such that the computed stresses produced in the elements by the allowable stress load combinations do not exceed the specified allowable stress. Also known as *Working Stress Design (WDS) & Service Load Design*.

e.g.: calculated bending stress ≤ allowable bending stress

$$\sigma_{actual} \leq \sigma_{allowable}$$

Alteration: Any construction or renovation to an existing structure other than an addition.

Alluvium: Loosely compacted gravel, sand, silt, or clay deposited by streams.

Amplification: An increase in seismic-signal amplitude within some range of frequency as waves propagate through different earth materials. The signal is both amplified and deamplified at the same site in a manner that is dependent on the frequency band. The degree of amplification is also a complex function of the level of shaking such that, as the level of shaking increases, the amount of amplification may decrease. Shaking levels at a site may also be increased by focusing of seismic energy caused by the geometry of the sediment velocity structure, such as basin subsurface topography, or by surface topography.

Amplitude: Zero-to-peak value of any wavelike disturbance.

Appendage: An architectural component such as a canopy, marquee, ornamental balcony, or statuary.

Approval: The written acceptance by the authority having jurisdiction of documentation that establishes the qualification of a material, system, component, procedure, or person to fulfill the requirements of this standard for the intended use.

Attachment: Means by which components and their supports are secured or connected to the seismic force-resisting system of the structure. Such attachments include anchor bolts, welded connections, and mechanical fasteners.

Attenuation: Characteristic decrease in amplitude of the seismic waves with distance from source. Attenuation results from geometric spreading of propagating waves, energy absorption, and scattering of waves.

Base: is the level at which the earthquake motions are considered to be imparted to the structure, or the level at which the structure as a dynamic vibrator is supported.

Base shear (V): is the total design lateral force or shear at the base of the structure. Horizontal shear force transmitted to the structure from oscillating bearing soils or rock. The base shear is numerically equal to the resultant equivalent lateral force in pseudostatic analyses and is assumed to be transmitted to supporting subgrade material or piles at the level of the base of spread foundations or the base of pile-caps.

Bearing Wall: is a wall that supports any vertical load in addition to its own weight.

Bearing Wall System: is a structural system that without a complete vertical load-carrying space frame.

Bedrock: Relatively hard, solid rock that commonly underlies softer rock, sediment, or soil.

Boundary Element: is an element at the perimeter of shear walls or diaphragms, or at the edges of openings.

Braced Frame: is an essentially vertical truss system, of the concentric or eccentric type, that is provided to resist lateral forces.

Brittle-ductile boundary: A depth in the crust across which the thermo mechanical properties of the crust change from brittle above to ductile below. A large percentage of the earthquakes in the crust initiate at or above this depth on high-angle faults; below this depth, fault slip may be a seismic and may grade from high angle to low angle.

Building frame system: A structural system with an essentially complete space frame providing support for vertical loads. Seismic force resistance is provided by shear walls or braced frames.

Bulk density: The mass of a material divided by its volume, including the volume of its pore spaces.

Cantilevered Column Element: is a column element in a lateral-force-resisting system that cantilevers from a fixed base and has minimal moment capacity at the top, with lateral forces essentially applied at the top.

Chord: is a boundary element of a horizontal diaphragm (or shear wall) that is perpendicular to the direction of lateral load under consideration. A chord is assumed to resist axial stresses (tension and compression) due to flexure, analogous to the flanges of a steel beam.

Collector: is a boundary element (or member) of a horizontal diaphragm that is parallel to the direction of lateral load under consideration, which collects and transmits the diaphragm shear to the vertical lateral-force-resisting elements (e.g. shear walls, braced frames, etc.). Such members may take axial tension or compression.

Column: is a member subjected primarily to axial compressive load.

Component: is a part or element of an architectural, electrical, mechanical or structural system.

Component, Equipment: is a mechanical or electrical component or element that is part of a mechanical and/or electrical system.

Component, Flexible: is a component, including its attachments, having a fundamental period greater than 0.06 second.

Component, Rigid: is a component, including its attachments, having a fundamental period less than or equal to 0.06 second.

Compression: is a force (action of one body on another) or stress (force per unit area) that tends to shorten a member (or crush it).

Concentrically Braced Frame is a braced frame in which the members are subjected primarily to axial (tension or compression) forces.

Colluvium: Loose soil or rock fragments on or at the base of gentle slopes or hillsides. Deposited by or moving under the influence of rain wash or downhill creep.

Compressional wave: See P wave.

Convergence zone: The area along plate boundaries where plates collide and the boundaries are absorbed by shortening and thickening or subduction.

Creep: Slow, more or less continuous movement occurring on faults due to ongoing tectonic deformation. Also applied to slow movement of landslide masses down a slope because of gravitational forces. Faults that undergo significant and (or) ongoing creep are likely to be a seismic or capable of only small or moderate earthquakes.

Critical acceleration: The magnitude of seismic acceleration, directed down slope, that imposes a force on a sliding mass that equals the available resistance on the assumed sliding plane minus the resistance needed for static stability. Accelerations larger than this magnitude may cause a net down slope movement of the sliding mass.

Critical facilities: Structures whose ongoing performance during an emergency is required or whose failure could threaten many lives. May include (1) structures such as nuclear power reactors or large dams whose failure might be catastrophic; (2) major communication, utility, and transportation systems; (3) involuntary- or high-occupancy buildings such as schools or prisons; and (4) emergency facilities such as hospitals, police and fire stations, and disaster-response centers.

Critical failure plane: An element in static and pseudostatic stability analysis. It is the lower boundary of the free body x whose stability obtains the minimum safety factor in the analysis. It is determined by a series of trials and can be a curvilinear surface, a straight plane, or a surface formed by continuous segments of various forms.

Crust: The outermost major layer of the Earth, ranging from about 10 to 65 km in thickness worldwide. The continental crust is about 40 km thick in the Pacific Northwest. The thickness of the oceanic crust in this region varies between about 10 and 15 km. The crust is characterized by P-wave velocities less than about 8 km/s. The uppermost 15-35 km of crust is brittle enough to produce earthquakes. The seismic crust is separated from the lower crust by the brittle-ductile boundary.

Cyclic shear test: A laboratory test commonly performed in a biaxial direct shear or torsional shear device where a cyclic load is applied at a uniform frequency and at a selected maximum value of shear stress less than the peak static failure strength.

Cyclic stress ratio: The ratio of maximum value of applied cyclic shear stress to the initial effective consolidation stress on the failure plane of a cyclic shear test. It is a function of the number of applied cycles necessary to obtain failure. Laboratory values of the ratio are compared to the expected stress ratio induced by the design earthquake to estimate the susceptibility of a soil to liquefaction after a given number of applied stress cycles.

Damping: The reduction in amplitude of a seismic wave or oscillator due to friction and (or) the internal absorption of energy by matter.

Dead Load: is the vertical load due to the weight of all permanent structural and nonstructural components of a building, such as walls, floors, roofs and fixed service equipment.

Deflection: is the movement of structural element (beam, horizontal diaphragm, or shear wall) from its original location when a load is applied.

Design Basis Ground Motion: is that ground motion that has a 2 percent chance of being exceeded in 50 years as determined by a site-specific hazard analysis or may be determined from a hazard map. A suite of ground motion time histories with dynamic properties representative of the site characteristics shall be used to represent this ground motion. The dynamic effects of the Design Basis Ground Motion may be represented by the Design Response Spectrum.

Design earthquake: The postulated earthquake (commonly including a specification of the ground motion at a site) that is used for evaluating the earthquake resistance of a particular structure.

Design Response Spectrum: is an elastic response spectrum for 5 percent equivalent viscous damping used to represent the dynamic effects of the Design Basis Ground Motion for the design of structures.

Design Seismic Force: is the minimum total strength design base shear, factored and distributed in accordance with

Design spectra: A set of curves used for design that shows the envelope of maximum acceleration, velocity, or displacement (usually absolute acceleration, relative velocity, and relative displacement) of the vibrating mass of an elastic body reacting to earthquake shaking applied at its base as a function of the natural period of vibration and damping of the body.

Diaphragm: is a horizontal (or nearly horizontal) system acting to transmit lateral forces (wind or earthquake) to the vertical lateral-force-resisting elements. The term "diaphragm" includes horizontal bracing systems.

Diaphragm Strut: See Collector.

Dip: Inclination of a planar geologic surface (for example, a fault or a bed) from the horizontal.

Dip slip: See Fault.

Displacement: The difference between the initial position of a reference point and any later position. (1) In seismology, displacement is the ground motion commonly inferred from a seismogram. For example, it may be calculated by integrating an accelerogram twice with respect to time and is expressed in units of length, such as centimeters. (2) In geology, displacement is the permanent offset of a geologic or man-made reference point along a fault or a landslide.

Drag Force Diagram: is a graphical representation of the value of the drag (or collector) force at any point along a collector (or drag strut) member's length.

Drag Strut: See Collector.

Drift: The relative lateral displacement between adjacent floors in a building during earthquake shaking.

Drift limit: The allowable drift permitted in the design as defined by building codes.

Dual structural system: A structural system with an essentially complete space frame providing support for vertical loads. The total seismic force resistance is provided by the combination of the special moment frame and shear walls or braced frames in proportion to their relative rigidities. *ASCE7-05 §12.2.5.1*

Ductility: The property of yielding to deformation by an element of a structure in which elastic and plastic deformations are sustained without significant loss of strength.

Ductility factor: The ratio of the maximum elastic plus plastic deflection developed in a structural member during a design earthquake to the maximum elastic-limit deformation.

Ductility Factor = U_T / U_R = Toughness (Total Energy)/Resilience (Elastic Energy)

Duration of strong ground motion: The length of time during which ground motion at a site exceeds a designated threshold of severity.

Dynamic analysis: Refers to an analysis where an earthquake base acceleration is numerically propagated through an idealized structure or earth mass to determine the response accelerations of points within the structure or earth mass. Responses are functions of the characteristic modes of vibration of elements of the structure or points within the mass. The response accelerations determine the magnitudes of earthquake forces that may produce instabilities. The stability analysis of a soil mass should use resisting shear strengths that are reduced in proportion to the pore pressure buildup during shaking.

Earthquake: Ground shaking and radiated seismic energy caused most commonly by sudden slip on a fault, volcanic or magmatic activity, or other sudden stress changes in the Earth. An earthquake of magnitude 8 or larger is termed a great earthquake.

Note: The following definitions are for specific earthquake terms. The usages are highly specialized, often redundant, sometimes limited to requirements of special groups, and not always felicitous. The "recommended definitions" are suitable for general use.

1. **Maximum Possible Earthquake:** The largest earthquake that can be postulated to occur. Conceptual only. Probable magnitude 8.7 to 9.5.

2. **Maximum Credible Earthquake (MCE).**

2.1 RECOMMENDED DEFINITION**:** The largest earthquake effects considered by applicable Code or Standard.

3. **Maximum Expectable Earthquake**: The largest earthquake that can be reasonably expected to occur. (U.S. Geological Survey. Same as Maximum credible earthquake.)

Earthquake hazard: Any physical phenomenon associated with an earthquake that may produce adverse effects on human activities. This includes surface faulting, ground shaking, landslides, liquefaction, tectonic deformation, tsunami, and their effects on land use, manmade structures, and socioeconomic systems. A commonly used restricted definition of earthquake hazard is the probability of occurrence of a specified level of ground shaking in a specified period of time.

Earthquake risk : The expected (or probable) life loss, injury, or building damage that will happen, given the probability that some earthquake hazard occurs. Earthquake risk and earthquake hazard are occasionally used interchangeably.

Eccentric braced frame: A diagonally steel-braced frame in which at least one end of each brace frames into a beam a short distance from a beam column joint or from another diagonal brace.

Effective peak acceleration: The maximum value of acceleration in an earthquake accelerogram after the record has been adjusted to delete motions considered unimportant for structural response.

***Elastic limit*:** Deformation or strain beyond which a structural element begins to distort plastically and loses the capacity of recovering size and shape after deformation.

Elastic Response Parameters: are forces and deformations determined from an elastic dynamic analysis using an unreduced ground motion representation.

Epicenter: The point on the Earth's surface vertically above the point (focus or hypocenter) in the crust where the initial earthquake (seismic rupture) ground motion originates.

Equivalent lateral force (ELF): A static horizontal force distributed vertically on a structure. The ELF is intended to simulate the effect on the structure of the dynamic loads that will occur during the design earthquake. It is usually computed from building code stipulations as a proportion of the building's dead weight.

Equivalent seismic coefficient: That proportion of the total weight of a structure or structural element that is applied horizontally and vertically in a pseudostatic analysis to approximate earthquake-generated forces.

Essential Facilities: are those structures that are necessary for emergency operations subsequent to a natural disaster.

Factored Load: is the product of a nominal load and a load factor specified in *IBC §1605.*

Fault: A fracture along which there has been significant displacement of the two sides relative to each other parallel to the fracture. Strike-slip faults are vertical (or nearly vertical) fractures along which rock masses have mostly shifted horizontally. If the block opposite an observer looking across the fault moves to the right, the slip style is termed right lateral; if the block moves to the left, the motion is termed left lateral. Dip-slip faults are inclined fractures along which rock masses have mostly shifted vertically. If the rock mass above an inclined fault is depressed by slip, the fault is termed normal, whereas if the rock above the fault is elevated by slip, the fault is termed reverse (or thrust). Oblique-slip faults have significant components of both slip styles.

Fault trace: Intersection of a fault with the ground surface; also, the line commonly plotted on geologic maps to represent a fault.

First motion: On a seismogram, the direction of ground motion as the P wave arrives at the seismometer. Upward ground motion indicates an expansion in the source region; downward motion indicates a contraction.

Flexible Element or System: is one whose deformation under lateral load is significantly larger than adjoining parts of the system.

Flexure rigidity: Also referred to as flexural stiffness. Expressed as the product of the modulus of elasticity (E) and the moment of inertia (I) of a structural member.

Flow slide: The ultimate response of a loose, saturated soil to the buildup of pore pressure created by applied shear stress, wherein pore pressure increases to essentially eliminate effective intergranular stresses and the soil mass fails very rapidly and flows in the manner of a liquid.

Focal depth: The vertical distance between the hypocenter or focus at which an earthquake is initiated and the epicenter at the ground surface.

Focus: The location within the earth where the slip responsible for an earthquake was initiated. Also called the hypocenter of an earthquake.

Foreshocks, main shock, aftershocks: Foreshocks are relatively smaller earthquakes that precede the biggest earthquake in a series, which is termed the main shock. Aftershocks are relatively smaller earthquakes that follow the main shock.

Frequency: Number of cycles occurring in unit time.

Friction heat: A temperature increase on the slip zone of a massive slide or along a moving fault plane that results from energy expended in overcoming friction. The consequence can be a dramatic rise in internal pore fluid pressures leading to rapid strength loss and acceleration of the sliding movements.

Fundamental period: The longest period for which a structure shows a maximum response. The reciprocal of natural frequency.

G or g: See Acceleration.

Geodetic: Referring to the determination of the size and shape of the Earth and the precise location of points on its surface.

Geometrical attenuation: That component of attenuation of seismic-wave amplitudes due to the radial spreading of seismic energy with distance from a given source.

Geomorphology: The study of the character and origin of landform.

Geotechnical: Referring to the use of scientific methods and engineering principles to acquire, interpret, and apply knowledge of earth materials for solving engineering problems.

Gravity: The attraction between two masses, such as the Earth and an object on its surface. Commonly referred to as the acceleration of gravity. Changes in the gravity field can be used to infer information about the structure of the earth's lithosphere and upper mantle. Interpretations of changes in the gravity field are generally applied to gravity values corrected for extraneous effects. The corrected values are referred to by various terms, such as free-air gravity, Bouguer gravity, and isostatic gravity, depending on the number of corrections.

Ground failure: A general reference to landslides, liquefaction, and lateral spreads.

Ground motion: Numerical values quantifying vibratory ground motion, such as particle acceleration, velocity, displacement, frequency content, predominant period, spectral values, intensity, and duration.

Ground motion (shaking): General term referring to the qualitative or quantitative aspects of movement of the Earth's surface from earthquakes or explosions. Ground motion is produced by waves that are generated by sudden slip on a fault or sudden pressure at the explosive source and travel through the Earth and along its surface.

Hazard: See Earthquake hazard.

Hertz (Hz): A unit of frequency. Expressed in cycles per second.

Holocene: Refers to a period of time between the present and 10,000 years before present. Applied to rocks or faults, this term indicates the period of rock formation or the time of most recent fault slip. Faults of this age are commonly considered active, based on the observation of historical activity on faults of this age in other locales.

Horizontal diaphragms: Floor and roof systems designed to support gravity loads and to transfer these loads to vertical structural members.

Hotspot: An area where the seismicity is anomalously high compared with a surrounding region.

Horizontal Bracing System: is a horizontal truss system that serves the same function as a diaphragm.

Hypocenter: The point within the Earth where an earthquake rupture initiates. Also commonly termed the focus.

Initial liquefaction: The stage in a cyclic shear test when the maximum pore pressure occurring during a cycle of shear equals the initial effective consolidation pressure, which is the pressure applied in the consolidation phase. Thus, at this point, the effective intergranular stress in the sample has been reduced to zero.

Intensity: A subjective numerical index describing the effects of an earthquake on humans, on their structures, and on the earth's surface at a particular place. The number is rated on the basis of an earthquake intensity scale. Several scales exist, but the ones most commonly known in the United States are the Modified Mercalli scale and the Rossi-Forel scale. The scale in common use in the U.S. today is the Modified Mercalli (MM) Intensity Scale of 1931 with intensities indicated by Roman numerals from I to XII. In general, for a given earthquake, intensity will decrease with distance from the epicenter. The following is abridgement of the scale:

I. Not felt except by a very few under especially favorable conditions.

II. Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing.

III. Felt quite noticeably indoors, especially on upper floors of buildings, but many people do not recognize it as an earthquake. Standing automobiles may rock slightly. Vibration like passing of truck. Duration can be estimated.

IV. During the day felt indoors by many, outdoors by few. At night some awakened. Dishes, windows, doors disturbed; walls make cracking sound. Sensation like heavy truck striking building. Standing automobiles rocked noticeably.

V. Felt by nearly everyone; many awakened. Some dishes, windows, and other fragile items broken; a few instances of cracked plaster; unstable objects overturned. Disturbance of frees, poles and other
tall objects sometimes noticed. Pendulum clocks may stop.

VI. Felt by all; many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight.

VII. Everybody runs outdoors. Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable in poorly built or badly designed structures. Some chimneys broken. Noticed by persons driving automobiles.

VIII. Damage slight in specially designed structures; considerable in ordinary substantial buildings with partial collapse. Great damage in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Persons driving automobiles disturbed.

IX. Damage considerable in specially designed structures; well-designed frame structures thrown out-of-plumb; damage great in substantial buildings, with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken

X. Some well-built wooden structures destroyed; most masonry and frame structures destroyed. Ground badly cracked. Railroad rails bent. Many landslides on river banks and steep slopes. Shifted sand and mud. Water splashed over banks of rivers and lakes.

XI. Few structures remain standing; Unreinforced masonry structures are nearly totally destroyed. Bridges destroyed. Broad fissures in ground. Underground pipe lines completely out of service. Earth slumps and land slips in soft ground. Railroad rails bent greatly.

XII. Damage total; Waves apparently seen on ground surfaces. Lines of sight and level appear visually distorted. Objects thrown upward into the air.

Isolation bearings: Absorption devices that dissipate seismic energy prior to its entering a structure through the effects of elastic deformation and mechanical damping.

Isoseismal: Referring to a line on a map bounding points of equal intensity for a particular earthquake.

Kinematic: Referring to the general movement patterns and directions of the Earth's rocks that produce rock deformation.

Landslide: The down slope movement of soil and (or) rock

Lateral-Force-Resisting System: is that part of the structural system designed to resist the Design Seismic Forces (or wind forces).

Lateral Load: is any horizontal load on a structure, including the load from wind or earthquake.

Limit State: is a condition in which a structure, or component, is judged to be no longer useful for its intended function (serviceability limit state) or judged to be unsafe (strength limit state).

Live Loads: are those loads produced by the use and occupancy of the building (or other structure). Live loads do not include dead load, construction load, or environmental loads (i.e. wind, snow, rain, earthquake, or flood loads).

Lifelines: Structures that are important or critical for urban functionality. Examples are roadways, pipelines, power lines, sewers, communications, and port facilities.

Limit state design: Design and selection of structural elements allowing stress up to a large percentage of the material's yield point stress. The elements are assumed to be acted on by loads which have been increased by "load factors" which are, in effect, safety factors. A load factor is chosen on the basis of the probable range of variation of a particular loading condition. Essentially, the load factor is an artifact that is intended to provide a more rational assessment of probability and that also allows the use of stresses higher than the ordinary working values.

Liquefaction: Process by which water-saturated sediment temporarily loses strength and acts as a fluid. This effect can be caused by earthquake shaking.

Lithology: The description of rock composition and texture.

Lithosphere: The outer solid part of the Earth, including the crust and uppermost mantle. The lithosphere is about 100 km thick, although its thickness is age dependent. The lithosphere below the crust is brittle enough at some locations to produce earthquakes by faulting, such as within a subducted oceanic plate.

Load and Resistance Factor Design (LRFD): is a method of proportioning structural elements using load and resistance factors such that no applicable limit state is reached when the structure is subjected to all appropriate load combinations. The term "LRFD" is used in the design of *steel* and *wood* structures.

example: (load) factored moment ≤ (resistance) factored moment strength

$$M_u \leq \phi M_n$$

Locked fault: A fault that is not slipping because frictional resistance on the fault is greater than the shear stress across the fault. Such faults may store strain for extended periods that is eventually released in an earthquake when frictional resistance is overcome. A locked fault condition contrasts with fault-creep conditions and an unlocked fault. See Interplate coupling.

Love wave: A type of seismic surface wave having a horizontal motion that is transverse to the direction of propagation.

Magnitude: A number that characterizes the relative size of an earthquake. It is a measure of the size of an earthquake related to the total strain energy released. Magnitude is based on measurement of the maximum motion recorded by a seismograph (sometimes for earthquake waves of a particular frequency), corrected for attenuation to a standardized distance. Several scales have been defined, but the most commonly used are:

1. Body wave magnitude (M_b). The m_b magnitude is measured as the common logarithm displacement amplitude in microns of the P-wave with period near one second. Developed to measure the magnitude of deep focus earthquakes, which do not ordinarily set up detectable surface waves with long periods. Magnitudes can be assigned from any suitable instrument whose constants are known. The body waves can be measured from either the first few cycles of the compression waves (M_b) or the 1-second period shear waves (M_{blg})

2. Local magnitude (M_L). The original magnitude definition by Richter. The magnitude of an earthquake measured as the common logarithm of the displacement amplitude, in microns, defined by a standard Wood-Anderson seismograph located on turn ground 100 km from the epicenter and having a magnification of 2800,a natural period of 0.8 second, and a damping coefficient of 80. The definition itself applies strictly only to earthquakes having focal depths smaller than about 30 km. Empirical charts and tables are available to correct to an epicentral distance of 100 km for other types of seismographs and for various conditions of

the ground. The correction charts are suitable up to epicentral distances of about 600 km. The correction charts are site dependent and have to be developed for each recording site.

3. Surface wave magnitude (Ms). This magnitude is measured as the common logarithm of the resultant of the maximum mutually perpendicular horizontal displacement amplitudes, in microns, of the 20-second period surface waves. The scale was developed to measure the magnitude of shallow focus earthquakes at relatively long distances. Magnitudes can be assigned from any suitable instrument whose constants are known.

4. Richter Magnitude (M). Richter magnitude is a general usage that is usually M_L up to 5.9, M_s for 5.9 to about 8.0, and M_w, up to 8.3.

Mantle: That part of the Earth's interior between the metallic core and the crust.

Moment magnitude: See Magnitude.

Moment Resisting Frame System (MRF): A structural system with an essentially complete space frame providing support for vertical loads. Seismic force resistance is provided by ordinary or special moment frames capable of resisting the total prescribed forces.

Multiple degrees of freedom (MDOF): The several independent vibration modes in structure under seismic excitation that sum to contribute to total displacements.

Natural frequency(ies): The discrete frequency(ies) at which a particular elastic system vibrates when it is set in motion by a single impulse and not influenced by other external forces or by damping. The reciprocal of fundamental period.

Non-Bearing Wall: is a wall that supports essentially no vertical load other than its own weight.

Non-linear response: Occurs when a structure is deformed beyond the elastic yield point and permanent displacements occur.

Normal stress: That stress component perpendicular to a given plane.

Orthogonal Effects: are the earthquake load effects on structural elements common to the lateral-force-resisting systems along two orthogonal axes.

Oceanic spreading ridge: A fracture zone along the ocean bottom that accommodates upwelling of mantle material to the surface, thus creating new crust. This fracture is topographically marked by a line of ridges that form as molten rock reaches the ocean bottom and solidifies.

Oceanic trench: A linear depression of the sea floor caused by and approximately coincident with a subduction thrust fault.

Ordinary moment frame: A space frame in which members and joints are capable of resisting forces by flexure as well as by direct stress along the axis of the members.

Oscillator: A mass that moves with oscillating motion under the influence of external forces and one or more forces that restore the mass to its stable at-rest position. In earthquake engineering, an oscillator is an idealized damped mass-spring system used as a model of the response of a structure to earthquake ground motion. A seismograph is also an oscillator of this type.

Overstrength: is a characteristic of structures where the actual strength is larger than the design strength. The degree of overstrength is material and system-dependent.

P- wave: A seismic body wave that involves particle motion (alternating compression and extension) in the direction of propagation.

P-Δ effect: The secondary effect on shears and moments of frame members due to the action of the vertical loads at eccentricities created by displacement of the building frame by seismic forces.

Particle acceleration: The time rate of change of particle velocity during earthquake shaking.

Particle displacement: The difference between the initial position of a particle and any later temporary position during earthquake shaking.

Particle velocity: The time rate of change of particle displacement during earthquake shaking.

Peak acceleration: See Acceleration.

Period: The time interval required for one full cycle of a wave.

Pile downdrag: Also known as "negative skin friction." Downward-directed shear stresses on the sides of a foundation- bearing pile caused by settlement of the surrounding ground with respect to the pile. This settlement may result from static surface loading or, in the case of contractive soils, could also be the result of consolidation or slumping from seismic shock.

Piping: In general, this is the movement of fine particles from a soil mass, either externally out of a crack or boil at a free surface or internally within a layered soil mass from a fine-grained base, into the voids of an adjacent coarse material. It is produced by a relatively steep hydraulic gradient and by an abrupt difference between the particle sizes of the fine base and the void sizes of the adjacent coarse material. It is aggravated by a single-size gradation of the base soil which prevents the formation of a natural filter. It can be initiated if seismic shock creates cracks or voids in an earth dam, levee, or slope with seepage. It may also result in sand or silt boils on level ground when internal pore-water pressures are generated by earthquake shaking.

Plate tectonics: A theory supported by a wide range of evidence that considers the Earth's crust and upper mantle to be composed of several large, thin, relatively rigid plates that move relative to one another. Slip on faults that define the plate boundaries commonly results in earthquakes. Several styles of faults bound the plates, including thrust faults along which plate material is subducted or consumed in the mantle, oceanic spreading ridges along which new crustal material is produced, and transform faults that accommodate horizontal slip (strike slip) between adjoining plates.

Predominant period: The period(s) at which maximum spectral energy is concentrated.

Probabilistic earthquake hazard: See Earthquake hazard and Earthquake risk.

Pseudorelative acceleration spectrum: See Response spectrum.

Pseudostatic analysis: An analysis in which horizontal and vertical forces are taken as equivalent to selected horizontal and vertical seismic coefficients multiplied by the weight of a structure or portion of a structure to be analyzed. The intent is to approximate the dynamic effects of earthquake shaking on the structure by using the forces in conventional static analyses.

Random vibration theory: A theoretical probabilistic formulation that links band-limited Gaussian noise spectra, representing the spectra of earthquake ground motions, with corresponding time history peak values.

Reactions: are forces acting on the supports of a structure that hold the structure in equilibrium.

Recurrence interval: The average time span between events (such as large earthquakes, ground shaking exceeding a particular value, or liquefaction) at a particular site. Also termed return period.

Redundant structure: A statically indeterminate structure that requires more than one element to reach its ultimate capacity before the structure becomes unstable.

Reflection: The energy or wave from a seismic source that has been returned (reflected) from an interface between materials of different elastic properties within the Earth.

Reflector: An interface between materials of different elastic properties that reflects seismic waves.

Refraction: (1) The deflection of the ray path of a seismic wave caused by its passage from one material to another having different elastic properties. (2) Bending of a tsunami wave front owing to variations in the water depth along a coastline.

Relaxation theory: Concept wherein radiated seismic waves of an earthquake result when stored strain within the Earth is released at the time of slip along a fault; adjacent fault blocks reach new states of equilibrium.

Residual shear strength: Pertains particularly to heavily over-consolidated clay or clay-shale. It is the effective stress shear strength that remains after intense shear strains or large sliding movement on a narrow failure zone. The shearing generally involves formation of a smoothed or slick failure plane striated in the direction of movement. Also used recently and confusingly to refer to the shear strength of soil in a liquefied state.

Resonance: The condition when the frequency of an applied harmonic force is the same as the natural frequency of a vibrating body. At resonance, the vibrating body will exhibit the maximum amplitude of response displacement.

Response: The motion in a system resulting from shaking under specified conditions.

Response spectrum: A curve showing the mathematically computed maximum values of acceleration, velocity, or displacement experienced by single degree-of-freedom systems spanning a selected range of natural periods when subjected to a given time history of earthquake ground motion. The spectrum of maximum response values is expressed as a function of the natural period of single-degree-of-freedom systems for a given damping.

The response spectrum acceleration, velocity, and displacement values may be calculated from each other as function of the natural period by assuming that the motions are harmonic an undamped. When calculated m this manner these are sometimes referred to as pseudo-acceleration, pseudo-velocity, or pseudo-displacement spectrum values. These curves are used by engineers to estimate the maximum response of simple structures to complex ground motions. For example, the 5-percent spectral acceleration at 1 second is the maximum acceleration of the top of a structure with 5 percent damping whose natural period of vibration is 1 second when subjected to a given input ground-acceleration record.

Rigidity: See Shear modulus.

Risk: See Earthquake risk.

Root mean square: Square root of the mean square value of a random variable.

Root-mean-square acceleration (A_{rms}): The average resultant acceleration during the strong motions of accelerograms recording motion in two orthogonal directions. It is the square root of the sum of the square of the accelerations in the two directions. The A_{rms} can be calculated in the time or frequency domain.

S- wave: A seismic body wave that involves a shearing motion in a direction perpendicular to the direction of propagation. When it is resolved into two orthogonal components in the plane perpendicular to the direction of propagation, SH denotes the horizontal component and SV denotes the orthogonal component.

Sand boil: Sand and water ejected to the ground surface as a result of liquefaction at shallow depth; the conical sediment deposit that remains as evidence of liquefaction.

Saturation: The point where ground acceleration, velocity, and displacement reach upper-limit values determined by local properties of earth and rock density, strength, and stiffness. The values will not increase even though the earthquake energy release increases.

Scaling: An adjustment to a given earthquake time history or response spectrum where the amplitude of acceleration, velocity or displacement is increased or decreased, usually without change to the frequency content of the earthquake record, to model the effects of earthquakes of greater or lesser magnitude than the prototype event.

Segmentation: The breaking of faults along their lengths by other faults that cross them oe their limitations in length by other factors such as topography or bends in strikes of the faults. Segmentation can limit the length of faulting in a single earthquake to some fraction of total fault length, thus also limiting the size of the earthquake.

Seismic hazard: See Earthquake hazard and Earthquake risk.

Seismic load: The load on a structure resulting from the application of a prescribed time history or response spectrum to a structure, or the equivalent static base shear used for the design of a structure.

Seismic moment: A measure of the size of an earthquake based on the area of fault rupture, the average amount of slip, and the shear modulus of the rocks offset by faulting. Seismic moment can also be calculated from the amplitude spectra of seismic waves.

Seismic resisting system: That part of a structural system that is considered to provide the required resistance to seismic forces in design analyses.

Seismic risk: The probability of social or economic consequences of an earthquake. See Probabilistic earthquake hazard.

Seismic wave: An elastic wave generated by an impulse such as an earthquake or an explosion. Seismic waves may propagate either along or near the Earth's surface (for example, Rayleigh and Love waves) or through the Earth's interior (P and S waves).

Seismic zone: A geographic area characterized by a combination of geology and/or seismic history in which a given earthquake may occur anywhere.

Seismicity: The geographic and historical distribution of earthquakes.

Seismogenic: Capable of generating earthquakes.

Seismogram: A record written by a seismograph in response to ground motions produced by an earthquake, explosion, or other ground-motion sources.

***Seismometer or seismograph**:* A seismometer is a damped oscillating mass, such as a damped mass-spring system, used to detect seismic-wave energy. The motion of the mass is commonly transformed into an electrical voltage. The electrical voltage is recorded on paper, magnetic tape, or another recording medium. This record is proportional to the motion of the seismometer mass relative to the Earth, but it can be mathematically converted to a record of the absolute motion of the ground. Seismograph is a term that refers to the seismometer and its recording device as a single unit.

***Separation**:* The distance between any two parts of a reference plane (for example, a sedimentary bed or a geomorphic surface) offset by a fault, measured in any plane. Separation is the apparent amount of fault displacement and is nearly always less than the actual slip.

Shear: is a Stress that tends to make two members, or parts of a member, slide past each other.

Shear Diagram: is a graphical representation of the value of the shear force at any point along a structural members length.

***Shear modulus**:* The ratio of shear stress to shear strain of a material during simple shear.

***Shear stress**:* The stress component parallel to a given surface, such as a fault plane, that results from forces applied parallel to the surface or from remote forces transmitted through the surrounding rock.

Shear wall: A structural wall used for lateral stiffening to reduce inter-story displacement in a structure during seismic shaking.

Shear wave: See S wave.

Shear wave velocity: The velocity of propagation of shear wave vibrations in a material. In earth and rock, it can be measured in situ using standard geophysical procedures. The velocity is equal to the square root of the shear modulus of the material divided by its mass density.

Shear Wall: is a wall designed to resist lateral forces, parallel to the plane of the wall, caused by wind or earthquake forces. A shear wall is sometimes referred to as a vertical diaphragm or structural wall.

Shear Wall-Frame Interactive System: uses combinations of shear walls and frames designed to resist lateral forces in proportion to their relative rigidities, considering interaction between the shear walls and the frames on all levels.

Single-degree-of-freedom (SDOF) system: A structure that responds to seismic excitation in only one vibration mode.

Slip: The relative displacement of formerly adjacent points on opposite sides of a fault, measured on the fault surface.

Slip rate: The average rate of displacement at a point along a fault as determined from geodetic measurements, from offset man-made structures, or from offset geologic features whose age can be estimated. It is measured parallel to the predominant slip direction or estimated from the vertical or horizontal separation of geologic markers.

Slip zone: This usually refers to a narrow and well-delineated zone of failure in a soil mass where large shear strains occur. It may coincide with a geologic contact zone or discontinuity, and frequently forms the base of a large sliding mass.

Source: (1) The geologic structure that generates a particular earthquake. (2) The explosion used to generate acoustic or seismic waves.

Space Frame: is a three-dimensional structural system, without bearing walls, composed of members interconnected so as to function as a complete self-contained unit with or without the aid of horizontal diaphragms or floor-bracing systems.

Story: is the space between levels. Story x is the story below Level x.

Story Drift (Δ_x): is the lateral displacement of one level to the level below (or above).

Story Drift Ratio (Δ_x / h_s): is the story drift divided by the story height.

Story Shear: (V_x) is the summation of design lateral forces above the story under consideration.

Strength: is the capacity of an element or member to resist factored load as specified in Chapters 16, 18, 19, 21, 22 & 23 of 2006 IBC.

Strength Design (SD): is a method of proportioning structural elements such that the computed forces produced in the elements by the factored load combinations do not exceed the factored element strength. The term "strength design" is used in the design of concrete and masonry structures.

example: (load) factored moment ≤ (strength-reduction) factored moment strength

$$M_u \leq \phi M_n$$

Structure: is an assemblage of framing members designed to support gravity loads and resist lateral forces. Structures may be categorized as building structures or nonbuilding structures.

Stud Wall: is wall consisting of small, closely spaced members, called studs, and usually sheathed on both faces with a wall material.

Subdiaphragm: is a portion of a larger wood diaphragm designed to anchor and transfer local forces to primary diaphragm struts, and the main diaphragm.

Space frame. A structural system composed of interconnected members other than bearing walls that is capable of supporting vertical loads and that will also provide resistance to horizontal seismic forces.

Special lateral reinforcement: Spirals, closed stirrups or hoops, and supplementary cross-ties provided to restrain concrete. They qualify the portion of reinforced concrete where they are used as a confined region that will continue to support compressive stresses even if the confined concrete has fractured.

Special moment frame: A space frame in which members and joints are capable of resisting forces by flexure as well as by direct stress along the axis of the members.

Spectral acceleration: Commonly refers to either the Fourier amplitude spectrum of ground acceleration or the PSRV.

Spectral amplification: A measure of the relative shaking response of different geologic materials. The ratio of the Fourier amplitude spectrum of a seismogram recorded on one material to that computed from a seismogram recorded on another material for the same earthquake or explosion.

Spectrum: In seismology, a curve showing amplitude and phase as a function of frequency or period.

Standard deviation: The square root of the average of the squares of deviations about the mean of a set of data. Standard deviation is a statistical measure of spread or variability.

Standard penetration resistance: A measure of relative density expressed by the number of blows (blow count) needed to push a probe a standard distance into sediment. The standard penetration test determines the number of blows required to drive a standard sampling spoon 1 ft into the sediment by repeatedly dropping a 140-pound weight from a height of 30 in.

Statically indeterminate structure: Structure for which the reaction components and internal stresses cannot be completely determined by application of the three condition equations for static equilibrium.

Stick slip: The rapid displacement that occurs between two sides of a fault when the shear stress on the fault exceeds the frictional stress. Stick-slip displacement on a fault radiates energy in the form of seismic waves, creating an earthquake.

Strain: Small changes in length and volume associated with deformation of the Earth by tectonic stresses or by the passage of seismic waves.

Strain rate: Strain measurements are computed from observed changes in length on the Earth's surface, commonly along multiple paths. Because the changes in length are observed over varying time periods and path lengths, they are expressed as the change in length divided by the measurement distance divided by the measurement time period. This number, which is expressed as the change in length per unit length per unit time, is termed the strain rate. These measurements are used to infer the directions of principal strain and stress rates near the

Earth's surface.

Stress: Force per unit area acting on a plane within a body. Six values are required to characterize completely the stress at a point: three normal components and three shear components.

Stress drop: The difference between the stress across a fault before and after an earthquake. A parameter in many models of the earthquake source that has a bearing on the level of high-frequency shaking that the fault radiates. Commonly stated in units termed bars or megapascals (1 bar equals 1 kg/cm2, and 1 megapascal equals 10 bars).

Strike: Trend or bearing, relative to north, of the line defined by the intersection of a planar geologic surface (for example, a fault or a bed) and a horizontal surface.

Strike slip: See Fault.

Strong motion: Ground motion of sufficient amplitude and duration to be potentially damaging to a building's structural components or architectural features.

Structural system factor (R): Factor appearing in the denominator of the typical building code base shear formula that reflects the damping and ductility in the structure framing system at displacements great enough to exceed initial yield. This factor ranges upward from a value of 1 for a lightly damped building frame of brittle structural material to as much as 8.5 for a special moment-resisting space frame.

Structure importance factor (I): Multiplier present in the usual building code base shear formula which reflects the societal importance of the building and its role in post-earthquake recovery. It increases a structure's safety by increasing the level of the equivalent lateral force to be applied in its design.

Structure seismic coefficient (C_s): Factor in the usual building code base formula that reflects the structure's natural frequency of vibration, which determines the structure's response and degree of resonance with the frequency and energy distribution of the ground-based acceleration.

Subduction: A plate tectonics term for the process whereby the oceanic lithosphere collides with and descends beneath the continental lithosphere.

Subduction zone: A zone between a sinking plate and an overriding plate.

Surface faulting: Displacement that reaches the Earth's surface during slip along a fault.Commonly accompanies moderate and large earthquakes having focal depths less than 20 km. Surface faulting also may accompany a seismic tectonic creep or natural or man-induced subsidence.

Surface wave: Seismic wave that propagates along the Earth's surface. Love and Rayleigh waves are the most common.

Tension: a stress which tends to lengthen a member (stretch it).

Tectonic: Refers to rock-deforming processes and resulting structures that occur over regional sections of the lithosphere.

Time history: The sequence of values of any time - varying quantity (such as a ground motion measurement) measured at a set of fixed times. Also termed time series.

Truss: is a jointed structure designed to support vertical or horizontal loads and composed generally of straight members forming a number of triangles.

Tsunami: An impulsively generated sea wave of local or distant origin that results from large-scale seafloor displacements associated with large earthquakes, major submarine slides, or exploding volcanic islands.

Tsunami magnitude (Mt): A number used to compare sizes of tsunamis generated by different earthquakes and calculated from the logarithm of the maximum amplitude of the tsunami wave measured by a tide gauge distant from the tsunami source.

Velocity: In reference to earthquake shaking, velocity is the time rate of change of ground displacement of a reference point during the passage of earthquake seismic waves commonly expressed in centimeters per second.

Velocity structure: A generalized regional model of the Earth's crust that represents crustal structure using layers having different assumed seismic velocities.

Vertical Diaphragm: See Shear Wall.

Vertical Load-Carrying Frame: is a space frame designed to carry vertical gravity loads.

Water table: The upper surface of a body of unconfined ground water at which the water pressure is equal to the atmospheric pressure.

Wavelength: The distance between successive points of equal amplitude and phase on a wave (for example, crest to crest or trough to trough).

Weak Story: is one in which the story strength is less than 80 percent of the story above. See ASCE 7-05 Table12.3-2.

Working Stress: is the maximum allowable unit stress used in the design of a structural member.

Working stress design: Design and selection of structural elements assuming that they can be safely loaded up to a "working stress" which is typically 40 to 60 of yield point stress. The structure is assumed to be acted on by loads that represent realistic maximum values. The safety factor then becomes the ratio of yield point stresses to the selected working stresses.

e.g.: calculated bending stress ≤ allowable bending stress

$$\sigma_{actual} \leq \sigma_{allowable}$$

SYMBOLS AND NOTATIONS

A_x = torsional amplification factor (Section 12.8.4.3)

a_i = the acceleration at level i obtained from a modal analysis (Section 13.3.1)

a_p = the amplification factor related to the response of a system or component as affected by the type of seismic attachment, determined in Section 13.3.1

b_p = the width of the rectangular glass panel

C_d = deflection amplification factor as given in Tables 12.2-1, 15.4-1, or 15.4-2

C_s = seismic response coefficient determined in Section 12.8.1.1 and 19.3.1 (dimensionless)

C_T = building period coefficient in Section 12.8.2.1

C_{vx} = vertical distribution factor as determined in Section 12.8.3

c = distance from the neutral axis of a flexural member to the fiber of maximum compressive strain (in. or mm)

D = the effect of dead load 112 ASCE 7-05

D_s = the total depth of stratum in Eq. 19.2-12 (ft or m)

d_C = The total thickness of cohesive soil layers in the top 100 ft (30 m); see Section 20.4.3 (ft or m)

d_i = The thickness of any soil or rock layer i (between 0 and 100 ft [30 m]); see Section 20.4.1 (ft or m)

d_S = The total thickness of cohesionless soil layers in the top 100 ft (30 m); see Section 20.4.2 (ft or m)

E = effect of horizontal and vertical earthquake induced forces (Section 12.4)

F_a = short-period site coefficient (at 0.2 s-period); see Section 11.4.3

F_i, F_n, F_x = portion of the seismic base shear, V, induced at Level i, n, or x, respectively, as determined in Section 12.8.3

F_p = the seismic force acting on a component of a structure as determined in Section 13.3.1

F_v = long-period site coefficient (at 1.0 s-period); see Section 11.4.3

f'_c = specified compressive strength of concrete used in design

f'_s = ultimate tensile strength (psi or MPa) of the bolt, stud, or insert leg wires. For A307 bolts or A108 studs, it is permitted to be assumed to be 60,000 psi (415 MPa).

f_y = specified yield strength of reinforcement (psi or MPa)

f_{yh} = specified yield strength of the special lateral reinforcement (psi or kPa)

g = acceleration due to gravity

H = thickness of soil

h = height of a shear wall measured as the maximum clear height from top of foundation to bottom of diaphragm framing above, or the maximum clear height from top of diaphragm to bottom of diaphragm framing above

h = average roof height of structure with respect to the base; see Chapter 13

h_i , h_n, h_x = the height above the base to Level i , n, or x, respectively

h_{sx} = the story height below Level x = $(h_x - h_{x-1})$

I = the importance factor in Section 11.5.1

I_p = the component importance factor as prescribed in Section 13.3.1

i = the building level referred to by the subscript i ; $i = 1$ designates the first level above the base

K_p = the stiffness of the component or attachment, Section 13.6.2

KL/r = the lateral slenderness ratio of a compression member measured in terms of its effective length, KL, and the least radius of gyration of the member cross section, r

k = distribution exponent given in Section 12.8.3

L = overall length of the building (ft or m) at the base in the direction being analyzed

M_t = torsional moment resulting from eccentricity between the locations of center of mass and the center of rigidity (Section 12.8.4.1)

M_{ta} = accidental torsional moment as determined in Section 12.8.4.2

N = standard penetration resistance, ASTM 1586

N = number of stories (Section 12.8.2.1)

$\overline{N}$ = Average field standard penetration resistance for he top 100 ft (30 m); see Sections 20.3.3 and 0.4.2

$\overline{N}_{ch}$ = average standard penetration resistance for cohesionless soil layers for the top 100 ft (30 m); see Sections 20.3.3 and 20.4.2

N_i = Standard penetration resistance of any soil or rock layer i (between 0 and 100 ft [30 m]); see Section 20.4.2

n = designation for the level that is uppermost in the main portion of the building

P_x = total unfactored vertical design load at and above Level x, for use in Section 12.8.7

PI = plasticity index, ASTM D4318

Q_E = effect of horizontal seismic (earthquake-induced) forces

R = response modification coefficient as given in Tables 12.2-1, 12.14-1, 15.4-1, or 15.4-2 3

R_p = component response modification factor as defined in Section 13.3.1

S_S = mapped MCE, 5 percent damped, spectral response acceleration parameter at short periods as defined in Section 11.4.1

S_1 = mapped MCE, 5 percent damped, spectral response acceleration parameter at a period of 1 s as defined in Section 11.4.1

S_{aM} = the site-specific MCE spectral response acceleration at any period

S_{DS} = design, 5 percent damped, spectral response acceleration parameter at short periods as defined in Section 11.4.4

S_{D1} = design, 5 percent damped, spectral response acceleration parameter at a period of 1 s as defined in Section 11.4.4

S_{MS} = the MCE, 5 percent damped, spectral response acceleration at short periods adjusted for site class effects as defined in Section 11.4.3

S_{M1} = the MCE, 5 percent damped, spectral response acceleration at a period of 1 s adjusted for site class effects as defined in Section 11.4.3

s_u = undrained shear strength; see Section 20.4.3

$\bar{S}_u$ = average undrained shear strength in top 100 ft (30m); see Sections 20.3.3 and 20.4.3, ASTM D2166 or ASTM D2850

s_{ui} = undrained shear strength of any cohesive soil layer i (between 0 and 100 ft [30 m]); see Section 20.4.3

T = the fundamental period of the building

T_a = approximate fundamental period of the building as determined in Section 12.8.2

T_L = long-period transition period as defined in Section 11.4.5

T_p = fundamental period of the component and its attachment, Section 13.6.2

T_0 = $0.2S_{D1}/S_{DS}$

T_S = S_{D1}/S_{DS}

V = total design lateral force or shear at the base

V_t = design value of the seismic base shear as determined in Section 12.9.4

V_x = seismic design shear in story x as determined in Section 12.8.4 or 12.9.4

v_s = shear wave velocity at small shear strains (equal to 10-3 percent strain or less); see Section 20.4.1 (ft/s or m/s)

v_s = average shearwave velocity at small shear strains in top 100 ft (30 m); see Sections 20.3.3 and 20.4.1

v_{so} = average shear wave velocity for the soils beneath the foundation at small strain levels, Section 19.2.1.1 (ft/s or m/s)

W = effective seismic weight of the building as defined in Section 12.7.2. For calculation of seismic-isolated building period, W is the total effective seismic weight of the building as defined in Sections 19.2 and 19.3 (kip or kN)

$\overline{W}$ = effective seismic weight of the building as defined in Sections 19.2 and 19.3 (kip or kN)

W_c = gravity load of a component of the building

W_p = component operating weight (lb or N)

w_i, w_n, w_x = portion of W that is located at or assigned to Level i , n, or x, respectively

x = level under consideration, 1 designates the first level above the base

z = height in structure of point of attachment of component with respect to the base; see Section 13.3.1

β = ratio of shear demand to shear capacity for the story between Level x and x - 1

γ = average unit weight of soil (lb/ft3 or N/m3)

Δ = design story drift as determined in Section 12.8.6

$\Delta_{fallout}$ = the relative seismic displacement (drift) at which glass fallout from the curtain wall, storefront, or partition occurs

Δ = allowable story drift as specified in Section 12.12.1

δ_{max} = maximum displacement at Level x, considering torsion, Section 12.8.4.3

δ_{avg} = the average of the displacements at the extreme points of the structure at Level x, Section 12.8.4.3

δ_x = deflection of Level x at the center of the mass at and above Level x, Eq. 12.8-15

δ_{xe} = deflection of Level x at the center of the mass at and above Level x determined by an elastic analysis, Section 12.8-6

θ = stability coefficient for P-delta effects as determined in Section 12.8.7

ρ = a redundancy factor based on the extent of structural redundancy present in a building as defined in Section 12.3.4

Ω_0 = overstrength factor as defined in Tables 12.2-1, 5.4-1, and 15.3-1

SUMMARY BY TYPE

1-Building Structures

A-Equivalent Lateral Force Procedure (ELF) (§ 12.8)

	ASCE 7-05 Equations	Comments
Seismic Response Coefficient (C_S) & Base Shear (V)	$V = C_S W$ (12.8 – 1)	▪ Equation (12.8-1) gives the seismic design base shear (V) at the Strength Design (SD/LRFD) force level. C_s is the Seismic Response Coefficient and W is the Effective Seismic Weight.
	$C_S = \frac{S_{DS}}{\left(R/I\right)}$ (12.8 – 2)	▪ Equation (12.8-2) gives the ***maximum*** seismic response coefficient for ***short period*** structures when $T_0 = 0.2T_s \le T < T_S = S_{D1}/S_{DS}$. It governs for <u>low rise structures</u> ($\approx$3-4 stories). ***No need to check the minimum values of C_s per Equations 12.8-5 & 12.8-6.***
	$C_S = \frac{S_{D1}}{T\left(R/I\right)}$ (12.8 – 3)	▪ Equation (12.8-3) gives the seismic response coefficient when $T_S < T \le T_L$. It governs for <u>intermediate structures > 4</u> stories. T_L is the long-period transition period given by Figure 22-15 (ASCE7).
	$C_S = \frac{S_{D1} T_L}{T^2\left(R/I\right)}$ (12.8 – 4)	▪ Equation (12.8-4) gives the seismic response coefficient when $T > T_L$. It governs for <u>very long period structures</u>. T_L is the long-period transition period given by Figure 22-15 of ASCE 7-05.
	$(C_S)_{\min} \ge 0.01$ (12.8 – 5)	▪ Equation (12.8-5) gives the ***minimum*** seismic response coefficient that may govern for very long-period structures when $T > T_L$.
	$(C_S)_{\min} \ge \frac{0.5 S_1}{\left(R/I\right)}$ $(if\ S_1 \ge 0.6g)$ (12.8 – 6)	▪ Equation (12.8-6) gives the ***minimum*** seismic response coefficient that may govern for very long period structures when $T > T_L$.

Distribution of V (if V calculated based on

$$F_x = C_{vx} V \qquad (12.8-11)$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad (12.8-12)$$

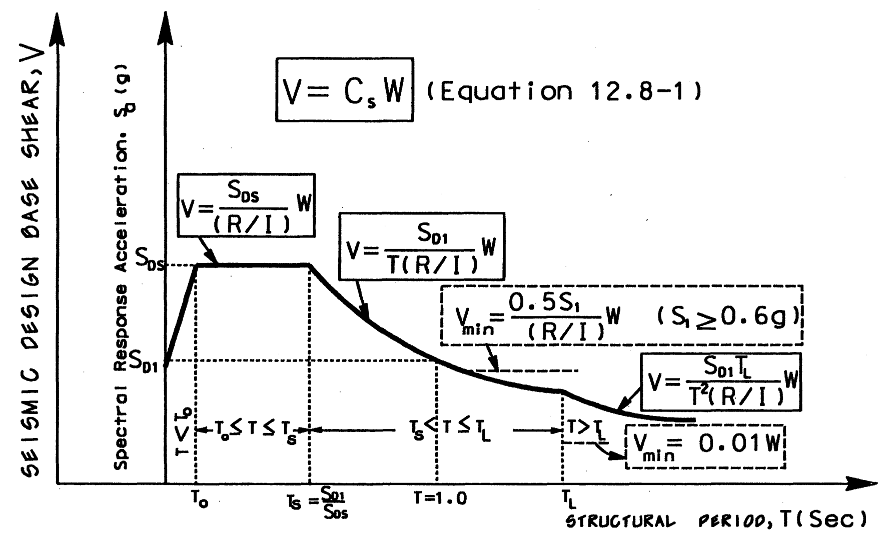
$$C_{vx} = \frac{w_x h_x}{\sum_{i=1}^{n} w_i h_i} \; \{linear\ Dist.\}\ if\ T \le 0.5\,\text{sec}$$

$$C_{vx} = \frac{w_x h_x^2}{\sum_{i=1}^{n} w_i h_i^2} \; \{Parabolic\ Dist.\}\ if\ T \ge 2.5\,\text{sec}$$

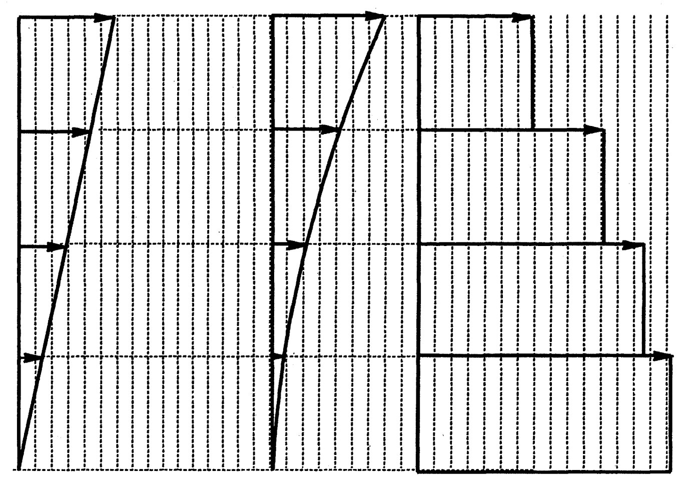
- Equation (12.8-11) distributes the base shear V among levels i.e. calculates the portion of V that will be acting at the center of mass of each level x.
- Equation (12.8-12) calculates the Vertical Distribution Factor (C_{vx}). k = an exponent related to the structure period as follows: for structures having a period of 0.5 s or less, k = 1 for structures having a period of 2.5s or more, k = 2 for structures having a period between 0.5 and 2.5 s, k shall be 2 or shall be determined by linear interpolation between 1 and 2

Values of the exponent "k"

T	$\le 0.5\,\text{sec}$	$0.5\,\text{sec} < T < 2.5\,\text{sec}$	$\ge 2.5\,\text{sec}$
k	1.0	2.0 OR Linear Interpolation between 1 & 2	2.0



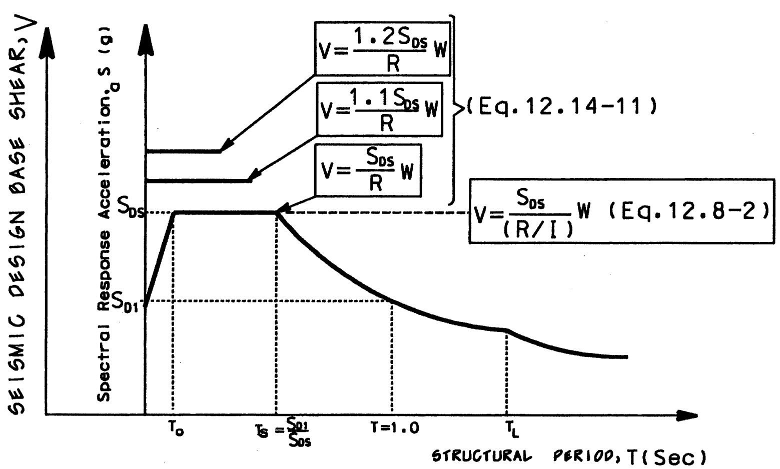
DESIGN RESPONSE SPECTRUM FOR BUILDING STRUCTURES



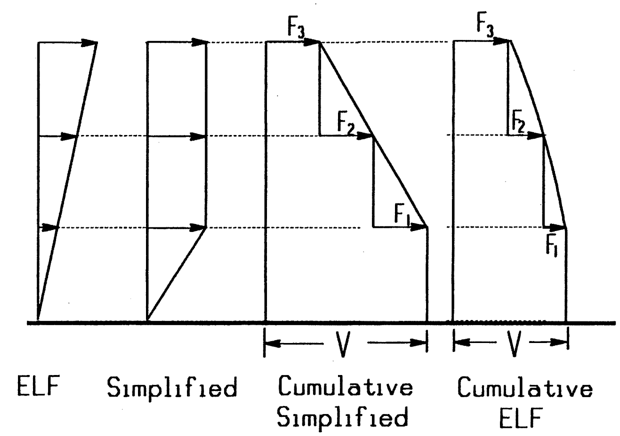
linear (k=1) Parabolic (k=2) Cumulative

B-Simplified Design Base Shear (§ 12.14)

	ASCE 7-05 Equations	Comments
Base Shear (V)	$V = \frac{FS_{DS}}{R} W \qquad (12.14-11)$ $V = \frac{S_{DS}}{R} W \quad \Rightarrow for\ 1-story$ $V = \frac{1.1 S_{DS}}{R} W \quad \Rightarrow for\ 2-story$ $V = \frac{1.2 S_{DS}}{R} W \quad \Rightarrow for\ 3-story$	▪ The simplified procedure may be used for the following structures of Occupancy Category I or II (I =1): 1- Bearing Wall and Building Frame Systems (No Moment Frame) ≤ 3 stories using light-frame construction (wood frame), or
Vertical Distribution	$F_x = \frac{w_x}{W} V \qquad (12.14-12)$ $= \frac{w_x}{W}\left(\frac{F S_{DS}}{R} W\right) = \left(\frac{F S_{DS}}{R}\right) w_x$	▪ Equation (12.14-12) calculates the story shear among levels i.e. distribution of the base shear vertically. ▪ Equation (12.8-11) can not be used in lieu of Equation (12.14-12).



SIMPLIFIED DESIGN RESPONSE SPECTRUM FOR BUILDING STRUCTURES



DISTRIBUTION & CUMULATIVE FOR ELF AND SIMPLIFIED

2- Nonbuilding Structures (Chapter 15 of ASCE 7-05)

Rigid Nonbuilding Structures $T < 0.06$ sec	Nonbuilding Structures **Similar** to Buildings $T \geq 0.06$ sec	Nonbuilding Structures **NOT Similar** to Buildings $T \geq 0.06$ sec
$V = 0.30 S_{DS} WI \quad (15.4-5)$	$V = C_S W \quad (12.8-1)$ $C_S = \dfrac{S_{DS}}{\left(R/I\right)} \quad (12.8-2)$ $C_S = \dfrac{S_{D1}}{T\left(R/I\right)} \quad (12.8-3)$ $C_S = \dfrac{S_{D1} T_L}{T^2\left(R/I\right)} \quad (12.8-4)$ $(C_s)_{\min} \geq 0.01 \quad (12.8\text{ - }5)$ $(C_s)_{\min} \geq \dfrac{0.5 S_1}{(R/I)} \quad (if\ S_1 \geq 0.6g) \quad (12.8-6)$ ***NOTE:*** R, Ω_o and C_d shall be taken from Table 15.4-1	$V = C_S W \quad (12.8-1)$ $C_S = \dfrac{S_{DS}}{\left(R/I\right)} \quad (12.8-2)$ $C_S = \dfrac{S_{D1}}{T\left(R/I\right)} \quad (12.8-3)$ $C_S = \dfrac{S_{D1} T_L}{T^2\left(R/I\right)} \quad (12.8-4)$ $(C_s)_{\min} \geq 0.03 \quad (15.4\text{-}1)$ $\Uparrow$ *New Minimum* $(C_s)_{\min} \geq \dfrac{0.8 S_1}{(R/I)} \quad (if\ S_1 \geq 0.6g) \quad (15.4-2)$ $\Uparrow$ *New Minimum* ***NOTE:*** R, Ω_o and C_d shall be taken from Table 15.4-2

3- Seismic Design for Nonstructural Components (Chapter 13-ASCE7-05)

ASCE 7-05

$$F_P = \frac{0.4 a_p S_{DS} W_p}{\left(R_p / I_p\right)}\left(1 + 2\frac{z}{h}\right) \quad (13.3-1)$$

$$(0.3 S_{DS} I_P W_P)_{\min} \le F_P \le (1.6 S_{DS} I_P W_P)_{\max}$$

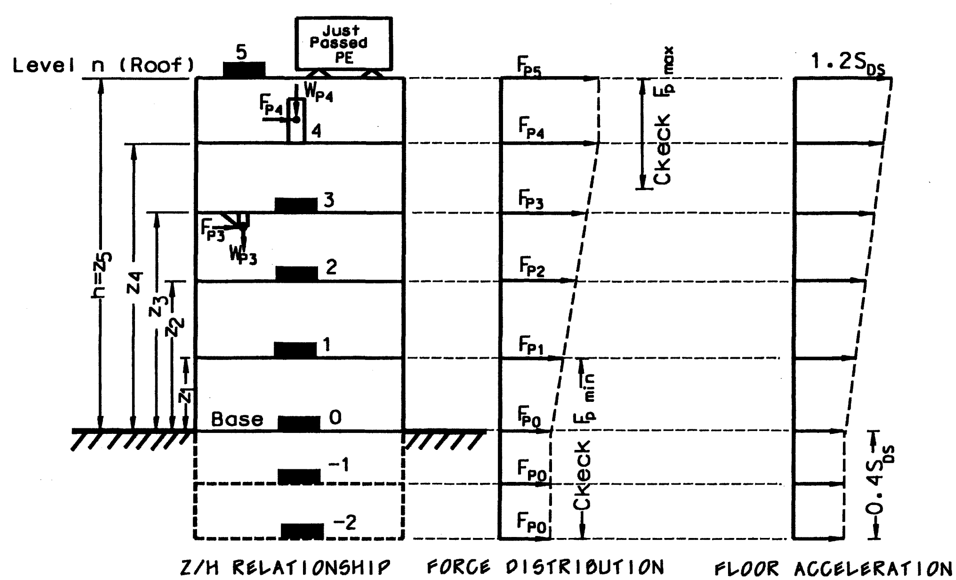
$\uparrow (13.3-3)$ $\qquad\qquad$ $\uparrow (13.3-2)$

- The force (F_p) shall be applied independently in at least two orthogonal horizontal directions in combination with service loads associated with the component, as appropriate.
- For vertically cantilevered systems, the force F_p shall be assumed to act in any horizontal direction. In addition, the component shall be designed for a concurrent vertical force $\pm 0.2\, S_{DS} W_p$

Comments

- a_P, and R_P to be determined from ASCE 7-05 Tables 13.5-1 for Architectural Components and 13.6-1 for Mechanical and Electrical Components
- z = the element of component attachment elevation with respect to grade. It will ***not be taken less than 0.0***
- h = the structure roof elevation with respect to grade.
- The value of $(1+2\, z/h)$ varies between 1.0 when $z = 0.0$ to 3.0 when $z = h$ (i.e. the equipment , component, …etc on the roof of the structure).
- Because all the parameters except W_p of the previous equations are dimentionless, the units of F_p will be the same units used for W_p (kips, k/ft, ksf, … etc)
- The force (F_p) shall be applied at the component's center of gravity and distributed relative to the component's mass distribution.
- The redundancy factor, ρ, is permitted to be taken equal to 1.0 and the overstrength factor, Ω_0, does not apply

Position	z/h
5 (Roof)	1
4	z_4/h
3	z_3/h
2	z_2/h
1	z_1/h
0 (Base)	0
-1	0
-2	0



4- Diaphragms (ASCE 7- 05)

A- Equivalent Lateral Force Procedure

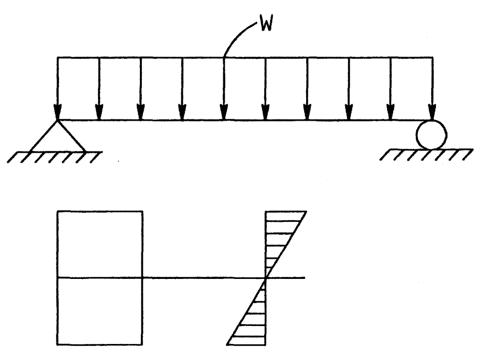
§ 12.10.1.1 (ASCE 7-05)	Comments
$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \qquad (12.10-1)$$ $$(0.2S_{DS}Iw_{px})_{\min} \le F_{px} \le (0.4S_{DS}Iw_{px})_{\max}$$	▪ Equation (12.10-1) gives the design value of the diaphragm force at each level. Note the ***sum from "x to n "NOT from "1 to n". This is not the same for Equation 12.8-12 (from 1 to n)*** ▪ The equation is applicable for both rigid and flexible diaphragm. ▪ For single-story buildings, F_{px} equals the base shear V but the minimum value must be checked

B - Simplified Design Base Shear

§ 12.14.7.4 (ASCE 7-05)	Comments
$$F_{Px} = \underset{\uparrow \text{ §12.14.7.4}}{\frac{w_{px}}{W} V} = \frac{w_{px}}{W}\left(\underset{\uparrow \text{ Equation 12.14-11}}{\frac{FS_{DS}}{R} W}\right) = \left(\frac{FS_{DS}}{R}\right) w_{px}$$	▪ This equation calculates the diaphragm force in case of using the simplified method. ▪ F = 1.0 for 1-story building excluding basement ▪ F = 1.1 for 2-story building excluding basement ▪ F = 1.2 for 3-story building excluding basement ▪ w_{px} = the weight tributary to the diaphragm at Level x ▪ W = effective seismic weight of the structure

SUMMARY OF NON-CODE EQUATIONS

1- Stresses Equations

	Equations	Comments
Bending Stress	$$\sigma = \frac{M\,C}{I} \quad (1)$$ Where: M = Bending Moment (kip-ft, N-m) C = Distance from neutral axis to extreme fibers (in., mm) I = Moment of inertia (2nd moment of area) (in^4, mm^4)	• Equation (1) calculates the bending stress (flexure stress). • The bending stress is linear and the maximum occurs at the outer fibers. • The bending stress is zero at the neutral axis
	$$\sigma = \frac{M}{S} \quad (2)$$ Where: S = Elastic Section Modulus = I / C 	• Equation (2) calculates the bending stress (flexure stress) using the elastic section modulus.

Shear Stresses	Direct	$\tau = \dfrac{P}{A}$ (3) $\tau = \dfrac{P}{2A}$ (4)	• Equation (3) calculates the single shear stress. • Equation (4) calculates the double shear stress.
	Indirect	$\tau = \dfrac{T\,r}{J}$ (5) T = Torque (in-lb, N-m) r = Radius of the shaft or half of the diameter (in, mm) J = Polar moment of inertia = $I_x + I_y$ (in^4, mm^4) $F_{t,i} = \dfrac{(TTM)\ R_i r_i}{J} = \dfrac{(TTM)\ R_i r_i}{\sum R_i r_i^2}$ (6) M_t = Inherent torsional moment due to the actual eccentricity $= V(e_{actual})$ (7) M_{ta} = Torsional moment due to the accidental eccentricity $= V(\pm e_{accidental})$ (8) Total torsional moment = TTM = $M_t + M_{ta}$ $= V(e_{actual} \pm e_{accidental}) = V\,e_{total}$ (9) R_i = Relative or absolute rigidity of wall "i" r_i = The perpendicular distance from wall "i" to the center of rigidity (CR).	• Equation (5) calculates the shear stress due to torsion. This equation has the following assumptions: i) Circular shafts ii) Stress within the elastic limit iii) The material obeys Hooke's law • Equation (6) calculates the shear force in shear walls due to torsional moment generated by seismic forces. • Equation (7) calculates the torsional moment due to the actual eccentricity. • Equation (8) calculates the torsional moment due to the accidental eccentricity. • Equation (9) calculates the torsional moment due to the total eccentricity.

2- Natural Period and Frequency

Equations		Comments
$f=\frac{1}{T}=\frac{1}{2\pi}\sqrt{\frac{Kg}{W}}$	(10)	• Equation (10) shows the relationship between the natural period (T) and the linear natural frequency (f).
$\omega=2\pi f=\frac{2\pi}{T}$	(11)	• Equation (11) calculates angular natural frequency or the circular frequency or angular frequency (ω)
$\omega=\sqrt{\frac{K}{m}}=\sqrt{\frac{Fg}{\Delta_{stat}W}}$	(12)	• Equation (12) shows the relationship between the angular frequency(ω) and the stiffness (K) and the mass (m).
$T=2\pi\sqrt{\frac{W}{gK}}=2\pi\sqrt{\frac{m}{K}}$	(13)	• Equation (13) calculates the natural period of a single degree of freedom system (SDOF) the stiffness (K) and the mass (m) and gravity acceleration (g).
$K_t=K_e=K_1+K_2+\ldots\ldots+K_n$	(14)	• Equation (14) calculates the total stiffness (spring constant) of a system having springs or walls connected in parallel.
$\frac{1}{K_e}=\frac{1}{K_1}+\frac{1}{K_2}\ldots\ldots\ldots+\frac{1}{K_n}$	(15)	• Equation (15) calculates the total stiffness (spring constant) of a system having springs or walls connected in series.

3- Shear Walls and Diaphragms Equations

	Equations	Comments
Shear walls	$\tau_{wall} = \dfrac{F_{total}/2}{L_w} = \dfrac{F_{total}}{2L_w}$ (16)	• Equation (16) calculates the shear stress <u>per unit length</u> in a shear wall. "L_w" is the length of the wall.
	$\tau_{wall} = \dfrac{F_{total}/2}{L_w t} = \dfrac{F_{total}}{2bt}$ (17)	• Equation (17) calculates the shear stress <u>per unit area</u> in a shear wall. "t" is the thickness of the wall.
	$V_{wall\,A} = \dfrac{R_A}{R_A + R_B + \ldots\ldots + R_n} V\,(or\,F)$ (18)	• Equation (18) distributes the shear among the resisting shear walls in proportional to their rigidities (rigid diaphragm).
Diaphragms	$\tau_{diaphragm} = \dfrac{F_{total}/2}{d} = \dfrac{F_{total}}{2d} = \dfrac{F_{diaphragm}}{2d}$ (19)	• Equation (19) calculates the shear stress <u>per unit length</u> in the diaphragm (along the diaphragm). "d" is the dimension parallel to the applied force.
	$C = \dfrac{M}{d} = \dfrac{wL^2}{8d} = \dfrac{F_{diaphragm} L}{8d}$ (20)	• Equation (20) calculates the chord force. "L" is the dimension perpendicular to the applied force
	$Chord\ \ Area = A_{chord} = \dfrac{C}{\sigma_{allowable}}$ (21)	• Equation (21) calculates the area of the chord.
Center of Mass & Rigidity	$X_R = \dfrac{\sum_{i=1}^{n}(R_i x_i)}{\sum_{i=1}^{n} R_i}$ & $Y_R = \dfrac{\sum_{i=1}^{n} R_i y_i}{\sum_{i=1}^{n} R_i}$ (22)	• Equation (22) calculates the coordinates of the center of rigidity for a group of walls. Remember not necessarily that all walls will be part of the two equations (see 7 RULES).
	$X_m = \dfrac{\sum_{i=1}^{n}(m_i x_i)}{\sum_{i=1}^{n} m_i}$ & $Y_m = \dfrac{\sum_{i=1}^{n} m_i y_i}{\sum_{i=1}^{n} m_i}$ (23)	• Equation (23) calculates the coordinates of the center of mass. Remember that all masses will be part of the two equations (see 7 RULES).

Chapter 1

Loads, Design Philosophies and Structural Analysis

Topics to be covered

- Classification of Loads
- Design Philosophies (ASD, SD/LRFD)
- Factor of Safety
- Overturning Moment and Uplift Forces
- Seismic and Wind Loads
- Load Combinations
- Transfer of Vertical and Lateral Loads
- Stresses(Normal, Shear, Bending, Torsion)

19 Example Problems

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Chapter 1- Loads, Design Philosophies and Structural Analysis

1.1 INRODUCTION

Loads are the cornerstone in any analysis or design process. The following is a schematic showing the major five steps that any structure go through:

1- Function(s) and type of the structure.

2-Loads are selected and applied on the structure.

3-Analysis of the structure. V, N, M are determined.

4-Design and selection of members and connections.

5-Construction

1.2 CLASSIFICATION OF LOADS

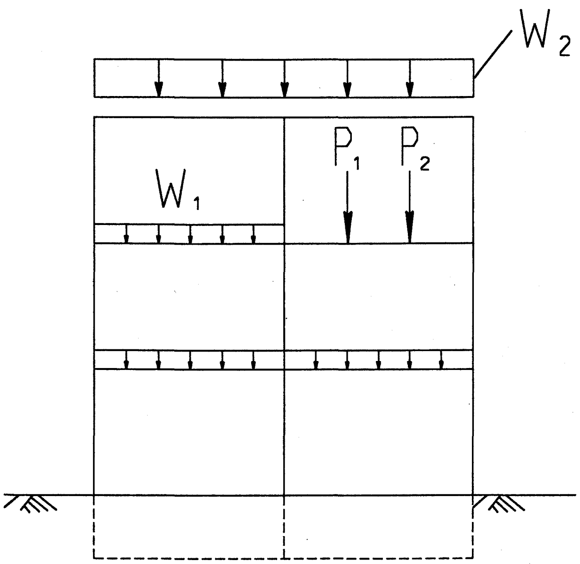
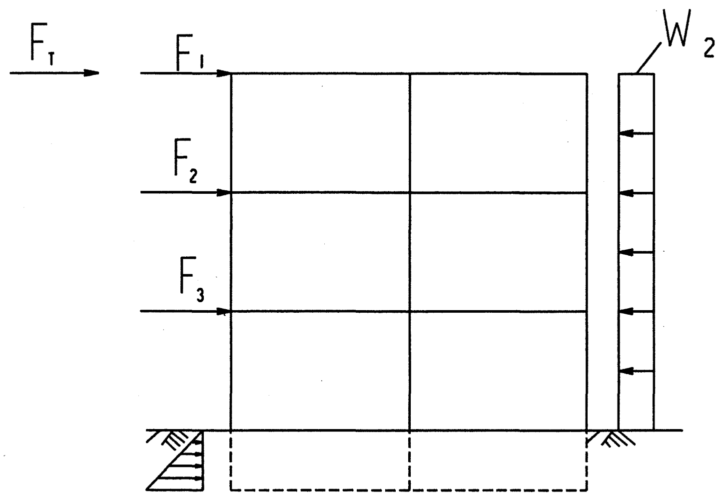
Structural loads usually are classified based on their characteristics and duration as follows:

Table 1-1 Types of Loads

Dead loads	Live loads	Environmental Loads
➤ Constant in magnitude ➤ Remain in one position ➤ Include the weight of the structure and any fixtures that are permanently attached to it.	➤ Change in magnitude ➤ Change position ➤ Include occupancy, warehouse, construction and equipment operating loads.	➤ Change in magnitude ➤ Change position ➤ Include earthquake, rain, snow, wind, temperature loads. Also known as live loads caused by environment.

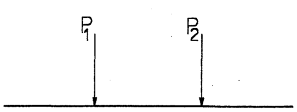
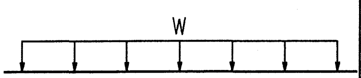
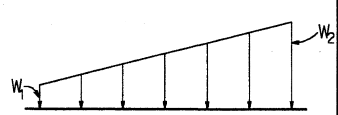
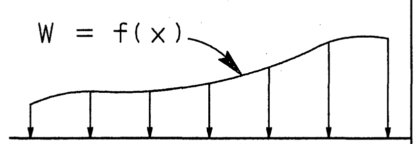
Loads can be also classified according to the direction of application as gravity and lateral loads as follows:

Table 1-2 Types of Loads

Gravity Loads	Lateral Loads
➢ All loads caused by gravity such as dead, live and snow loads.	➢ Loads acting in the lateral direction such as earthquake, wind, earth and fluid lateral pressure.
	

Also, loads could be classified according to the intensity and the area acting over as follows:

Table 1-3 Types of Loads

Concentrated loads	Distributed Loads		
	Uniform	Linear	Nonlinear
➢Loads are assumed to be acting over a point except as indicated in codes.	➢Loads are uniformly distributed over an area or per unit length e.g. lb/ft^2 k/ft^2, lb/ft, N/m	➢Loads vary linearly as shown below	➢ Loads are specified by a function
			

Finally, loads can be classified as service and factor loads:

Table 1-4 Types of Loads

Service Loads	Factored Loads
➢ Used in the Allowable Stress Design (ASD). Also, known as Working Stress Design (WSD) or Service Load Design. ➢ Loads without factors (or a factor of ONE) except "0.7E" and "0.6D" in some load combinations equations.	➢ Used in Strength Stress Design (SD) and Load and Resistance Factor Design (LRFD). Also, known as Factored Load Design. ➢ All loads are multiplied by factors (≥ 1 or ≤ 1) depending on the type of the load. ➢ Load factor for earthquake load is ONE.

1.3 DESIGN PHILOSOPHIES

Essentially there are two design philosophies as follows:

1- Allowable Stress Design (ASD)
2- Ultimate Strength Design (SD) - Concrete and Masonry
Load and Resistance Factor Design (LRFD) - Steel and Wood

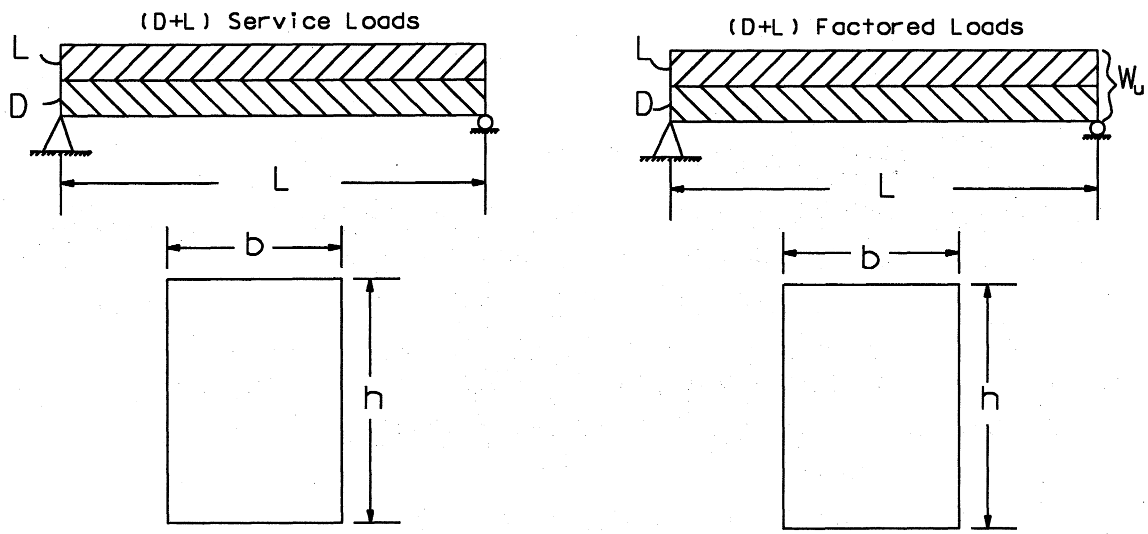


Figure 1-1 Allowable Stress Design (ASD) vs. Strength Design (SD)

$$\sigma_{all} = \frac{M_{\max}}{S}$$

$$S = I/C = \frac{bh^3}{12} \Big/ h/2 = \frac{bh^2}{6}$$

$$Solve\ \ for\ b\ \&\ h$$

$$Assume\ b\ \&\ h$$

$$M_U \leq \phi M_n$$

$$Required \leq Capacity$$

The following table shows the difference between these two design philosophies:

Table 1-5 Allowable Stress Design Vs. Load and Resistance Factor Design

Allowable Stress Design (ASD)	Ultimate Strength Design (SD)/ Load and Resistance Factor Design (LRFD)
➢ Also known as working stress design (WSD) and Service Lad design ➢ Loads used without factors except "E" and "D" in some lad combinations ➢ Method of proportioning structural elements such that the computed stresses produced in the elements by the allowable stress load combinations do not exceed the specified (by Codes) allowable stress $calculated\ stress \le allowable\ stress$ $\sigma_{actual} \le \sigma_{allowable}$ $\tau_{actual} \le \tau_{allowable}$ And $calculated\ deflection \le allowable\ deflection$ $\delta_{actual} \le \delta_{allowable}$	➢ Also known as Factored Load Design. ➢ Loads are multiplied by factors ➢ Method of proportioning structural elements using load and resistance factors such that no applicable limit state is reached when the structure is subjected to all appropriate load combinations. (load) factored moment ≤ (resistance) factored moment strength $M_u \le \phi M_n$ Required Strength ≤ Design Strength Required ≤ Capacity

1.4 FACTOR OF SAFETY

Factor of safety is used in the allowable stress design (ASD) where a maximum load that a structure member or a machine component will be allowed to carry under normal conditions , which is smaller than the ultimate load. This smaller load is called the *allowable load* (also know as *working or design load*). Therefore, only a fraction of the ultimate load capacity of the member is utilized when the allowable load is applied. The remaining portion of the load-carrying capacity is kept in reserve to assure safe performance.

$$\text{Factor of safety (Factor of ignorance)} = \text{F.S.} = \frac{Ultimate\ Load}{Allowable\ Load} = \frac{P_{ult}}{P_{all}} \tag{1-1}$$

$$\text{Factor of safety (Factor of ignorance)} = \text{F.S.} = \frac{Ultimate\ Stress}{Allowable\ Stress} = \frac{\sigma_{ult}}{\sigma_{all}} \tag{1-2}$$

The reasons and the magnitude for using factor of safety in the allowable stress design could be summarized as follows:

1- Variations in the loadings during the life of the structure
2- Variations in the properties of the materials used in construction

3- Approximation and uncertainty in the methods of analysis of structures
4- Uncertainty and variability in the environmental loads as earthquake, wind, snow and rain loads, and
5- The type of failure that may occur (yielding, fracture, buckling, brittle and ductile type failure).

When structures are subjected to lateral loads, the following two factors of safety should be considered:

1- Factor of safety against sliding:

$$\text{Factor of safety} = \frac{\textit{Resisting Forces against Sliding}}{\textit{Disturbing Forces Initiating Sliding}} \qquad (1\text{-}3)$$

2- Factor of safety against overturning:

$$\text{Factor of safety} = \frac{\textit{Resisting Forces against Overturning}}{\textit{Disturbing Forces Creating Overturning}} \qquad (1\text{-}4)$$

When subject to static earth pressures, **2007 *CBC§1806*** requires that the factor of safety to resist overturning and the factor of safety to resist sliding shall be at least 1.5, using the Allowable Stress Design (ASD) load combinations.

SECTION 1801
GENERAL

1801.2 Design. Allowable bearing pressure, allowable stresses and design formulas provided in this chapter shall be used with the allowable stress design load combinations specified in Section 1605.3 ……..

1801.2.1 Foundation design for seismic overturning. Where the foundation is proportioned using the load combinations of Section 1605.2, and the computation of the seismic overturning moment is by the equivalent later-force method or the modal analysis method, the proportioning shall be in accordance with Section 12.13.4 of ASCE 7.

SECTION 1806
RETAINING WALLS

1806.1 General. Retaining walls shall be designed to ensure stability against overturning, sliding, excessive foundation pressure and water uplift. Retaining walls shall be designed for a factor of safety of 1.5 against lateral sliding and overturning.

1.5 OVERTURNING MOMENT AND UPLIFT FORCES

Overturning moment (OTM) is defined as:

$$\text{OTM} = \sum_{F=1}^{n} (\text{Lateral Force} \times \text{Moment Arm}) \qquad (1\text{-}5)$$

There are three definitions related to the OTM

i) Design OTM = Gross OTM – Resisting Moment (RM)

ii) Gross OTM = $\sum_{F=1}^{n}$ (Lateral Force × Moment Arm)

iii) Resisting Moment (RM) = Moment of the gravity dead load x 0.9

It should be noted that if the design OTM is a positive value (i.e. the gross OTM is more than 90 percent of the RM), the structure will have to be tied (hold down) to the foundation. The overturning moment will increase the compressive stress in the outer columns on the opposite side of the structure. The following figure shows the OTM at different levels of a building structure:

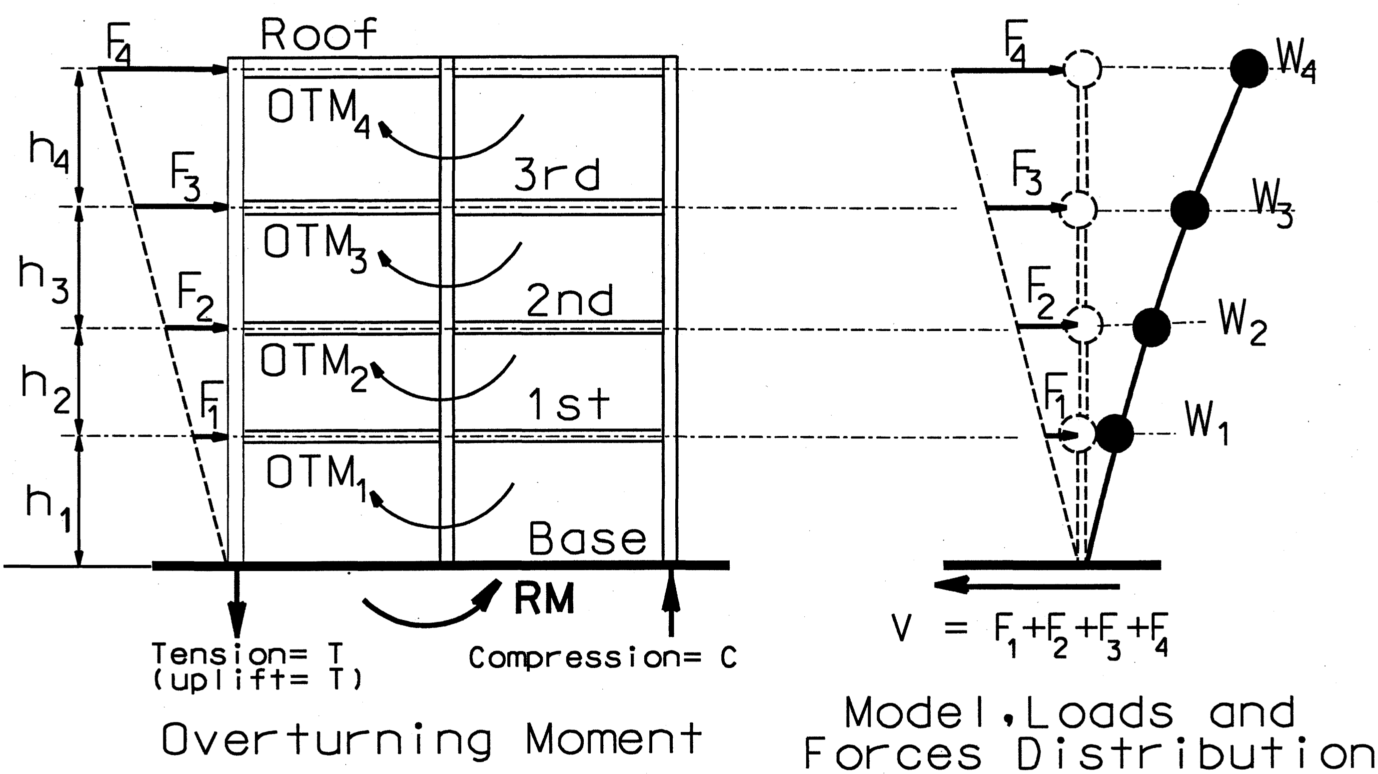


Figure 1-2 Overturning Moment and Uplift Forces

$$OTM\ at\ level\ 4 = ZERO$$

$$OTM\ at\ level\ 3 = F_4 \times h_4$$

$$OTM\ at\ level\ 2 = F_3 \times h_3 + F_4 \times (h_4 + h_3)$$

$$OTM\ at\ level\ 1 = F_2 \times h_2 + F_3 \times (h_3 + h_2) + F_4 \times (h_4 + h_3 + h_2)$$

$$OTM\ at\ the\ base = F_1 \times h_1 + F_2 \times (h_2 + h_1) + F_3 \times (h_3 + h_2 + h_1) + F_4 \times (h_4 + h_3 + h_2 + h_1)$$

The design OTM can be replaced by a couple (*T* and *C*). The tension force *T* is known as the design ***uplift force***. If the design OTM is negative or zero (i.e. the gross OTM is less than or equal to 90 percent of the RM), there will be no uplift force.

1.6 SEISMIC AND WIND LOADS

Seismic or earthquake and wind loads are lateral loads that are caused by environmental sources. The determination of the wind loads acting on a building is often the subject of an entire course. The determination of the seismic load will be discussed in detail in the following chapters.

When the code-prescribed wind design produces greater effects, the structure shall be:

1- Designed for the larger force (wind force); and

2- Detailed in accordance with the requirements and limitations prescribed by the UBC seismic design provisions.

The following table shows the major differences between seismic and wind loads:

Table 1-6 Seismic Loads Vs. Wind Loads

Seismic Loads	Wind Loads
➢ Result from the inertial response of a structure to the accelerations and displacements from the earthquake ground shaking. ➢ Tend to have somewhat unpredictable upper limits. ➢ Structures are assumed to resist the design seismic forces based on post-elastic energy dissipation.	➢ Result from aerodynamic pressures applied to an exterior surface of a structure. ➢ Tend to have somewhat predictable upper limits (e.g., 100 year storm, etc.). ➢ Structures are designed to resist the design wind forces elastically.

1.7 LOAD COMBINATIONS

Design codes as AISC (American Institute of Steel Construction), ACI (American Concrete Institute), AASHTO (American Association of State Highway and Transportation Officials), AFPA (American Forest and Paper Association), and NDS (National Design Specification for Wood Construction) provide different load conditions that need to be considered as a minimum in the analysis and design of a structural system. The **2007 CBC and 2006 IBC** have the following load combinations:

SECTION 1605
LOAD COMBINATIONS

1605.1 General. Buildings and other structures and portions thereof shall be designed to resist the load combinations specified in Section 1605.2 or 1605.3 and Chapters 18 through 23, and the special seismic load combinations of Section 1605.4 where required by Section 12.3.3.3 or 12.10.2.1 of ASCE 7. Applicable loads shall be considered, including both earthquake and wind, in accordance with the specified load combinations. Each load combination shall also be investigated with one or more of the variable loads set to zero.

1605.2 Load combinations using strength design or load and resistance factor design.

1605.2.1 Basic load combinations. Where strength design or load and resistance factor design is used, structures and portions thereof shall resist the most critical effects from the following combinations of factored loads:

$$1.4(D+F) \quad \text{(Equation 16-1)}$$
$$1.2(D+F+T)+1.6(L+H)+0.5(L_r \text{ or } S \text{ or } R) \quad \text{(Equation 16-2)}$$
$$1.2D+1.6(L_r \text{ or } S \text{ or } R)+(f_1L \text{ or } 0.8W) \quad \text{(Equation 16-3)}$$
$$1.2D+1.6W+f_1L+0.5(L_r \text{ or } S \text{ or } R) \quad \text{(Equation 16-4)}$$
$$1.2D+1.0E+f_1L+f_2S \quad \text{(Equation 16-5)}$$
$$0.9D+1.6W+1.6H \quad \text{(Equation 16-6)}$$
$$0.9D+1.0E+1.6H \quad \text{(Equation 16-7)} *$$

f_1 = 1 for floors in places of public assembly, for live loads in excess of 100 pounds per square foot (4.79 KN/m^2) and for parking garage live load, and
= 0.5 for other live loads.

f_2 = 0.7 for roof configurations (such as saw tooth) that do not shed snow off the structure, and
= 0.2 for other roof configurations.

Exception: Where other factored load combinations are specifically required by the provisions of this code, such combinations shall take precedence.

1605.2.2 Other loads. Where F_a is to be considered in the design, the load combinations of Section 2.3.3 of ASCE 7 shall be used.

1605.3 Load combinations using allowable stress design.

1605.3.1 Basic load combinations. Where allowable stress design (working stress design), as permitted by this code, is used, structures and portions thereof shall resist the most critical effects resulting from the following combinations of loads:

$$D+F \quad \text{(Equation 16-8)}$$
$$D+H+F+L+T \quad \text{(Equation 16-9)}$$
$$D+H+F+(L_r \text{ or } S \text{ or } R) \quad \text{(Equation 16-10)}$$
$$D+H+F+0.75(L+T)+0.75(L_r \text{ or } S \text{ or } R) \quad \text{(Equation 16-11)}$$
$$D+H+F+(W \text{ or } 0.7E) \quad \text{(Equation 16-12)}$$
$$D+H+F+0.75(W \text{ or } 0.7E)+0.75L+0.75(L_r \text{ or } S \text{ or } R) \quad \text{(Equation 16-13)}$$
$$0.6D+W+H \quad \text{(Equation 16-14)}$$
$$0.6D+0.7E+H \quad \text{(Equation 16-15)} *$$

Exceptions:

1. Crane hook loads need not be combined with roof live load or with more than three-fourths of the snow load or one-half of the wind load.

2. Flat roof snow loads of 30 psf (1.44 kN/m^2) or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m^2), 20 percent shall be combined with seismic loads.

1605.3.1.1 Stress increases. Increases in allowable stresses specified in the appropriate material chapter or the referenced standards **shall not** be used with the load combinations of Section 1605.3.1, except that a duration of load increase shall be permitted in accordance with Chapter 23.

1605.3.1.2 Other loads. Where is to be considered in design, the load combinations of Section 2.4.2 of ASCE 7 shall be used.

1605.3.2 Alternative basic load combinations. In lieu of the basic load combinations specified in Section 1605.3.1, structures and portions thereof shall be permitted to be designed for the most critical effects resulting from the following combinations. When using these alternative basic load combinations that include wind or seismic loads, allowable stresses are permitted to be increased or load combinations reduced where permitted by the material chapter of this code or the standards. For load combinations that include the counteracting effects of dead and wind loads, only two -thirds of the minimum dead load likely to be in during a design wind event shall be used. Where wind loads are calculated in accordance with Chapter 6 of ASCE 7, the coefficient ω in the following equations shall be taken as 1.3. For other wind loads, ω shall be taken as 1. When using these alternative load combinations to evaluate sliding, overturning and soil bearing at the soil-structure interface, the reduction of foundation over-turning from Section 12.13.4 in ASCE 7 **shall not** be used. When using these alternative basic load combinations for proportioning foundations for loadings, which include seismic loads, the vertical seismic load effect, E_v in Equation 12.4-4 of ASCE 7 is permitted to be taken **equal to zero**.

Exception: [OSHPD2] *Intermittent connections such as inserts for anchorage of nonstructural components shall not be allowed the one-third increase in allowable stresses.*

$D+L+(L_r \text{ or } S \text{ or } R)$ (Equation 16-16)

$D+L+(\omega W)$ (Equation 16-17)

$\boxed{\frac{E}{1.4} \equiv 0.7E}$

$D+L+\omega W+S/2$ (Equation 16-18)

$D+L+S+\omega W/2$ (Equation 16-19)

$D+L+S+E/1.4$ (Equation 16-20)*

$0.9D+E/1.4$ (Equation 16-21)

Exceptions:

1. Crane hook loads need not be combined with roof live loads or with more than three-fourths of the snow load or one-half of the wind load.

2. Flat roof snow loads of 30 psf (1.44 kN/m^2) or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m^2), 20 percent shall be combined with seismic loads.

1605.3.2.1 Other loads. Where F, H or T are to be considered in the design, each applicable load shall be added to the combinations specified in Section 1605.3.2.

1605.4 Special seismic load combinations. For both allowable stress design and strength design methods where specifically required by Section 1605.1 or by Chapters 18 through 23, elements and components shall be designed to resist the forces calculated using Equation 16-22 when the effects of the seismic ground motion are additive to gravity forces and those calculated using Equation 16-23 when the effects of the seismic ground motion counteract gravity forces.

$$1.2D + f_1 L + E_m \quad \text{(Equation 16-22)}$$

$$0.9D + E_m \quad \text{(Equation 16-23)}$$

where:

E_m = The maximum effect of horizontal and vertical forces as set forth in Section 12.4.3 of ASCE 7.

f_1 = 1 for floors in places of public assembly, for live loads in excess of 100 psf (4.79 kN/m^2) and for garage live load, or
= 0.5 for other live loads.

The following table shows the summary of load combinations

Table 1-7 Load Combinations

<table>
<tr><th>Load and Resistance Factor Design/ Strength Design SD/LRFD</th><th>Basic Load Combinations
(without stress increase except load duration of Chapter 23)</th></tr>
<tr><td>

ASCE # — *CBC/IBC #*
1) $1.4(D+F)$ (Equation 16-1)
2) $1.2(D+F+T)+1.6(L+H)+0.5(L_r \text{ or } S \text{ or } R)$ (Equation 16-2)
3) $1.2D+1.6(L_r \text{ or } S \text{ or } R)+(f_1L \text{ or } 0.8W)$ (Equation 16-3)
4) $1.2D+1.6W+f_1L+0.5(L_r \text{ or } S \text{ or } R)$ (Equation 16-4)
5) $1.2D+1.0E+f_1L+f_2S$ (Equation 16-5)
6) $0.9D+1.6W+1.6H$ (Equation 16-6)
7) $0.9D+1.0E+1.6H$ (Equation 16-7)

f_1 = 1 for floors in places of public assembly, for live loads in excess of 100 pounds per square foot (4.79kN/m^2) and for parking garage live load, and
= 0.5 for other live loads.
f_2 = 0.7 for roof configurations (such as saw tooth) that do not shed snow off the structure, and
= 0.2 for other roof configurations.

</td><td>

ASCE # — *CBC/IBC #*
1) $D+F$ (Equation 16-8)
2) $D+H+F+L+T$ (Equation 16-9)
3) $D+H+F+(L_r \text{ or } S \text{ or } R)$ (Equation 16-10)
4) $D+H+F+0.75(L+T)+0.75(L_r \text{ or } S \text{ or } R)$ (Equation 16-11)
5) $D+H+F+(W \text{ or } 0.7E)$ (Equation 16-12)
6) $D+H+F+0.75(W \text{ or } 0.7E)+0.75L+$
$0.75(L_r \text{ or} S \text{ or } R)$ (*Equation* 16 – 13)
7) $0.6D+W+H$ (Equation 16-14)
8) $0.6D+0.7E+H$ (Equation 16-15)

</td></tr>
<tr><td colspan="2">

$1.2D+f_1L+E_m$ (Equation 16-22)
$0.9D+E_m$ (Equation 16-23)
where:
E_m = The maximum effect of horizontal and vertical forces as set forth in Section 12.4.3 of ASCE 7.
f_1 = 1 for floors in places of public assembly, for live loads in excess of 100 psf (4.79 KN/m^2) and for parking garage live load, or
= 0.5 for other live loads.

</td></tr>
</table>

1.8 TRANSFER OF VERTICAL (GRAVITY) LOADS

The gravity loads are transmitted to the foundations through the floor/roof systems via the connections of the horizontal and vertical resisting elements. This is accomplished through joist, floor beams, beams, girders and columns or walls.

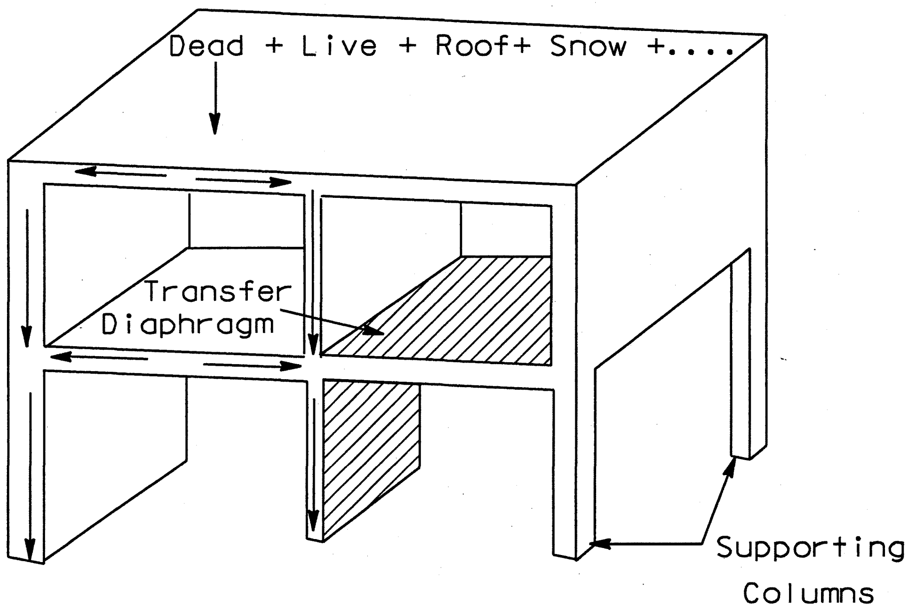


Figure 1-3 Path of Gravity Loads to Foundations

1.9 TRANSFER OF LATERAL LOADS

A continuous (complete) load path is necessary to transfer the lateral loads (seismic or wind) from the upper portion of any structure to the foundation. The lateral loads as wind and earthquakes are transferred (transmitted) to the foundations through the diaphragms (flexible or rigid), braces of braced frames and moment resisting frames.

The lateral-force-resisting-system (LFRS) may consist of shear walls, braced frames, unbraced frames and moment resisting frames. A combination of different systems could be used as in the case of dual systems. Also, a system may exist in one direction while another system in the orthogonal direction (see horizontal combination).

The total lateral force is distributed throughout the building in a manner that simulates the behavior of the building during an earthquake.

The load-path concept involves the systematic analysis of loads throughout a structure, from points of origin or application, to the final points of resistance. This analysis can be done at a "macro" level; in which general forces carried by diaphragms, collectors, walls, etc. are determined; and at a "micro" level, in which forces through bolts, stiffeners and welds within a connection are traced.

The 2007 CBC *§1604.4* requires a **complete load path** as one of its basic requirements of transferring loads from their point of origin to the load-resisting elements. Any system or method of construction to be used shall be based on a rational analysis in accordance with well-established principles of mechanics. Such analysis shall result in a system that provides a complete load path capable of transferring all loads and forces from their point of origin to the load-resisting elements. Detailed provisions for identifying and designing this load path are given in the appropriate design and material sections of the CBC and ASCE 7.

While providing a continuous (complete) load path is an obvious design requirement for vertical loads, experience has shown that it is sometimes overlooked or not completely developed when designing for lateral loads. The failure to review all members and connections for each combination of load can lead to weak-links in the load path.

Important aspects of developing and designing/detailing the lateral load path include:

1- All of the inertia forces originating from the masses on and within the structure must be transmitted from their source to the base of the structure.

2- Forces normal to the plane of a wall must be transferred either vertically to the floors above and below or horizontally to columns that are capable of transferring the forces vertically to the floors above and below.

3- Diaphragms acting as horizontal beams must transfer inertia forces to the frames and/or shear walls. Collector elements need to be provided when transferring loads from diaphragms (roof or floors) into walls, braces, or frames.

4- Frames and shear walls must transfer forces contributed from the diaphragms as well as their own inertia forces to the foundations (see Fig. 1.4).

5- Forces applied to the foundations by the shear walls and frames must be transmitted into the ground. Consideration needs to be given when transferring seismic loads through the foundation system into the surrounding soil. It is not sufficient to assume that seismic forces are "resolved" when they reach the soil system. A rational means, using each element of the foundation system (e.g., bearing, friction, passive pressure), must be employed to resist seismic loads.

6- Connections between all elements must be capable of transferring the applied forces from one element to another.

The following figure shows the load path for the lateral loads.

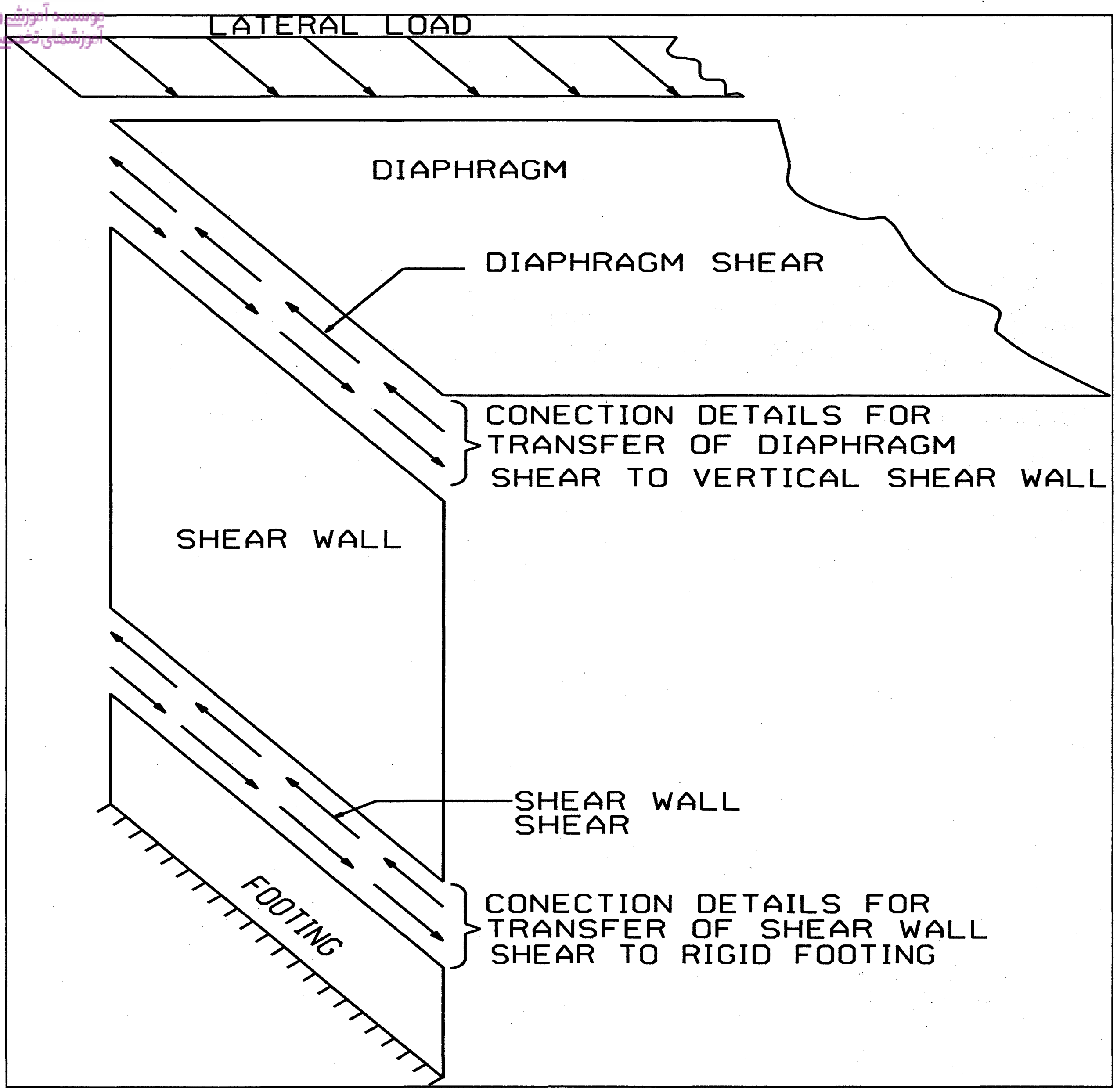


Figure 1-4 Path of Lateral Loads to Foundation

1.10 EARTHQUAKE LOADS E, E_m, E_h & E_V

The earthquake load acing on a structure consists of the following two components:

E_h = the horizontal component of the seismic force due to V or F_x or F_p
E_V = the vertical component of the seismic force

Table 1-8 Earthquake Loads

E_h	E_v	E	E_m
➢ Horizontal component ➢ F_x, F_p as determined from equations ASCE 12.8-11, 13.3-1 $F_x = C_{vx} V$ (12.8-11) and $C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$ (12.8.12) $F_p = \frac{0.4 a_p S_{DS} W_p}{\left(\frac{R_p}{I_p}\right)}\left(1+2\frac{z}{h}\right)$ $(13.3-1)$	➢ Vertical component $E_v = 0.2\, S_{DS} D$ ASCE (12.4-4) S_{DS} = design spectral response acceleration parameter at short periods obtained from Section 11.4.4 D = effect of dead load ➢ *EXCEPTIONS:* *1-E_v = 0 where S_{DS} is equal to or less than 0.125.* *2. E_v = 0 in Eq. 12.4-2 where determining demands on the soil-structure interface of foundations.*	➢ Earthquake force for load combination 5 of SD and 5 & 6 for ASD $E = E_h + E_v$ ASCE (12.4-1) ➢ Earthquake force for load combination 7 of SD and 8 for ASD $E = E_h - E_v$ ASCE (12.4-2) $E_h = \rho\, Q_E$ *ASCE* (12.4-3) where : Q_E = effects of horizontal seismic forces from V or F_P. ρ = redundancy factor, as defined in Section 12.3.4	➢ Seismic load including overstrength factor ➢ Earthquake force for load combination 5 of SD and 5 & 6 for ASD $E_m = E_{mh} + E_v$ *ASCE (12.4-5)* ➢ Earthquake force for load combination 7 of SD and 8 for ASD $E_m = E_{mh} - E_v$ *ASCE (12.4-6)* $E_{mh} = \Omega_0\, Q_E$ *ASCE (12.4-7)* where Q_E = effects of horizontal seismic forces from V or F_p Ω_0 = overstrength factor (Table 12.2-1)

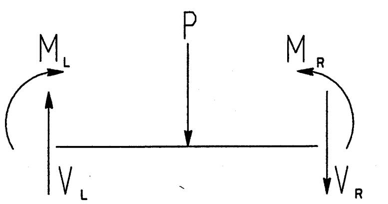
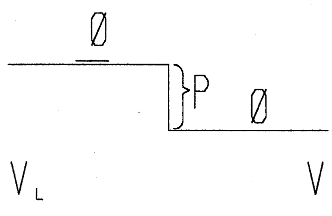
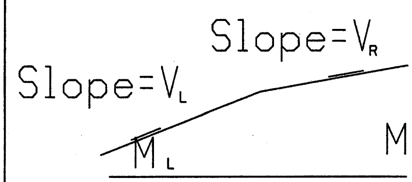
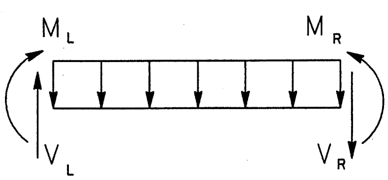
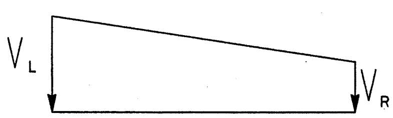
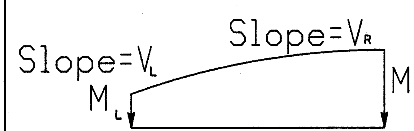
1.11 SHEAR AND MOMENT DIAGRAMS

Shear and moment diagrams are the cornerstone in the analysis and the design process of any structural system. The following are the relationships between load (w) and shear force (V) and moment (M):

$$\frac{dV}{dx} = -w = \text{Slope of the shear diagram equals the load} \qquad (1-6)$$

$$\frac{dM}{dx} = V = \text{Slope of the moment diagram equals the shear} \qquad (1-7)$$

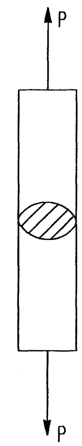
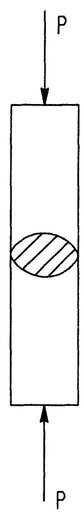
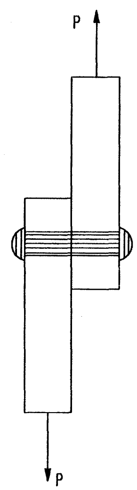
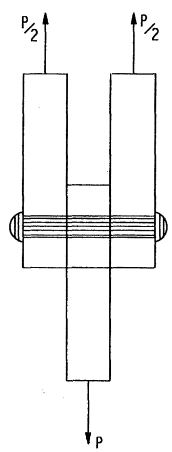
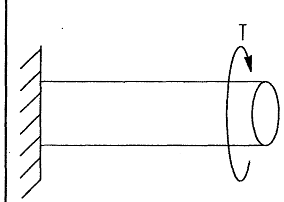
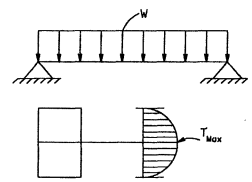
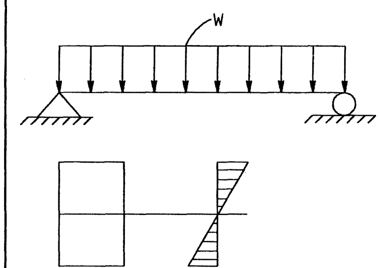
Table 1-9 Load-Shear-Moment Relationships

Case		Loading	Shear Diagrams $\frac{dV}{dx} = -w$	Moment Diagrams $\frac{dM}{dx} = V$
1	Concentrated Load		 zero slope	 positive increasing slope
2	Uniform Distributed Load		 Constant slope	 positive decreasing slope

1.12 NORMAL, SHEAR, TORSION AND FLEXURE STRESSES

Stresses developed in structures or members due to the applications of loads are required for the design process. Stresses are classified as follows:

Table 1-10 Types of Stresses

Normal Stress - σ		Shear Stress- τ				Bending Stress (Flexure Stress)
		Direct		Indirect		
Tension	Compression	Single	Double	Due to Torsion	Due to Bending	
$\sigma = \dfrac{P}{A}$  P = force A = area perpendicular to P	$\sigma = \dfrac{P}{A}$  P = force A = area perpendicular to P	$\tau = \dfrac{P}{A}$  P = force A= area parallel to P	$\tau = \dfrac{P}{2A}$  P = force A = area parallel to P	$\tau = \dfrac{Tr}{J}$  T= torque r = radius J = polar moment of inertia = $I_x + I_y$	$\tau = \dfrac{VQ}{Ib}$  V = shear force Q = 1st moment of area I = moment of inertia b = width of section	$\sigma = \dfrac{MC}{I}$ $\sigma = \dfrac{M}{S}$  M = moment C = distance from N.A. to extreme fibers I = moment of inertia S = elastic section modulus

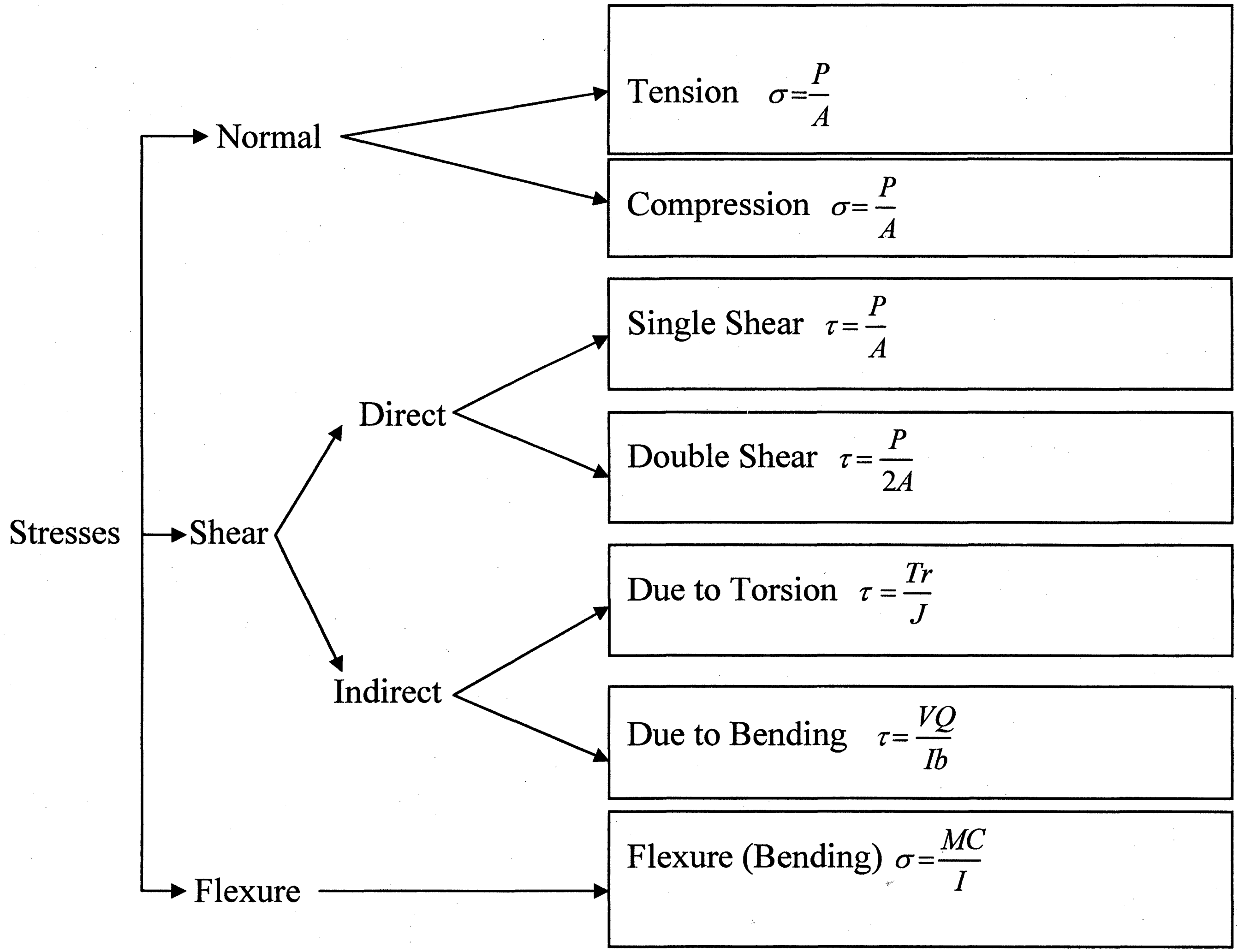
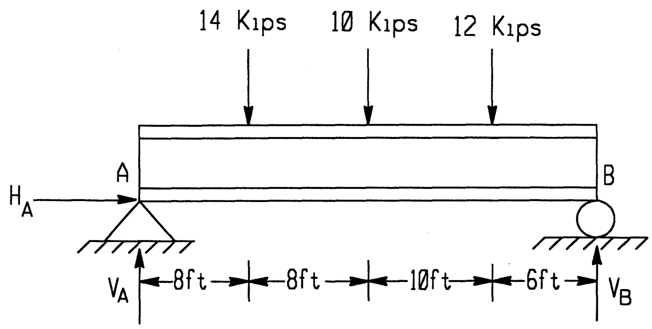


Figure 1-5 Types of Stresses

1.13 SAMPLE PROBLEMS

Sample Problem 1-1: Loads & Reactions

Given: A simple beam with the loading as shown



Find: The reaction at support A is most nearly:

Answer:

A) 17.50 kips
B) 17.75 kips
C) 18.50 kips
D) 18.75 kips

Solution:

The beam is statically determined (# of unknowns = # of equilibrium equations).
Two unknown reactions at support "A" (pin or hinge) and one unknown at support "B" (roller support).

H_A, V_A & V_B are the three unknowns:

$\Sigma F_H = 0$, $\Sigma F_V = 0$ & $\Sigma M = 0$ *three equilibrium equations*

By inspection $H_A = 0$

Applying one of the equilibrium equations yields:

$\Sigma F_H = H_A = 0 \qquad \Rightarrow H_A = 0$

Summing moment about support "B" assuming clockwise moment is positive

$\Sigma M_B = 0$

$\Sigma M_B = V_A(32) - 12(6) - 10(16) - 14(24) = 0 \qquad \Rightarrow \boxed{V_A = 17.75 \text{ kips} \uparrow}$

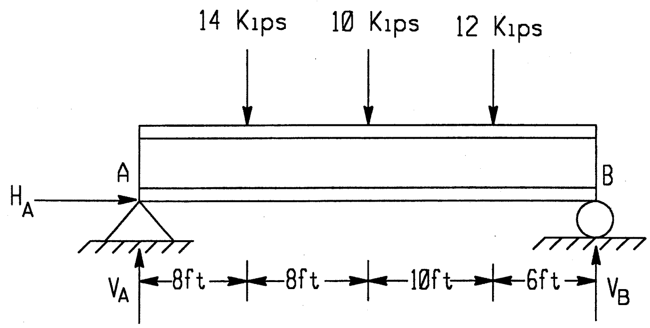
Notes: 1-The positive sign of the reaction at A indicates that the assumption of the direction of the reaction was correct.

2- If the sign of the final answer is negative, that means the assumed direction was not correct and the direction of the force must be reversed.

Answer: B ⇐

Sample Problem 1-2: Shear and Moment Relationship and Diagrams

Given: A simple beam with the loading as shown



Find: The maximum bending moment is most nearly:

Answer:

A) 110 kip-ft
B) 142 kip-ft
C) 172 kip-ft
D) 192 kip-ft

Solution:

Step 1: Find external reactions (if necessary)
Step 2: Draw shear diagram and calculate areas under the shear diagram
Step 3: Construct moment diagram from the areas of the shear diagram using this relationship between shear and moment

$$\boxed{\Delta M_{1-2} = \int_1^2 V(x)\,dx \quad \textit{i.e. Change in Moment = Area Under Shear Diagram}}$$

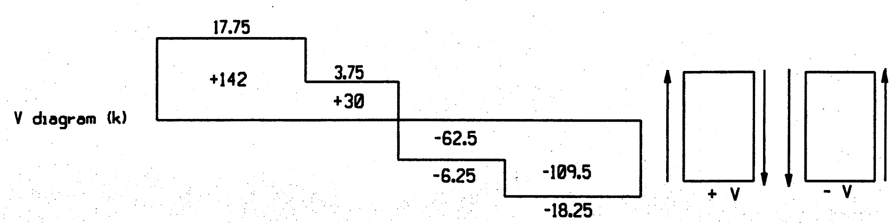
From Sample 1-1 $\boxed{V_A = 17.75 \text{ kips} \uparrow}$

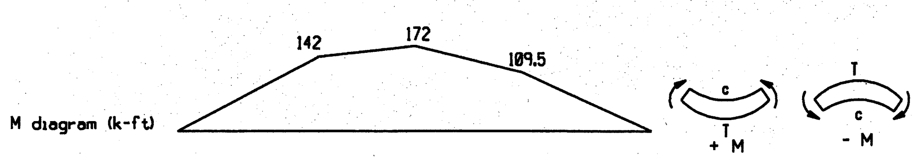
$\Sigma M_A = 0$

$\Sigma M_A = V_B(32) - 12(26) - 10(16) - 14(8) = 0 \quad \Rightarrow \boxed{V_B = 18.25 \text{ kips} \uparrow}$

Check: $\sum F_V = 0 \uparrow \oplus \quad -14 - 10 - 12 + 17.75 + 18.25 = 0$ OK

Sign conventions for shear and moment diagrams:

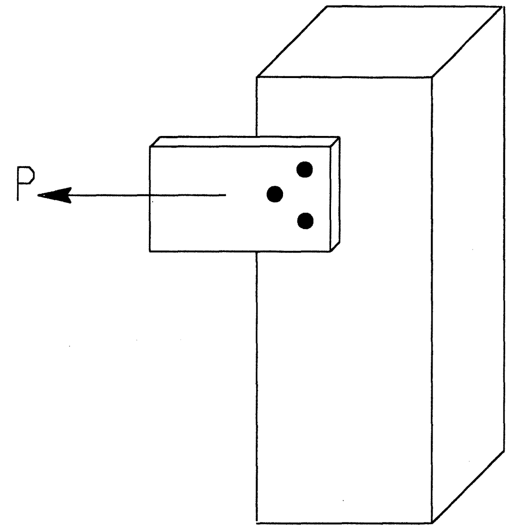




Answer: C ⇐

Sample Problem 1-3: Direct Shear (single and double)

Given: Three steel bolts are used to attach the steel plate to a vertical column in a building as shown.
It was determined that the lateral seismic force P is 25 kips. The ultimate shear stress for the steel is 51,000 psi and a factor of safety of 2.5 is required.



Find: The diameter for the bolts is most nearly:

Answer: A) 0.625 inches B) 0.750 inches
C) 1.00 inches D) 1.125 inches

Solution:
This is a single (one shear plane) direct shear stress

The load carried by each bolt = P/3 = 25/3 = 8.33 kips.

$$\boxed{\text{Factor of safety (Factor of ignorance)} = \text{F.S.} = \frac{\textit{Ultimate Load}}{\textit{Allowable Load}} = \frac{P_{ult}}{P_{all}} \quad (1\text{-}1)}$$

$$P_{ult} = (\text{F.S})\,(P_{all})$$
$$= (2.5)\,(8.33) = 20.83 \text{ kips}$$

$$\text{Allowable shear stress} = 51{,}000 \text{ psi} = \tau_{ult} = \frac{P_{ult}}{Area} = \frac{20.33\,kips}{\pi d^2/4}$$

$$d = \sqrt{\frac{(4)\,(20.33\,kips)\,(1000\,lb/kip)}{(51000\,psi)\,(\pi)}} = 0.721 \; inches$$

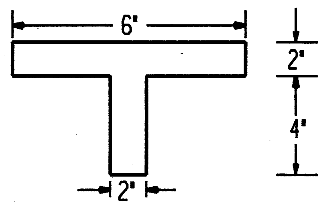
$$\boxed{\text{Select } 3/4 \text{ inch bolts}}$$

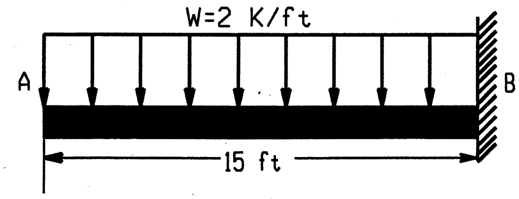
Note: The same solution could be obtained using stress instead of load

Answer: B ⇐

Sample Problem 1-4: Bending Stress

Given: The cantilever beam has a cross section and is loaded as shown





Find: The maximum bending stress is most nearly:

Answer: A) 177.3 ksi tension B) 177.3 ksi compression
C) 17.1 ksi tension D) 17.1 ksi compression

Solution:

The centroid of the area is located with respect to the bottom of the web as follows:

$$\bar{y}=\frac{\sum a_i\,y_i}{a_i}=\frac{(4x2)(2)+((6x2)(5)}{(2x4)+(2x6)}=3.8\ inches$$

The moment of inertia about the centriod using parallel axis theorem as follows:

$$I=I_{\bar{x}}+Ad^2=\frac{(2)(4)^3}{12}+(2x4)(3.8-2)^2+\frac{(6)(2)^3}{12}+(2x6)(5-3.8)^2=57.87\ in^4$$

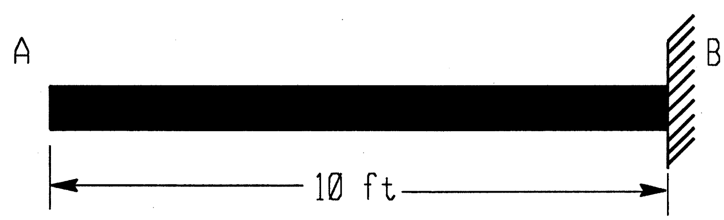
Maximum bending moment occurs at the fixed end = (2)(15)(15)/2 = 225 k-ft =2700 k-in
Maximum bending stress occurs at the maximum bending moment and the larger distance "C" i.e. at the bottom of the web.

$$\sigma_{\max}=\frac{(2700)(3.8)}{57.87}=\underline{177.3\ \ ksi\ (compressive\ bending\ stress)}\Leftarrow$$

Answer: B ⇐

Sample Problem 1-5: Seismic Forces

Given:
For the non-prestressed cantilever beam shown. Beam unit weight = 200 plf. Seismic Design Category D and $S_{DS}=1.0g$



Find: The upward seismic forces on the beam is most nearly:

Answer: A) 40 plf B) 60 plf C) 80 plf D) 120 plf

Solution:

$$E=E_h+E_v \qquad (12.4\text{-}1)$$

$$E=E_h-E_v \qquad (12.4\text{-}2)$$

$$E_h=\rho\,Q_E \qquad (12.4\text{-}3)$$

$Q_E=0$ (no vertical load)

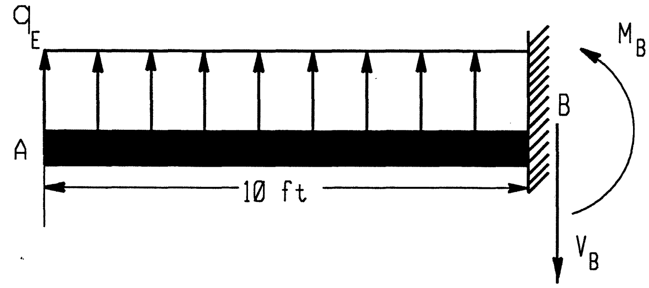
$$E_v=0.2S_{DS}D \qquad (12.4\text{-}4)$$

$= 0.2\ (1)\ (200\ \text{plf}) = 40\ \text{plf}$

Answer A ⇐

Sample Problem 1-6: Bending Moment due to Seismic Force

Given: For the non-prestressed cantilever beam shown.
Beam unit weight = 200 plf
Seismic Design Category D and
$S_{DS} = 1.0g$. Use the answer from problem 1-5



Find: The maximum bending moment is most nearly:

Answer: A) 2000 lb-ft B) 2800 lb-ft C) 3000 lb-ft D) 3200 lb-ft

Solution: The maximum bending moment occurs at the fixed end of the beam

$$M_{\max} = \frac{q_E l^2}{2} = \frac{(40)(10)^2}{2} = \underline{2000 \ \ lb-ft} \Leftarrow$$

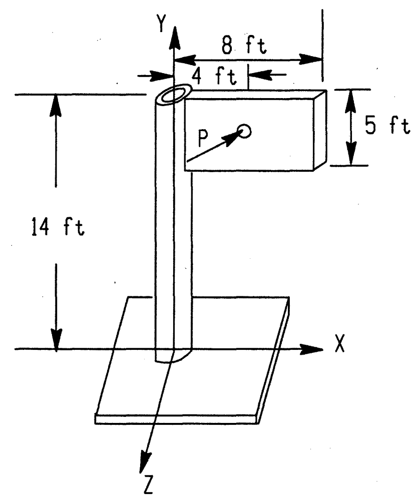
Also the maximum shear occurs at the fixed end and the value is

$$V_{\max} = q_E l = 40\ plf\ x 10\, ft\ = 400\, lbs$$

Answer A ⇐

Sample Problem 1-7: Torsional Stress

Given:
A sign is subjected to a total lateral wind force of 1.1 kips acting at the C.G. as shown. The vertical post is a hollow steel pipe with inside and outside diameters of 12 and 14 inches respectively.



Find: The maximum shear in the post due to torsional moment is most nearly

Answer: A) 0.15 ksi B) 0.18 ksi C) 0.20 ksi D) 0.21 ksi

Solution:

The torsional moment = M_T = Force x Eccentricity = Pe = (1.1 kips) (4 ft) = 4.4 k-ft

This is an indirect shear stress due to torsion (see section 1.12)

$$\tau = \frac{Tr}{J}$$

T = torque
r = radius

J = polar moment of inertia = $I_x + I_y$ $= \frac{\pi}{2}(r_o^4 - r_i^4) = \frac{\pi}{2}(7^4 - 6^4) = 1735.73\ in^4$

The maximum shear stress will occur on the outer fibers of the post

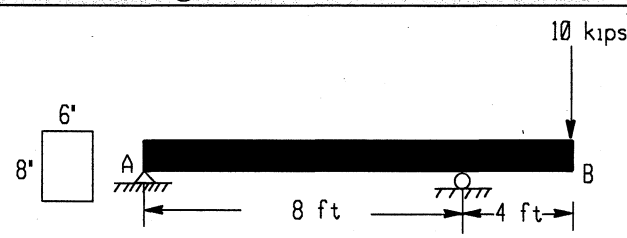
$$\tau = \frac{Tr}{J} = \frac{(4.4k-ft)(12in/ft)(7\,in)}{173573\ in^4} = \underline{0.21\ ksi} \Leftarrow$$

It should be noted that the above equation for calculating shear stress due to torsion is limited to circular shafts (hollow and solid). This equation will be used later to calculate the torsional shear due to lateral seismic forces. However, the polar moment of inertia will be modified to account for the non-circular shapes of structures.

Answer D ⇐

Sample Problem 1-8: Bending Stress

Given:
The overhang beam is loaded as shown.
The beam has a rectangular cross-section as shown



Find: The maximum bending stress:

Answer: A) 6.5 ksi B) 7.5 ksi
C) 8.5 ksi D) 11.5 ksi

Solution:
The maximum bending moment is 40 k-ft (occurs at the roller support/right support)

The moment of inertia of rectangular cross section is = $bh^3/12$

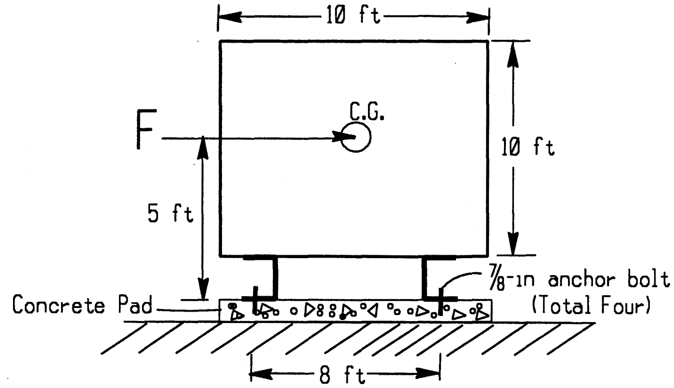
The maximum bending stress occurs at the maximum bending moment and at the furthest point from N.A.

$$\sigma = \frac{MC}{I} \text{ OR } \sigma = \frac{M}{S} = \frac{(40k-ft)(12in/ft)(4in)}{(6)(8)^3/12} = \underline{7.5\ ksi} \Leftarrow$$

Answer B ⇐

Sample Problem 1-9: Overturning Moment

Given: A 10 ft x 10 ft x 10 ft storage tank weighs with its contents 62.4 kips and is supported by two channels as shown. The total seismic lateral force was calculated and found to be 19.22 kips acting at the centriod which is 5 ft from the concrete pad. The steel channels are connected to the concrete pad by four-7/8-inch A-325 anchor bolts.



The tank is located in SDC "C" and the concrete has a 28-day strength of 3000 psi.

Find: The over turning moment (OTM) is most nearly

Answer: A) 96.10 k-ft B) 98.10 k-ft C) 100.10 k-ft D) 104.10 k-ft

Solution:

§ 1630.8 of the Code addressing the issue of the overturning moment:

$$\text{Moment} = \text{Force x distance (arm)} = F\,d$$

The overtuning moment is resulted from the lateral seismic force

$$\text{OTM} = (19.22 \text{ kips})\ (5 \text{ ft}) = \underline{96.10 \text{ k-ft}} \Leftarrow$$

For structures that consist of more than one level, the overturning moment to be resisted shall be determined using the seismic force (F_x) that act on levels above the level under consideration (see Figure 1-2)

NOTE: This tank is a good example of a "nonbuilding structure" where the tank is self supporting. If the seismic lateral force is not given, the appropriate equation in Chapter 15 of the ASCE will be used.

§15.4.2 Rigid Nonbuilding Structures. Nonbuilding structures that have a fundamental period, T, less than 0.06 s, including their anchorages, shall be designed for the lateral force obtained from the following:

$$V = 0.30 S_{DS} W I \qquad (15.4\text{-}5)$$

Answer A ⇐

Sample Problem 1-10: Shear Stress due to a Seismic Force

Given: Same as Sample problem 1-9

Find: The service shear force per bolt is most nearly:

Answer: A) 4.81 kips B) 4.61 kips C) 4.01 kips D) 3.43 kips

Solution:

The seismic lateral force given in the problem statement is based on STRENGTH DESIGN (SD). To convert from the SD level to ASD (service) level, divide by a factor of 1.4 as follows:

$$\boxed{ASD(value) = \frac{SD(value)}{1.4}}$$

Therefore, service shear per bolt $= \dfrac{19.22\ kips}{4\,bolts\ x 1.4} = \underline{3.43\,kips} \Leftarrow$ Answer: D⇐

Sample Problem 1-11: Resisting Moment (RM)

Given: Same as Sample problem 1-9

Find: The resisting moment is most nearly:

Answer: A) 312 k-ft B) 249.60 k-ft C) 224.64 k-ft D) 220.34 k-ft

Solution: The resisting moment (RM) is resulted from the gravity dead load. Taking the moment about one of the anchor bolts (either left or right):

Resisting Moment (RM) = Fd = (62.4 kips)(4 ft)(0.90) = 224.64 k-ft ⇐

Note: A common mistake is to ignore the factor of 0.90

Answer: C⇐

Sample Problem 1-12: Factor of Safety Against Overturning

Given: Same as Sample problem 1-9

Find: The factor of safety against overturning is most nearly:

Answer: A) 1.50 B) 1.75 C) 2.14 D) 2.34

Solution:

$$\boxed{\text{Factor of safety} = \frac{Resisting\ Moment}{Overturning\ Moment} = \frac{224.64\,k-ft}{96.1\,k-ft} = 2.34} \Leftarrow$$

If the resisting moment is less than the overturning moment, the bolts will be under tension due to the uplift force. A minimum factor of safety of 1.5 is required by the Code.

Answer: D⇐

Sample Problem 1-13: Tension in Anchor Bolts

Given: Same as Sample problem 1-9

Find: The maximum tensile force in the anchor bolts is most nearly

Answer: A) 0 kip B) 1.2 kips C) 1.8 kips D) 2.1 kips

Solution:
The tensile force in the anchor bolts will be generated if the overturning moment (OTM) is grater than the resisting moment (RM).

In this example OTM < RM ⇒ 96.1 k-ft < 224.64 k-ft
Therefore, the tensile force in the anchor bolts is ZERO. ⇐
Note: The same concept will be used in calculating the uplift and compressive force for shear walls.

Answer: A ⇐

Sample Problem 1-14: Factor of Safety

Given: When designing for overturning, which factor must be applied to the dead load?
A) 0.50 B) 0.70 C) 0.90 D) 1.1

Solution: $0.9D + 1.0E + 1.6H$ (Equation 16-7)
From the load combination load shown, the factor is 0.9 Answer: C ⇐

Sample Problem 1-15: Factor of Safety

Given: What factor of safety against overturning is required for seismic loads?
A) 0.75 B) 1.0 C) 1.25 D) 1.5

Solution:
§1806.1 General. ….. "Retaining walls shall be designed for a factor of safety of 1.5 against lateral sliding and overturning". Answer: D ⇐

Sample Problem 1-16: Wind Versus Seismic

Given: When wind loads on a structure exceed seismic loads on the structure, which of the following is true?

A) The wind design shall govern, but structure detailing requirements and limitations shall comply with the CBC earthquake regulations
B) Wind loads must be considered in combination with seismic loads
C) Wind load governs, and structural detailing shall be in accordance with CBC & ASCE 7
D) Both "a" and "b"

Solution: **§1604.10** ***Seismic and Wind detailing***. Lateral-force-resisting systems shall meet seismic detailing requirements and limitations prescribed in this code and ASCE 7, excluding Chapter 14 and Appendix 11 A, even when wind code prescribed load effects are greater than seismic load effects.

Answer: C⇐

Sample Problem 1-17: Stress Increase (Load Duration Factor)

Given: Stress increase is allowed in wood design using the load combinations given in 2007 CBC Section §1605.3.1 by an amount of :

A) 0.15
B) 0.25
C) 0.60
D) 1.00

Solution:

Load duration factor C_D per NDS 2005 for wind or seismic force is 1.6 i.e. there is a 60% increase for the tabulated stresses in the NDS supplement.

§1605.3.1.1 stated that "Increases in allowable stresses specified in the appropriate material chapter or the referenced standards shall not be used with the load combinations of Section 1605.3.1, except that a duration of load increase shall be permitted in accordance with Chapter 23."

Answer: C ⇐

Sample Problem 1-18: Snow Load Combination with Seismic Load

Given: Based on 2007 CBC Section § 1605.3.1, snow loads need not to be combined with seismic load when the snow loads is:

A) ≤ 30 pounds per square foot
B) > 30 pounds per square foot
C) 35 pounds per square foot
D) 40 pounds per square foot

Solution:

§1605.3.1 on page 8 of the 2007 CBC.

Answer: A ⇐

Sample Problem 1-19: Snow Load Reduction with Seismic Load

Given: Snow loads may be reduced when combined with seismic load by an amount of :

A) 0.30
B) 0.40
C) 0.60
D) 0.80

Solution:

§1605.3.1 on page 8 of the 2007 CBC

Answer: D ⇐

Chapter 2

Earthquakes: Basic Theory & Characteristics

Topics to be covered

- Types of Faults
- California Faults
- Seismic Waves
- Earthquake Measurements (Intensity, Magnitude, Number and Probability of Events)

17 Sample Problems

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Chapter 2- Earthquakes: Basic Theory & Characteristics

2.1 INRODUCTION

An earthquake is a ground shaking that radiates seismic energy caused most commonly by sudden slip on a fault, volcanic or magnetic activity, or other sudden stress changes in the earth. The following figure shows the terminology used with earthquakes

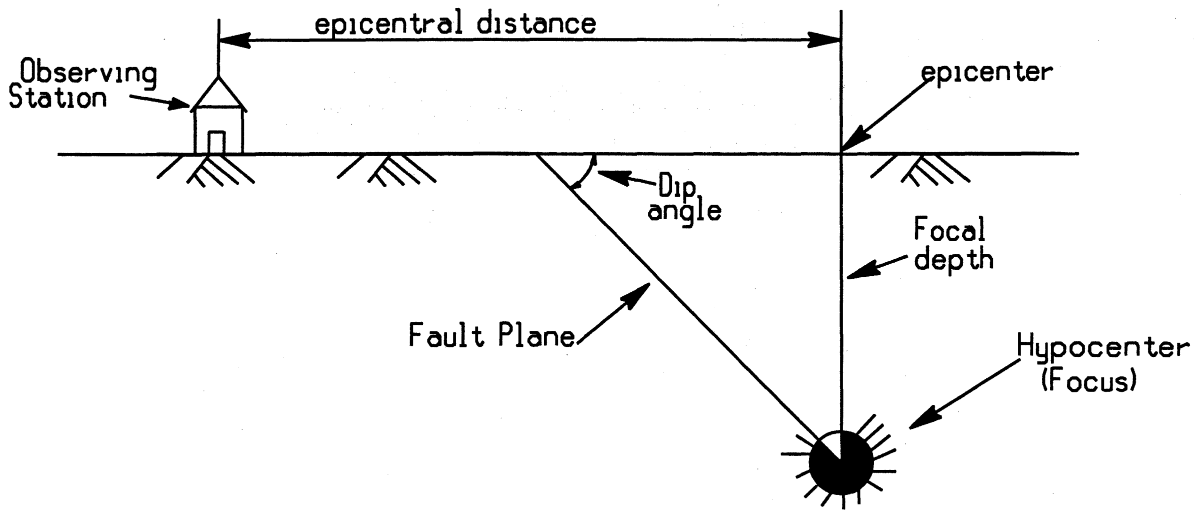


Figure 2-1 Earthquake Terminology

Focus (or hypocenter): the center of the initial rupture causing the earthquake

Epicenter: a point on the earth's surface directly above the focus

Epicentral distance: the distance from the epicenter to the site

Focal depth: the depth of the focus beneath the surface

Hypocentral distance: the distance from the focus or hypocenter to the site.

The dip of a fault is the angle that fault surface makes with a horizontal plane while strike is the direction of the fault line exposed at the ground surface relative to the north

The magnitude of the focal depth classifies types of earthquakes as follows:

i) Shallow Earthquakes: have a focal depth of about 45 miles (72 kilometers). They are caused by the fracturing the crust of the earth or when the internal strain energy ($U = \sigma\varepsilon/2$) exceeds the friction looking the two sides of the fault. California earthquakes are shallow where the focal depth can be as small as 10 miles (16 km).

ii) Intermediate Earthquakes: have a focal depth that varies between 45 to 185 miles (72 – 298 kilometers).

iii) Deep Earthquakes: have a focal depth greater than 185 miles and can reach 400 miles (298-640 kilometers).

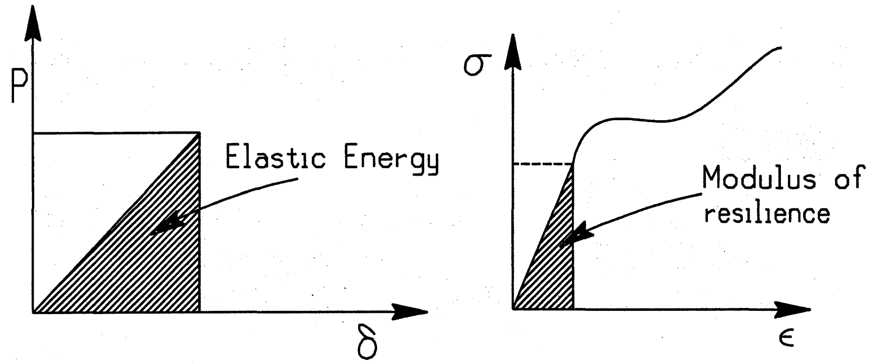


Figure 2-2 Strain Energy

Table 2-1 Classification of Earthquakes

Classification	Depth of Focus (mile)
Shallow	0-45
Intermediate	45-185
Deep	> 185

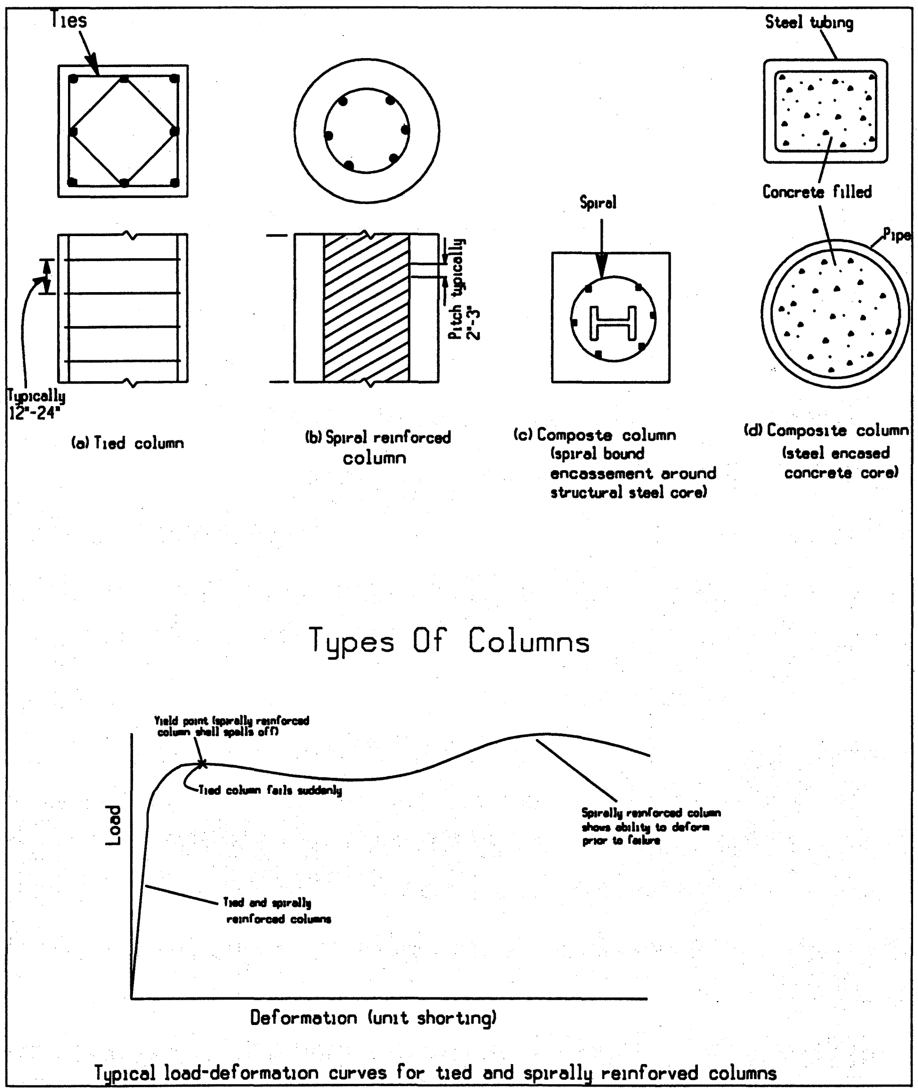


Figure 2-3 Tied and Spiral Columns

The following definitions are commonly used with the earthquakes:

1. **Maximum possible earthquake.** The largest earthquake that can be postulated to occur. Conceptual only. Probable magnitude 8.7 to 9.5.
2. **Maximum credible earthquake (MCE).** The largest earthquake that can be reasonably expected to occur.
3. **Maximum expectable earthquake**. The largest earthquake that can be reasonably expected to occur. (USGS Definition)
4. **Maximum probable earthquake.** The worst historic earthquake. Alternatively it is (a) the 100-year recurrence earthquake, or (b) the maximum earthquake that may occur during the life of the structure at a specified, probabilistic level of occurrence.
5. **Design basis earthquake (DBE)**. Same as Maximum credible earthquake. Maximum expectable earthquake.
6. **DESIGN EARTHQUAKE (ASCE 7- 05, Chapter 11)**: The earthquake effects that are two-thirds of the corresponding Maximum Considered Earthquake (MCE) effects.
7. **DESIGN EARTHQUAKE GROUND MOTION (ASCE 7-05 Chapter 11)**: The earthquake ground motions that are two-thirds of the corresponding MCE ground motions.

2.2 TYPES OF FAULTS

A fault is a fracture in the crust of the earth along which rocks on one side have moved relative to those on the other side. Most faults are the result of repeated displacements over a long period of time. A fault trace is the line on the earth's surface defining the fault. An active fault is one that has moved in the last 11,000 years as defined in the Alquist-Priolo Act of 1972.

The abrupt changes in the structure of rocks along a plane is known as *"a fault"*. The length of faults vary from few inches to many miles. The existence of these faults indicate that a movement has been occurred in the past along these faults lines. The movement could be either a *slow slip* which generates no ground shaking or *a sudden and strong rupture* which produces *earthquakes.*

The classification of faults depends on the geometry and direction of relative slip or movement of one part relative to the other. A fault in which the movement is vertical is called a dip-slip fault as the normal and reverse faults. A fault in which the movement is horizontal is called a right or left lateral fault. When the two blocks of the fault move vertically and horizontally at the same time, the fault is named by combining the two movements as shown in table 2.2. Figure 2.4 shows the terminology associated with the classification of faults.

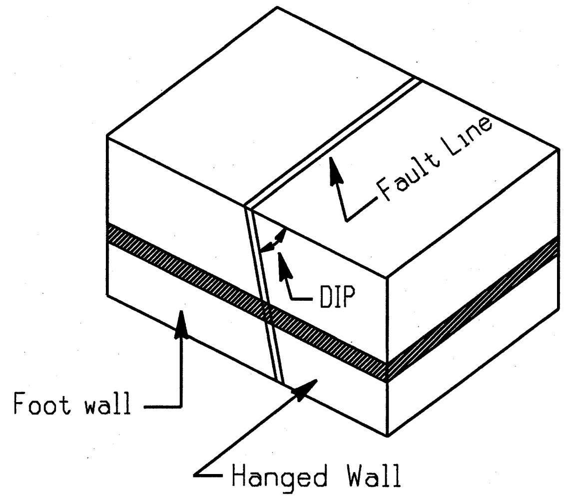
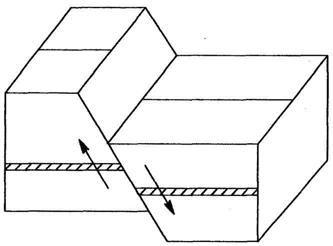
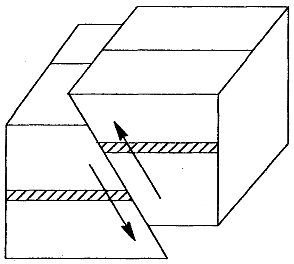
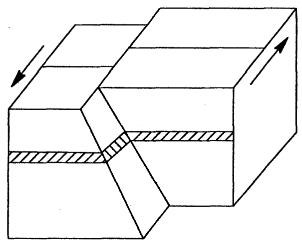
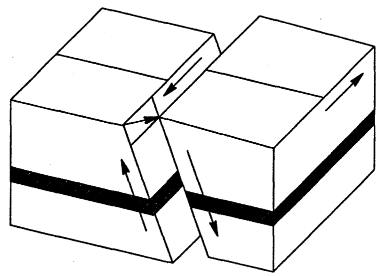
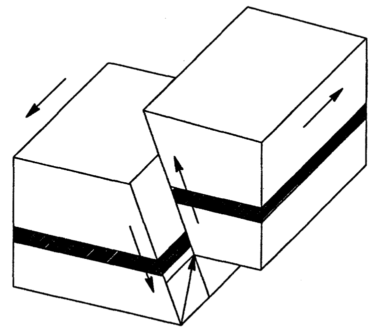


Figure 2-4 Faults Terminology

Fault displacement can be horizontal , vertical , or a combination of both. Most of the faults in California are vertical or nearly vertical. However, the 1906 San Francisco earthquake was almost entirely horizontal along the San Andreas fault. The San Andreas fault is a right lateral fault (see California Faults).

Table 2-2 Types of Faults

Normal Fault	Reverse Fault	Left Lateral Fault (Strike Slip)	Left Lateral Normal Fault (left Oblique Normal)	Left Lateral Reverse Fault (left Oblique Reverse)
➢ Hanging wall moves downward relative to foot wall	➢ Hanging wall moves upward relative to footwall	➢ Hanging wall moves left relative to footwall	➢ Hanging wall moves downward and to the left relative to footwall	➢ Hanging wall moves upward and to the left relative to footwall
				

2.3 CAUSES OF EARTHQUAKES

Currently, there are different theories explaining the causes of earthquakes. The energy released into the crust of the earth can be generated from different sources such as:

- i) Tectonic Earthquakes -dislocation of portion of the crust of the earth
- ii) Volcanic activities
- iii) Underground explosions
- iv) Reservoir

➢ Tectonic Earthquakes:

Over the years, many theories have been proposed to explain the causes of earthquakes. The plate tectonic theory is generally considered to be the most reliable. According to this theory, the earth's outer layer, referred to as the lithosphere, consists of approximately one dozen hard tectonic plates as shown in Figure 2.5. These plates, having an average thickness of 50 miles, sit on a comparatively soft asthenosphere and move as rigid bodies. The plates interact with one another, and these interactions have been the primary cause of orogeny (i.e. formation of mountains) throughout geological history.

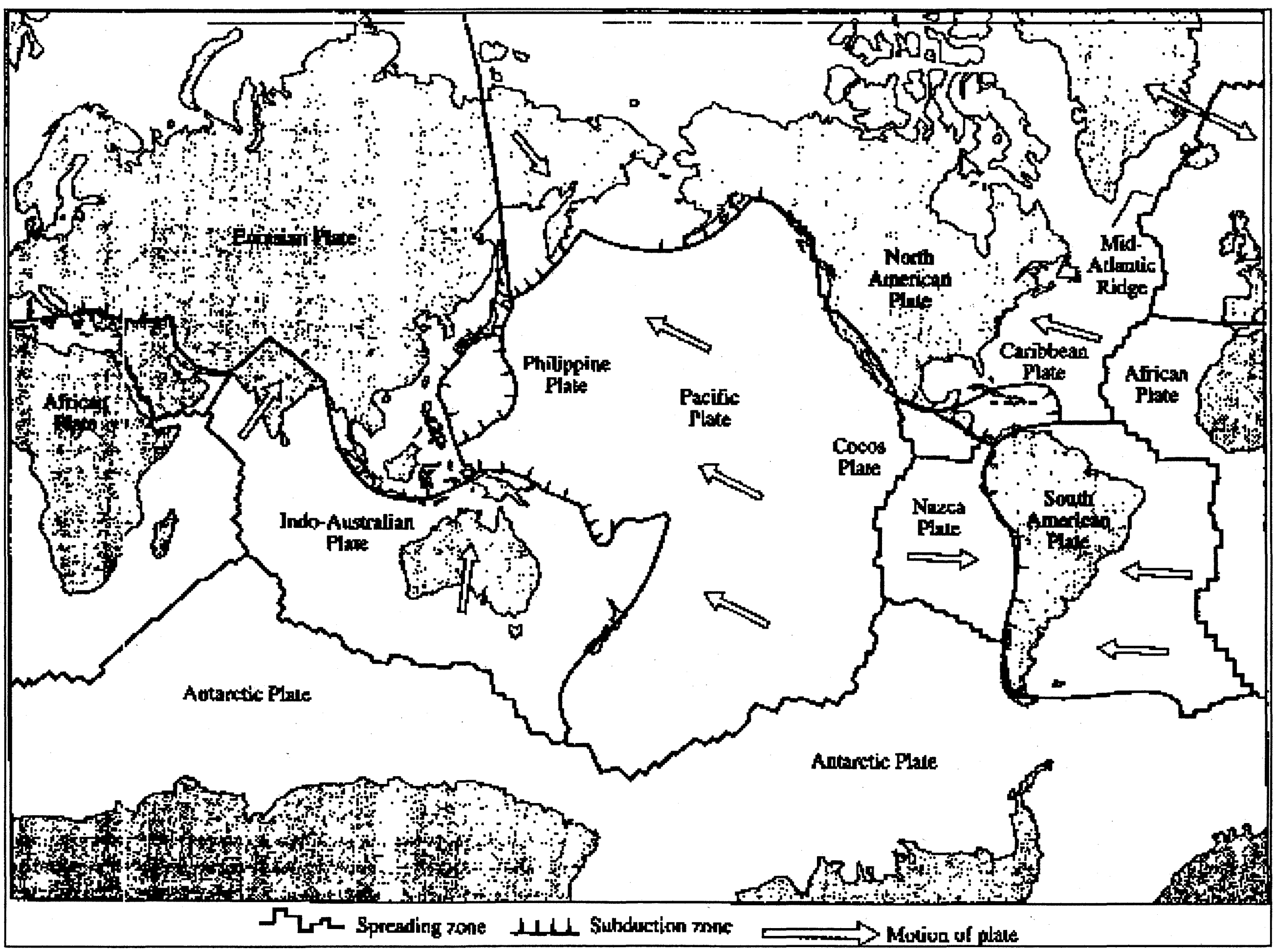


Figure 2.5 Major Tectonic Plates Worldwide

Three basic types of plate interactions can occur:

1- The first form or interaction involves two plates slipping apart. As this slippage occurs, hot mantle flows up toward the earth's surface and cools down, forming midoceanic ridges.
2- The second type of interaction occur when two plates slide horizontally, one over the other, and create a transform fault.
3- The third type of interaction occurs when a high-density oceanic plates subducts (creating subdeuction zone) beneath a low-density continental plate, forming a trench and an island arc.

➢ Volcanic Earthquakes:

Volcanoes and earthquakes often occur together along the margins of plates around the world. Despite the connections between volcanoes and earthquakes, there is no evidence that moderate to shallow earthquakes are not essentially all of tectonic nature due to volcanoes. The earthquakes that can be reasonably related to volcanoes are relatively rare and they are three categories as follows:

2) volcanic explosions
3) shallow earthquakes arising from magma movements, and
4) sympathetic tectonic earthquakes.

➢ Explosions Earthquakes:

Earthquakes may be produced by the underground detonation of nuclear or chemical devices. When a nuclear device is detonated in a borehole underground, huge nuclear energy is released. Underground nuclear explosions fired in the past several decades at different sites around the world have produced substantial earthquakes up to a magnitude of 7.0

➢ Large Reservoir-Induced Earthquakes:

The idea that earthquakes might be triggered by impounding surface water is not new. In the 1870s, the U.S. Corps of Engineers rejected proposals for major water storage in the Salton Sea in southern California on the grounds that such action might cause earthquakes. The first evidence of such an effect came in 1935 with the filling of Lake Mead behind Hoover Dam whose height of about 725 feet. Although there may be have been some local seismicity before 1935, after 1936 earthquakes were much more common.

2.4 CALIFORNIA FAULTS

The earthquakes of California are caused by the movement of huge blocks of the earth's crust. Southern California straddles the boundary between the Pacific and North American plates. These large sections of the earth's crust (the North American plate extends east to Iceland while the Pacific plate extends west to Japan) are moving past each other.

The Pacific plate is moving northwest, scraping horizontally past North America at a rate of about 50 millimeters (2 inches) per year. About two-thirds of this 50 millimeters per year occurs on the San Andreas fault and some parallel faults- the San Jacinto, Elsinore, and Imperial faults. These four faults are among the fastest moving, and therefore most dangerous, in Southern California. Over time, these four faults produce about half of the significant earthquakes of our region. It should be noted that the San Andreas is a right-lateral fault.

However, this is not the whole picture. Unlike Central and Northern California, much of the plate movement in Southern California is not parallel to the San Andreas fault. Between the southern end of the San Joaquin Valley and the San Bernardino mountains, in the so-called "big bend," the San Andreas fault runs in a more westerly direction.

Where the fault bends, plate motion is complex. The Pacific and North American plates push into each other, compressing the earth's crust into the mountains of Southern California and producing faults and earthquakes. While these 300 or so faults are generally much shorter and slower moving than the four faults mentioned previously, over half of the significant earthquakes in Southern California occur on these faults.

The greatest concentration of these faults is in and near the mountains that have formed around the big bend of the San Andreas fault (the San Bernardino, San Gabriel, and Santa Ynez mountains). These mountains, like most mountains in California, are there because earthquakes are pushing them up. Many of these faults can be seen at the earth's surface, though some are buried beneath the sediments of the Los Angeles basin and the inland valleys.

2.6 SEISMIC SEA WAVES

When an earthquake occurs at the ocean floor, the movement of the curst of the earth up and down creates waves known as seismic sea waves, tidal waves or Tsunami (in Japanese). It should be noted that tsunami is caused by the normal faults (dip-slip) and reverse faults (thrust faults)

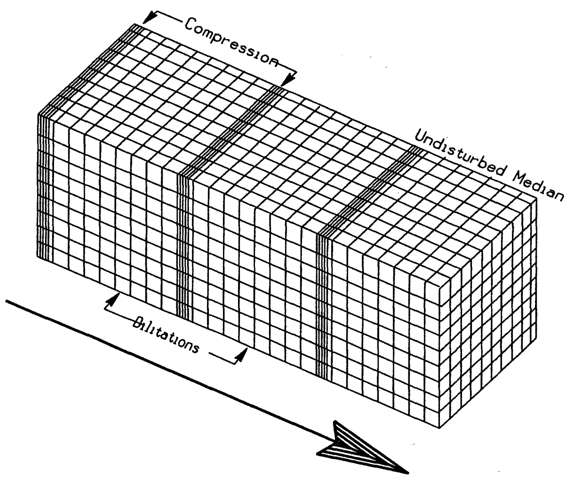
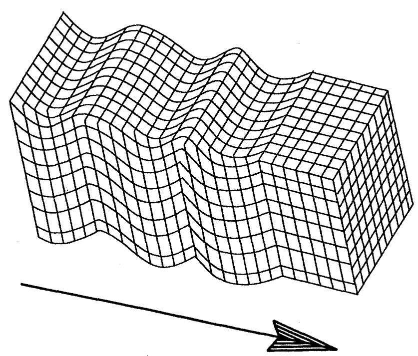
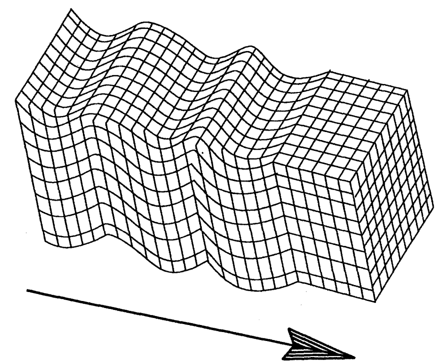
Tsunamis are long-wavelength, long-period sea waves generated by an abrupt movement of large volumes of water. In the open ocean, the distance between wave crests can be greater than 100 kilometers, and the wave periods can vary from five minutes to one hour. Such tsunamis travel 600-800 kilometers per hour, depending on water depth. Large subduction earthquakes causing vertical displacement of the sea floor and having magnitudes greater than 7.5 are the most common cause of destructive tsunamis.

Large waves produced by an earthquake or a submarine landslide can overrun nearby coastal areas in a matter of minutes. Tsunamis can also travel thousands of kilometers across open ocean and wreak destruction on far shores hours after the earthquake that generated them.

Tsunami wave heights at sea are usually less than one meter, and the waves are not frequently noticed by people in ships. As tsunami waves approach the shallow water of the coast, their heights increase and sometimes exceed 20 meters.

2.5 SEISIMIC WAVES

Table 2-3 Types of Seismic Waves

Compression Waves (P Waves)	**Shear Waves (S waves)**	**Surface Waves (Love waves)**
➢ Known as longitudinal waves	➢ Known as the transverse waves. <u>They are responsible for the strong ground motion and cause the majority of damage to structures.</u>	➢ Known as Rayleigh waves or L-waves
➢ Reach the surface first	➢ Reach the surface after the P-waves	➢ They arrive after the P & S waves.
➢ Can pass through the earth's molten core	➢ Cannot pass through the earth's molten core.	➢ Cannot pass through the earth's molten core.
➢ Travel at great speed (about 19000 ft/sec in granite)  	➢ Travel at less speed (about 10,000 ft/sec in granite 	➢ Travel at less speed compared to P & S waves (about 9,000 ft/sec in granite) 

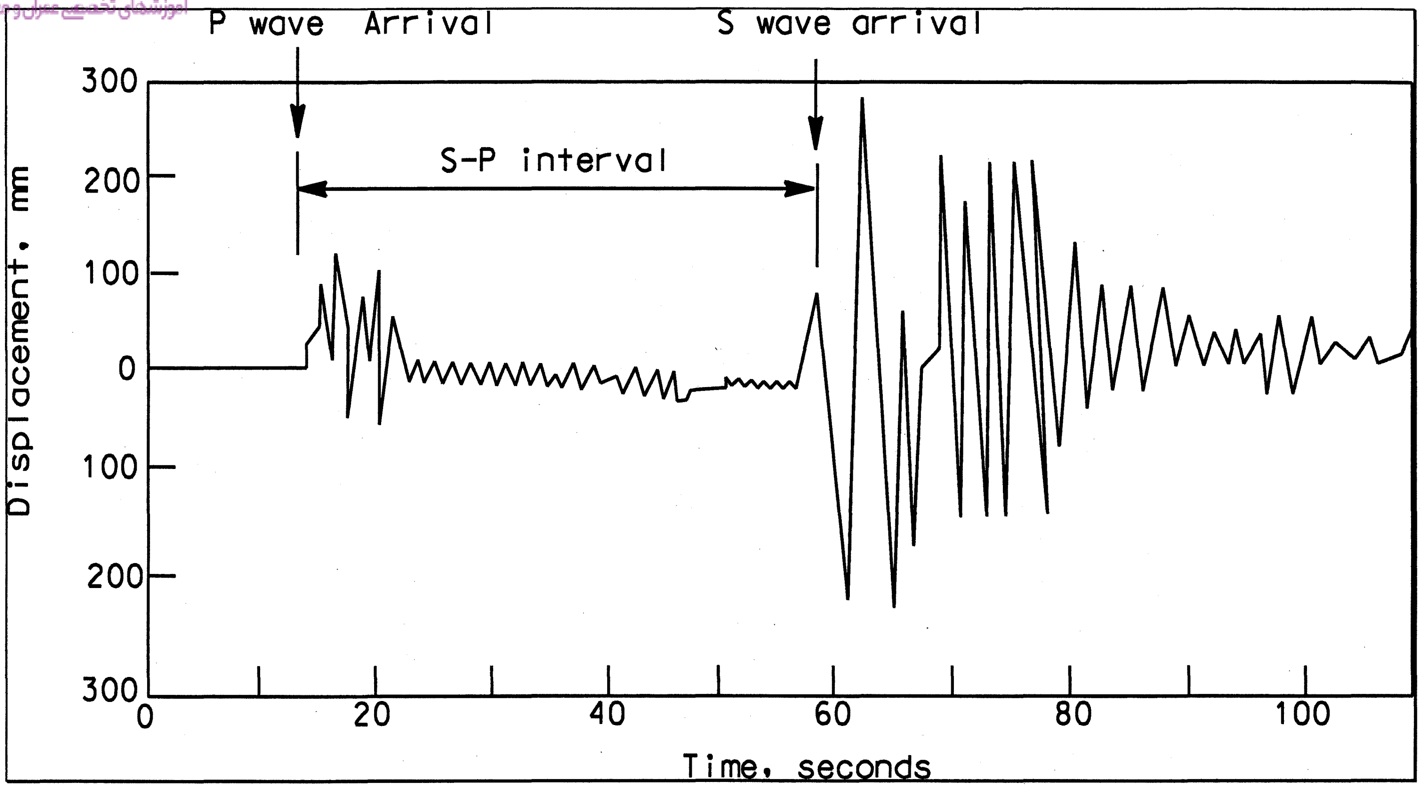


Figure 2.6 Typical Seismogram Recording

See CH4-50

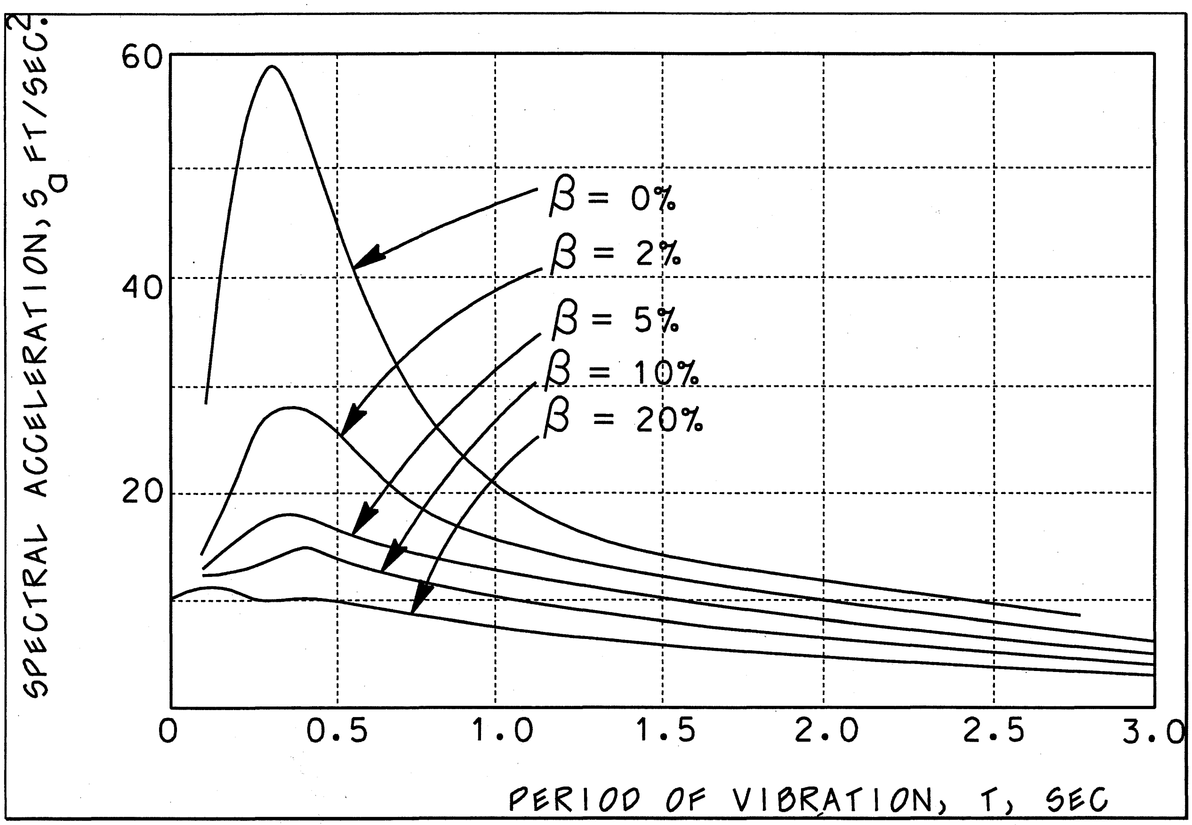


Figure 2.7 Average Elastic Response Spectra with Different Damping

2.7 EARTHQUAKE MEASUREMENTS

There are measurements for earthquakes:

A- Intensity (based on the damage and how people felt)
B- Magnitude (based on the total strain energy released)

A-Intensity:

A subjective numerical index describing the effects of an earthquake on humans, on their structures, and on the earth's surface at a particular place. The number is rated on the basis of an earthquake intensity scale. The scale commonly use in the U.S. today is the Modified Mercalli (MM) Intensity Scale of 1931 with intensities indicated by Roman numerals from I to XII. In general, for a given earthquake, intensity will decrease with distance from the epicenter. The following is abridgement of the scale:

I. Not felt except by a very few under especially favorable conditions.

II. Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing.

III. Felt quite noticeably indoors, especially on upper floors of buildings, but many people do not recognize it as an earthquake. Standing automobiles may rock slightly. Vibration like passing of truck. Duration can be estimated.

IV. During the day felt indoors by many, outdoors by few. At night some awakened. Dishes, windows, doors disturbed; walls make cracking sound. Sensation like heavy truck striking building. Standing automobiles rocked noticeably.

V. Felt by nearly everyone; many awakened. Some dishes, windows, and other fragile items broken; a few instances of cracked plaster; unstable objects overturned. Disturbance of frees, poles and other tall objects sometimes noticed. Pendulum clocks may stop.

VI. Felt by all; many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight.

VII. Everybody runs outdoors. Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable in poorly built or badly designed structures. Some chimneys broken. Noticed by persons driving automobiles.

VIII. Damage slight in specially designed structures; considerable in ordinary substantial buildings with partial collapse. Great damage in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Persons driving automobiles disturbed.

IX. Damage considerable in specially designed structures; well-designed frame structures thrown out-of-plumb; damage great in substantial buildings, with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken.

X. Some well-built wooden structures destroyed; most masonry and frame structures destroyed. Ground badly cracked. Railroad rails bent. Many landslides on river banks and steep slopes. Shifted sand and mud. Water splashed over banks of rivers and lakes.

XI. Few structures remain standing; Unreinforced masonry structures are nearly totally destroyed. Bridges destroyed. Broad fissures in ground. Underground pipe lines completely out of service. Earth slumps and land slips in soft ground. Railroad rails bent greatly.

XII. Damage total; Waves apparently seen on ground surfaces. Lines of sight and level appear visually distorted. Objects thrown upward into the air.

Table 2-4 Summary of Modified Mercalli Scale

MM Scale	I	II-III	IV	V	VI	VII	VIII	IX	X-XII
How Much Damage?	None	None	None	Very Light	Light	Moderate	Moderate to heavy	Heavy	Very Heavy
How People Felt?	Not Felt Except very Few	Weak	Light	Moderate	Strong	Very Strong	Severe	Violent	Extreme

B- Magnitude:

The magnitude is a number that characterizes the relative size of an earthquake. It is a measure of the size of an earthquake related to the total strain energy released. Magnitude is based on measurement of the maximum motion recorded by a seismograph, corrected for attenuation to a standardized distance.

The following formula was developed by Professor Charles F. Richter of the California Institute of Technology in 1935.

$$M = \log\left(\frac{A}{A_0}\right) \qquad (2-1)$$

Where:

$A_0 =$ is the seismometer reading produced by an earthquake of standard size (a calibration earthquake). Normally, A_0 is 3.94 x 10^{-5} in or 0.001 mm.

$A =$ The maximum amplitude of the seismometer trace.

The above equation is based on an distance of 100 km (62 mi) between the seismometer and the epicenter. Empirical charts and tables are available to correct to an epicentral distance of

100 km for other types of seismographs and for various conditions of the ground. The correction charts are suitable up to epicentral distances of about 600 km. The correction charts are site dependent and have to be developed for each recording site.

The amount of energy released from an earthquake is given by the following formula:

$$Log_E = 11.8 + 1.5M \qquad (2-2)$$

$$E = 10^{11.8+1.5M} \qquad (2-3)$$

As can be seen from the above logarithmic equation that an increase in magnitude of ONE would radiate about 32 times more energy. The following table shows the different magnitudes as used by the United States Geological Service (USGS):

Table 2-4 USGS Classification of Earthquakes

Magnitude (M)	Earthquake
<3	Micro
3- 3.9	Minor
4- 4.9	Light
5- 5.9	Moderate
6- 6.9	Strong
7-7.9	Major
>8	Great

Another equation (known as recurrence formula) which calculates the expected number (N) of earthquakes having a magnitude grater than (M) during (Y) years is given as follows:

$$N = CYe^{-M/B} \qquad (2-4)$$

Where C and B are 19,700 and 0.463 respectively for the San Francisco area.

The probability $p\{M\}$ that an earthquake of magnitude M or greater will occur in a specific region is given by the following equation which is derived from the application of Poisson's distribution to calculate the probability of an infrequent event:

$$p\{M\} = e^{-M/B} \qquad (2-5)$$

Where B is a parameter, estimated to be 2.1 for all of California.

2.8 SAMPLE PROBLEMS

Sample Problem 2-1: Earthquake Terminology

Given: The epicenter of an earthquake is:

A) The same as the focus of the earthquake
B) Located at the point of maximum structural damage
C) Located on the surface of the earth above the focus of the earthquake.
D) None of the above.

Solution:
See Figure 2.1 on page 2-1. Epicenter is the point located on the surface of the earth immediately above the hypocenter (focus)

Answer: C ⇐

Sample Problem 2-2: Seismic Waves

Given: A type of wave <u>not</u> generated by a seismic event is:

A) P Wave B) S Wave
C) Q wave D) Love wave

Solution:
See Table 2.1 on page 2-4. The three seismic waves are generated by a seismic event are: Primary wave (P), Shear wave (S) and Love wave. Q wave is not generated by a seismic event

Answer: C ⇐

Sample Problem 2-3: Earthquakes

Given: Subduction is best defined by which of the following?

A) The energy released when one crustal plate suddenly against an adjacent plate.
B) The sliding of a crustal plate beneath an adjacent plate
C) The magma film beneath plates that allow them to move with little friction
D) Another name of lithosphere

Solution:
See page 2-9 "Three basic types of plate interactions"

Answer: B ⇐

Sample Problem 2-4: Types of Earthquakes

Given: Most damage is associated with which kind of earthquake?

A) Deep focus
B) Intermediate focus
C) Shallow focus
D) Both "a" and "b"

Solution: Answer: C ⇐

Sample Problem 2-5: Faults

Given: Dip-slip and strike-slip are terms related to which of the following?

A) The angle of the epicenter from focus
B) The amount of energy released during an earthquake
C) The inclination and direction of an earthquake fault
D) All of the above

Solution: See Figure 2.1 Answer: C ⇐

Sample Problem 2-6: Earthquake Magnitude

Given: A Richter magnitude earthquake of 4.5 represents how much of an increased in measured amplitude over a 2.5 magnitude earthquake?

A) 10 times
B) 100 times
C) 200 times
D) 1000 times

Solution:

$$M = \log\left(\frac{A}{A_0}\right) \qquad (2-1)$$

$$A_{4.5} = 10^{M} \times 10^{-3} = 10^{4.5} \times 10^{-3}$$

$$A_{2.5} = 10^{M} \times 10^{-3} = 10^{2.5} \times 10^{-3}$$

$$\frac{A_{4.5}}{A_{2.5}} = 10^{4.5-2.5} = 10^{2} = 100$$

Answer: B ⇐

<u>**Note :**</u> The change in the amplitude is $\Delta A = (10)^{\Delta M}$

Sample Problem 2-7: Earthquake Magnitude

Given: A Richter magnitude scale 5.5 earthquake would radiate how much more energy than a 3.5 magnitude earthquake?

A) 32 times
B) 10 times
C) 100 times
D) 1000 times

$$Log_E = 11.8 + 1.5M \qquad (2-2)$$

$$E = 10^{11.8+1.5M}, \quad E_{5.5} = 10^{11.8+1.5\times5.5} = 10^{20.05} \;\&\; E_{3.5} = 10^{11.8+1.5\times3.5} = 10^{17.05}$$

$$E_{5.5}/E_{3.5} = 10^{20.05-17.05} = 10^3 = 1000$$

Answer: D ⇐

Note : The change in the energy is $\Delta E = (31.62)^{\Delta M} \approx (32)^{\Delta M}$

Sample Problem 2-8: Earthquakes Scale

Given: The scale most commonly used to describe the intensity of an earthquake is:

A) Richter magnitude scale
B) Modified Mercalli scale
C) Logarithmic scale
D) Both "a" and "b"

Solution: Answer: B ⇐

Sample Problem 2-9: Earthquakes Scale

Given: What is the Modified Mercalli Scale Intensity of an earthquake which is felt by those in the upper stories of buildings but is imperceptible to those outside?

A) III B) IV C) V D) VI

Solution: Answer: A ⇐

Sample Problem 2-10: Earthquakes

Given: The expected number of earthquakes having a magnitude greater than M6.0 during a 100- year period in the San Francisco area is most nearly:

A) 3 B) 5 C) 7 D) 9

Solution:

$$N = CYe^{-M/B} \qquad (2-3)$$

$$N = CYe^{-M/B} = (19{,}700)(100)(e^{-6/0.463}) = 4.63$$

Answer: B ⇐

Sample Problem 2-11: Seismic Waves

Given: Which seismic wave is the most responsible for the strong ground motion of an earthquake?

A) P-Wave B) S- Wave C) Surface wave D) Love wave

Solution:

Answer: B ⇐

Sample Problem 2-12: Earthquakes

Given: The largest earthquake ground motion that is expected to occur sometimes in the life of a structure to be built at a specific site is the:

A) Maximum probable earthquake
B) Maximum considered earthquake
C) Maximum predictable earthquake
D) Maximum theoretical earthquake

Solution:

Answer: C ⇐

Sample Problem 2-13: Faults

Given: There is a footwall facing a hanging wall prior to an earthquake. The hanging wall moved up during an earthquake. Which type of fault occurred?

A) Strike-slip fault B) Reverse fault C) Normal fault D) Lateral fault

Solution:

Answer: B ⇐

Sample Problem 2-14: Seismic Sea Waves

Given: As seismic sea waves approach land, which of the following statement is correct?

A) Wave height increases
B) Wave velocity increases
C) Wave velocity decreases and wave height increases
D) Wave velocity and wave height increase

Solution:

As the sea waves approach land, the velocity decrease because of the increase of friction between the waves and the shallow sea bed. At the same time the wave height increases.

Answer: C ⇐

Sample Problem 2-15: Earthquakes

Given: The San Andreas fault of California is which type of fault?

A) Right –lateral fault
B) Left – lateral fault
C) Normal fault
D) Reverse fault

Solution:

Answer: A⇐

Sample Problem 2-16: Seismometers

Given: A seismometer measures which of the following?

A) Velocity and acceleration of the ground
B) Displacement of the crust of the earth
C) Energy released from an earthquake
D) Attenuation

Solution:

A seismometer is an instrument that measures the <u>displacement of the crust of the earth</u>. Normally this is done by selecting a reference point to be as a datum for such measurements.
Answer A: seismometer does not measure velocity or acceleration
Answer C: energy released can be calculated from the magnitude "M" Eq. (2-2)
Answer D: Attenuation is a decrease in amplitude (and hence energy) of the seismic waves with distance from the source.

Answer: B⇐

Sample Problem 2-17: Attenuation

Given: Attenuation is not affected by which of the following:

A) Energy released from an earthquake
B) Focal depth
C) Properties of the soil
D) Length and path line of the fault

Solution:

Attenuation is defined as <u>decrease in amplitude (and hence energy) of the seismic waves with distance from the source</u>. Focal depth, properties of the soil and length of the fault ALL are affecting attenuation. The only given parameter that does no affect attenuation is the energy released from an earthquake.

Answer: A ⇐

Chapter 3

Theory of Vibration, Stiffness and Rigidity

Topics to be covered

- Simple Harmonic Motion
- Single Degree of Freedom Systems
- Natural Period & Frequency
- Damping and Damping Ratio
- Resonance
- Stiffness, Flexibility & Rigidity

8 Practice Problems

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Chapter 3- Vibration Theory, Stiffness and Rigidity

3.1 INTRODUCTION

The analysis and design of structures to resist the effect produced by time dependent forces or motion; as earthquakes forces require simplifying assumptions and conceptual idealizations through which the actual system is represented by an idealized system known as analytical or mathematical model. These assumptions are as follows:

i) Loading assumptions: these assumptions are related to the point of application of the load and the way the load is applied on the system (suddenly, constant or periodic).

ii) Material assumptions: to simplify the analytical or mathematical mode, the material could be assumed homogenous, linear, or isotropic.

iii) Geometric assumptions: beams, frames and trusses could be considered unidirectional elements. Also, three dimensional structures or element could be assumed as two dimensional.

The current Codes (2006 IBC/ 2007 CBC & ASCE 7- 05) have three procedures to calculate the seismic force for a structure, nonbuilding structures or an element or component subjected to an earthquake force. These three methods are:

1- Dynamic Analysis Procedure: The building characteristics as weight, stiffness, and rigidity are calculated. A mathematical model of the structure which consists of masses lumped at certain locations over the structure is subjected to an artificial earthquake (mechanical shaker) (see § Section 12.9 ASCE 7-05). The behavior and response of the structure to this artificial ground motion is used to establish a response spectra (time-acceleration relationship). This method is costly and time consuming, but it is powerful, accurate and it is required for certain structures as high rise buildings, certain irregular structures and important structures such as nuclear power plants and dams.

The dynamic analysis procedure can be accomplished by:

i) Modal Response Spectrum Analysis - §12.9 ASCE 7-05

ii) Seismic Response History Procedures- Chapter 16- ASCE 7-05

2- Equivalent Lateral Force (ELF) Procedure: This approach is not as rigorous as the dynamic procedure and it is appropriate for many structures. This equivalent lateral force (ELF) procedure calculates the seismic force according to a set of equations using a generic design response spectra (see Figure11.4-1 & § 12.8 ASCE 7- 05). These equations estimates the seismic lateral force (earthquake force) as a percentage (about 20%) of the seismic dead load of the structure. For purposes of determining seismic loads, it is permitted to consider the structure to be fixed at the base.

3- Simplified Lateral Force Analysis Procedure: The simplified design procedure is permitted to be used in lieu of other analytical procedures in ASCE 7-05 Chapter12 (e.g. Dynamic Analysis, Equivalent Lateral Procedure) for the analysis and design of simple buildings with bearing wall or building frame systems, subject to all of the 12 limitations listed in Section §12.14.8.1 For purposes of determining seismic loads, it is permitted to consider the structure to be fixed at the base.

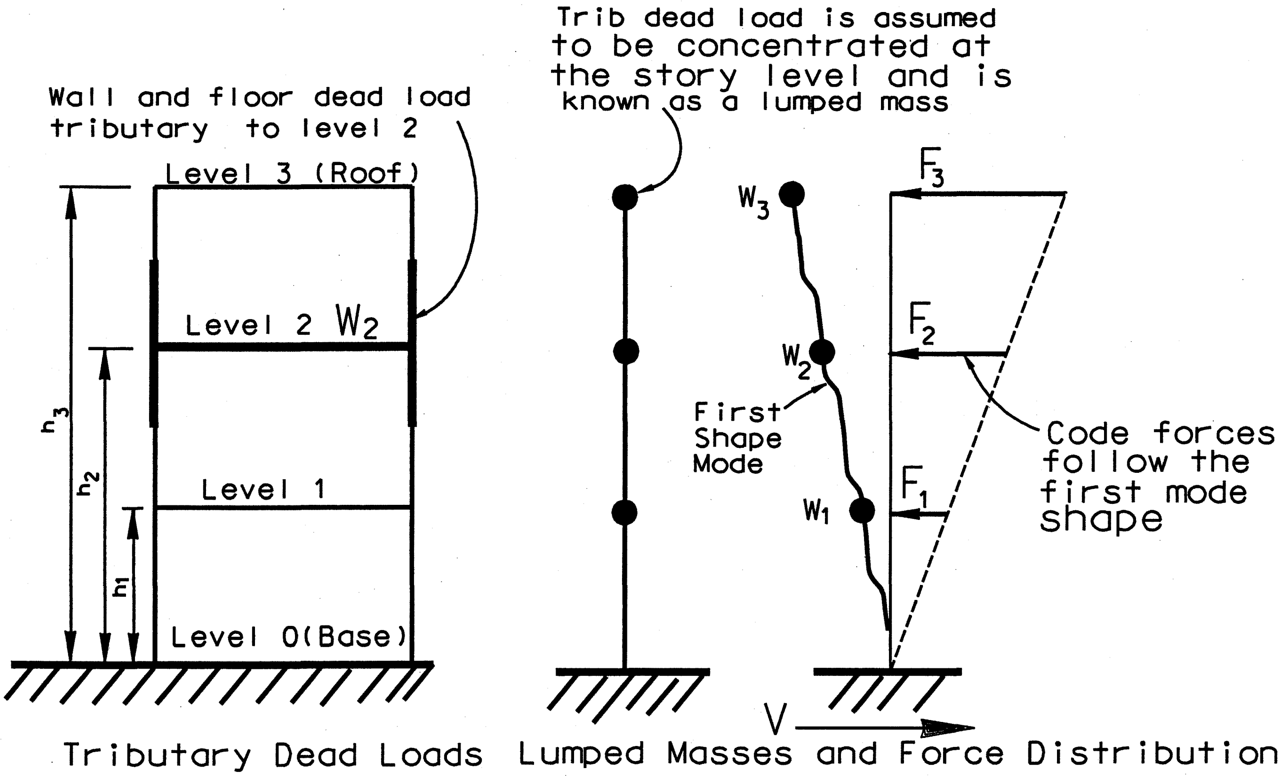


Figure 3-1 Lumped Masses of a Structure

3.2 SIMPLE HARMONIC MOTION

When the spring mass system shown, which is not subjected to an external force, vibrates under the effect of an initial displacement, it is known as the simple harmonic motion. This free vibration (no damping) under the initial displacement can be expressed in a form of relationship between time (t) and displacement (x) as shown in Figure 3-3

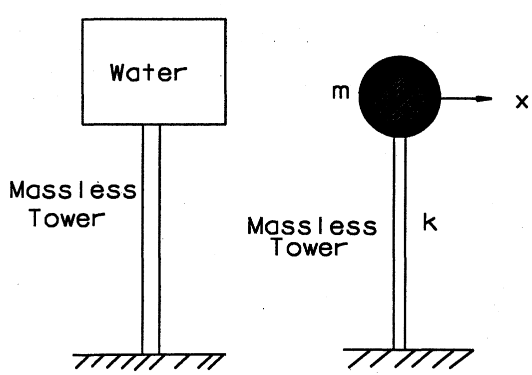


Figure 3-2 Idealized Structure as Simple Harmonic Oscillator

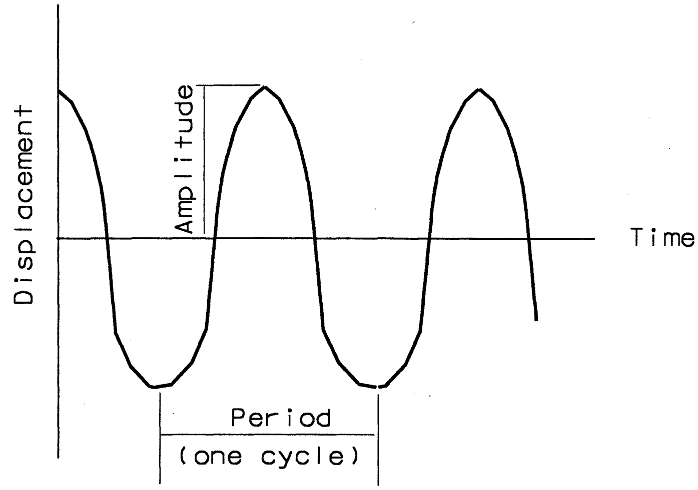


Figure 3-3 Time-Displacement Relationship for Simple Harmonic Motion

3.3 SINGLE DEGREE OF FREEDOM (SDOF)

The number of independent coordinates necessary to specify the configuration or position of a system at any time is refereed to as the number of degrees of freedom. The following figure shows structures that can be idealized as a single degree of freedom.

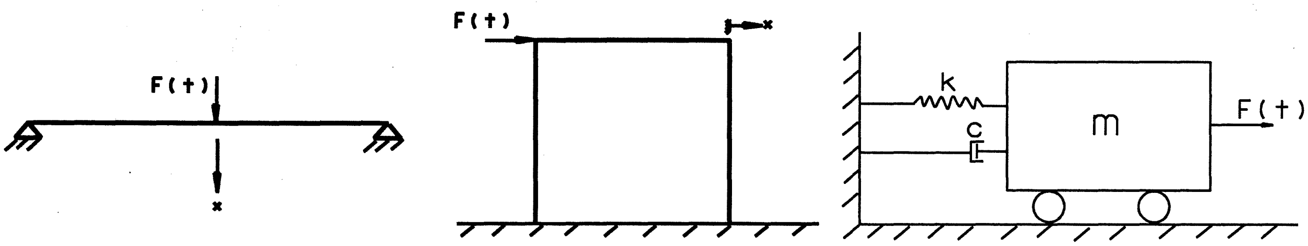


Figure 3-4 Structures modeled as one-degree-of-freedom (SDOF) systems

3.4 NATURAL PERIOD AND FREQUENCY

- Natural Period, T : is the time for a complete cycle of oscillation of a single degree of freedom system. The following Equation (3-1) calculates the natural period of a single degree of freedom system (SDOF) in terms of the stiffness (K), the mass (m) and the gravity acceleration (g).

$$T=2\pi\sqrt{\frac{W}{g\,K}}=2\pi\sqrt{\frac{m}{K}} \qquad (3-1)$$

- Natural Frequency, f : is the reciprocal of the natural period and is expressed in hertz (Hz).

$$f=\frac{1}{T}=\frac{1}{2\pi}\sqrt{\frac{K\,g}{W}} \qquad (3-2)$$

- Angular Natural Frequency, ω: is given by the following formula which calculates angular natural frequency or the circular frequency or angular frequency (ω) and is expressed in radians per second (rad/sec) :

$$\omega = 2\pi f = \frac{2\pi}{T} = \sqrt{\frac{K}{m}} = \sqrt{\frac{P g}{(\Delta)(W)}} \qquad (3-3)$$

Where:

P = Applied Force (load)

W = Weight of the Structure

g = Gravity Acceleration = 32.2 ft/sec^2 = 386.4 in/sec^2 = 9.81 m/sec^2

K = Stiffness

m = Mass

Δ = Static Deflection

- Springs in Parallel or in Series: In order to determine the equivalent spring constant for a system that has two or more springs arranged in a parallel or series as shown below, the following two equations will be used:

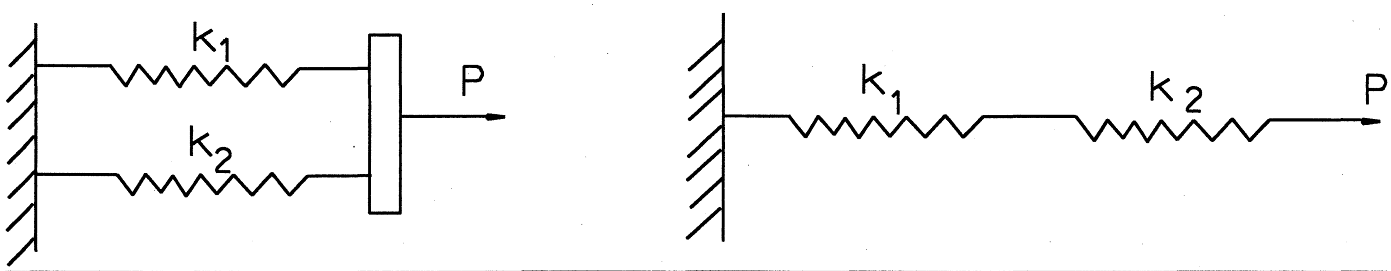


Figure 3-5 Springs in Parallel (left) or in Series (right)

The following equation calculates the total stiffness (spring constant) of a system having springs or columns that are connected in parallel:

$$K_t = K_e = K_1 + K_2 + \ldots\ldots + K_n \qquad (3-4)$$

The following equation calculates the total stiffness (spring constant) of a system having springs or columns that are connected in series:

$$\frac{1}{K_e} = \frac{1}{K_1} + \frac{1}{K_2} \ldots\ldots\ldots + \frac{1}{K_n} \qquad (3-5)$$

3.5 DAMPING AND DAMPING RATIO

Damping is the reduction in the amplitude of a seismic wave or oscillator due to friction and /or the internal absorption of energy. Simply it is the dissipation of energy mainly by friction. There is more than one source for damping. External damping and internal damping are among those sources.

For the internal viscous damping, the frictional damping force opposing the motion is given by the following formula:

$$F_{damping} = BV^n \qquad (3-6)$$

where "*B*" is the damping coefficient and "*n*" is a factor between one and two depending on the system.

Because of the damping, eventually the system will come to a stop (equilibrium position) after some time. It may take many cycles before the system reaches the equilibrium. However, there is one particular amount of damping that will bring the system to equilibrium in a minimum time. This is known as critical damping and the coefficient "*B*" will be "$B_{critical}$"

The ratio between the damping coefficient and the critical damping is known as damping ratio.

$$\xi = \frac{B}{B_{critical}} \qquad (3-7)$$

The following table shows approximate (exact values are difficult to evaluate) values for damping ratios for some buildings.

Table 3-1 Damping Ratio for Building Structures

Type	Damping Ratio ξ
Steel Frame	0.02 - 0.10
Concrete Frame	0.05 - 0.10
Concrete/Masonry shear Walls	0.10
Wood Frame	0.15

3.6 RESONANCE

Vibrations are generally classified as either free vibrations or forced vibrations. Free vibrations occur in the absence of externally applied force. The impetus for the free vibration is usually an initial displacement and/or velocity imparted to the mass. A system undergoing free vibration will oscillate one or more of its natural frequencies. A "SDOF" system has only one natural frequency.

Forced vibration occurs under the excitation of externally applied forces. If the excitation is transient (i.e. of short duration), the system response is at its natural frequency (once the disturbance terminates). However, if the excitation is oscillatory (periodically repetitive) and continues with time, the system vibrates at the excitation frequency. *In situations where the excitation frequency coincides with the natural frequency of the system a condition known as resonance*. At resonance, the amplitude of vibration becomes extremely large, and damage to the system is imminent if the vibration continues at the resonant frequency.

In Earthquake Design, when the natural period of a structure coincides with the period of the site (soil), the structure's response can be greatly amplified due to resonance where the amplitude becomes large.

Because the increase of the amplitude is due to resonance, the damage to structures becomes huge. Two examples will be discussed to illustrate the effect of resonance on the magnitude of the structure's damage.

Example #1: The 1985 Mexico City magnitude 8.1 earthquake with a 7.5 aftershock occurring the next day. About 400 buildings were destroyed, and 750 were damaged. It was concluded that the natural period of the ground coincided with the period for buildings taller than 8 stories. Taller buildings have higher natural periods compared to short buildings, and therefore they are more receptive to the resonance phenomenon. The resulting resonance-related yielding was the primary cause of structural failure.

Example #2: The collapse of the Oakland Interstate 880 Cypress structure during the October 17, 1989 Loma Prieta earthquake was also attributed to resonance. The structure had a natural frequency of 2 Hz to 4 Hz, which coincided with the 3 Hz to 5 Hz natural period of the deep mud that underlaid piles that supported the freeway that collapsed. The depth of the mud and the length of the piles varied between 20 ft to 80 ft.

3.7 STIFFNESS, FLEXIBILITY & RIGIDITY

- Stiffness, k: Force required to produce a unit displacement. When a linear spring (obeying Hooke's Law) is subjected to a force "P", the displacement that will take place can be expressed as:

$$P = k\Delta \qquad (3-8)$$

Where:

k = Spring stiffness (lb/ft, N/m)

Δ = Linear Displacement (deflection)

- Flexibility (compliance): Displacement (deflection) required to produce a unit force. Basically, it is the reciprocal of stiffness and therefore has units of ft/lb or m/N.

- Rigidity, R: The reciprocal of deflection

$$R = 1/\Delta \qquad (3-9)$$

It should be noted that the rigidity "R" is not the same as the response modification coefficient "R" listed in Table 12.2-1 of the ASCE 7-05

3.8 DEFLECTION & RGIDITY OF CANTILEVER PIERS AND WALLS WITHOUT OPENINGS

The deflection of a cantilever beam due to bending and shear is given by the following equation:

$$\Delta_C = \frac{Ph^3}{3EI} + \frac{1.2Ph}{AG} \qquad (3-10)$$

Where: $I = \frac{td^3}{12}$, $A = td$, $G = \frac{E}{2(1+\nu)} \cong 0.4E$

$$\Delta_C = \frac{P}{Et}\left[4\left(\frac{h}{d}\right)^3 + 3\left(\frac{h}{d}\right)\right] \qquad (3-11)$$

Since we are interested in the relative rigidity of the walls because the base shear is distributed among the resisting walls in proportion to their rigidities (rigid diaphragm), assumed values for P and E & t can be used. P = 100 kips, E = 1,000,000 psi & t = 1" were used. This will yield the following equation:

$$\Delta_C = 0.4\left(\frac{h}{d}\right)^3 + 0.3\left(\frac{h}{d}\right) \qquad (3-12)$$

Finally, the rigidity is the reciprocal of deflection, therefore:

$$R_C = \frac{1}{\Delta_C} \qquad (3-13)$$

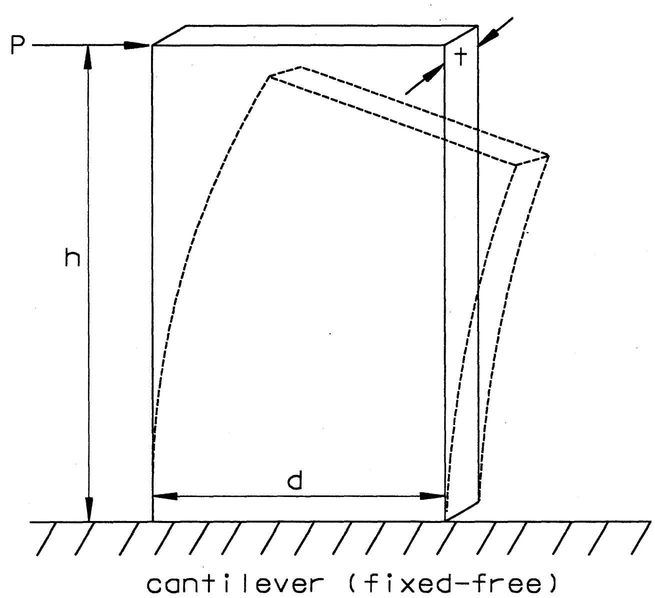


Figure 3-6 Cantilever (Fixed-Free) Piers and Walls Without Openings

3.9 DEFLECTION & RGIDITY OF FIXED PIERS AND WALLS WITHOUT OPENINGS

The deflection of a fixed beam at both ends due to bending and shear is given by the following equation:

$$\Delta_F = \frac{Ph^3}{12EI} + \frac{1.2Ph}{AG} \qquad (3-14)$$

Where:

$$I = \frac{td^3}{12}, \quad A = td, \quad G = \frac{E}{2(1+\nu)} \cong 0.4E$$

$$\Delta_F = \frac{P}{Et}\left[\left(\frac{h}{d}\right)^3 + 3\left(\frac{h}{d}\right)\right] \qquad (3-15)$$

Since we are interested in the relative rigidity of the walls because the base shear is distributed among the resisting walls in proportion to their rigidities (rigid diaphragm), assumed values for P and E & t can be used. P = 100 kips, E = 1,000,000 psi & t = 1" were used. This will yield the following equation:

$$\Delta_F = 0.1\left(\frac{h}{d}\right)^3 + 0.3\left(\frac{h}{d}\right) \qquad (3-16)$$

Finally, the rigidity is the reciprocal of deflection, therefore:

$$R_F = \frac{1}{\Delta_F} \qquad (3-17)$$

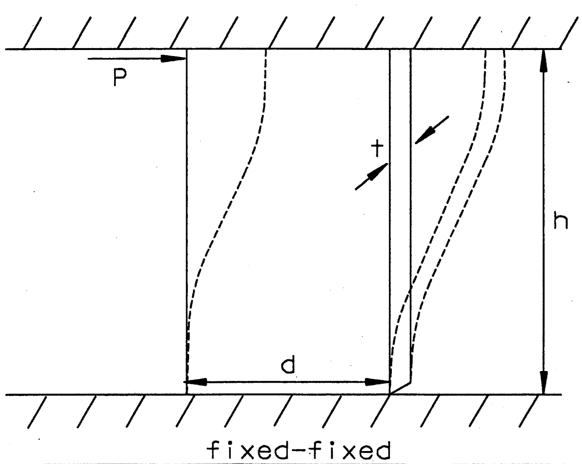


Figure 3-7 Fixed-Fixed Piers and Walls Without Openings

Series of Tables (Table 3-3 & 3-4) are prepared for both cases (fixed and cantilever) for determining the deflection and the rigidity.

3.10 DEFLECTION AND STIFFNESS FOR COULMNS & BRACING SYSTEMS

Table 3-2 Deflection & Stiffness For Columns & Bracing Systems

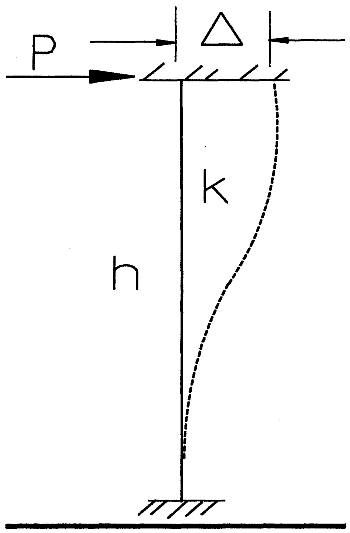
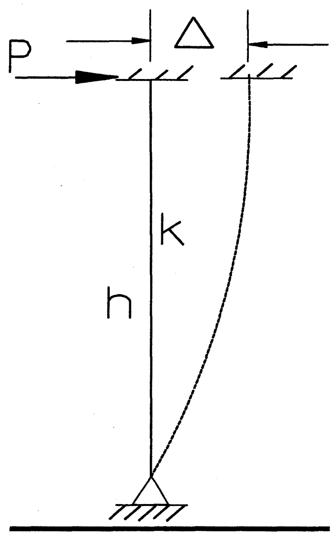
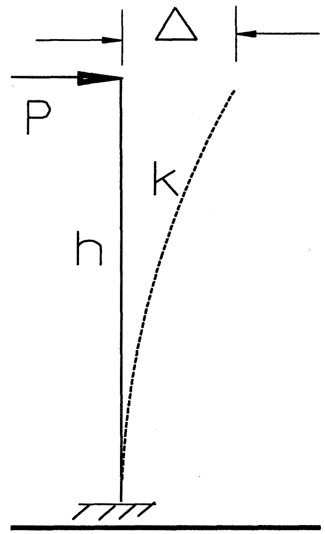
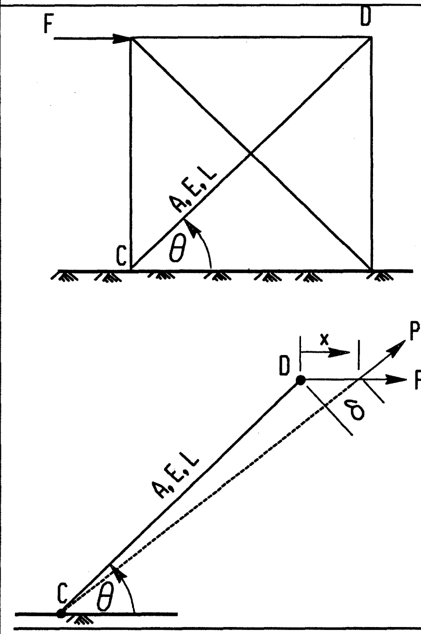
Type of Column	Fixed-Fixed	Hinged-Fixed	Fixed-Free	Brace
Deflected & Undeflected shapes				
Deflection	$\Delta = \dfrac{Ph^3}{12\,EI}$	$\Delta = \dfrac{Ph^3}{3\,EI}$	$\Delta = \dfrac{Ph^3}{3\,EI}$	$\delta = \dfrac{PL}{AE}$
Stiffness	$K = \dfrac{12EI}{h^3}$	$K = \dfrac{3EI}{h^3}$	$K = \dfrac{3EI}{h^3}$	$K_{Brace} = \dfrac{EA}{L} Cos^2\theta$

Table 3-3 Relative Deflection and Rigidity for Cantilever Piers and Walls Without Opening
(P= 100 kips, E= 1,000,000 psi & t= 1”)

h/d	Δ_c	R_c	*h/d*	Δ_c	R_c	*h/d*	Δ_c	R_c	*h/d*	Δ_c	R_c	*h/d*	Δ_c	R_c	*h/d*	Δ_c	R_c	*h/d*	Δ_c	R_c
0.10	0.03	32.895	**0.36**	0.13	7.895	**0.62**	0.28	3.555	**0.88**	0.537	1.864	**1.14**	0.93	1.070	**1.4**	1.518	0.659	**1.66**	2.328	0.430
0.11	0.03	29.822	**0.37**	0.13	7.618	**0.63**	0.29	3.460	**0.89**	0.549	1.822	**1.15**	0.95	1.049	**1.41**	1.544	0.648	**1.67**	2.364	0.423
0.12	0.04	27.254	**0.38**	0.14	7.356	**0.64**	0.30	3.369	**0.90**	0.562	1.781	**1.16**	0.97	1.028	**1.42**	1.571	0.636	**1.68**	2.401	0.417
0.13	0.04	25.076	**0.39**	0.14	7.106	**0.65**	0.30	3.280	**0.91**	0.574	1.741	**1.17**	0.99	1.008	**1.43**	1.599	0.626	**1.69**	2.438	0.410
0.14	0.04	23.203	**0.4**	0.15	6.868	**0.66**	0.31	3.195	**0.92**	0.587	1.702	**1.18**	1.01	0.989	**1.44**	1.626	0.615	**1.70**	2.475	0.404
0.15	0.05	21.575	**0.41**	0.15	6.641	**0.67**	0.32	3.112	**0.93**	0.601	1.665	**1.19**	1.03	0.970	**1.45**	1.654	0.604	**1.71**	2.513	0.398
0.16	0.05	20.146	**0.42**	0.16	6.425	**0.68**	0.33	3.032	**0.94**	0.614	1.628	**1.20**	1.05	0.951	**1.46**	1.683	0.594	**1.72**	2.551	0.392
0.17	0.05	18.880	**0.43**	0.16	6.219	**0.69**	0.34	2.955	**0.95**	0.628	1.592	**1.21**	1.07	0.933	**1.47**	1.712	0.584	**1.73**	2.590	0.386
0.18	0.06	17.752	**0.44**	0.17	6.021	**0.7**	0.35	2.880	**0.96**	0.642	1.558	**1.22**	1.09	0.915	**1.48**	1.741	0.574	**1.74**	2.629	0.380
0.19	0.06	16.738	**0.45**	0.17	5.833	**0.71**	0.36	2.808	**0.97**	0.656	1.524	**1.23**	1.11	0.898	**1.49**	1.770	0.565	**1.75**	2.669	0.375
0.20	0.06	15.823	**0.46**	0.18	5.652	**0.72**	0.37	2.737	**0.98**	0.670	1.491	**1.24**	1.13	0.881	**1.50**	1.800	0.556	**1.76**	2.709	0.369
0.21	0.07	14.992	**0.47**	0.18	5.479	**0.73**	0.37	2.669	**0.99**	0.685	1.460	**1.25**	1.16	0.865	**1.51**	1.830	0.546	**1.77**	2.749	0.364
0.22	0.07	14.233	**0.48**	0.19	5.312	**0.74**	0.38	2.604	**1.00**	0.700	1.429	**1.26**	1.18	0.849	**1.52**	1.861	0.537	**1.78**	2.790	0.358
0.23	0.07	13.538	**0.49**	0.19	5.153	**0.75**	0.39	2.540	**1.01**	0.715	1.398	**1.27**	1.20	0.833	**1.53**	1.892	0.529	**1.79**	2.831	0.353
0.24	0.08	12.898	**0.5**	0.20	5.000	**0.76**	0.40	2.478	**1.02**	0.730	1.369	**1.28**	1.22	0.818	**1.54**	1.923	0.520	**1.80**	2.873	0.348
0.25	0.08	12.308	**0.51**	0.21	4.853	**0.77**	0.41	2.418	**1.03**	0.746	1.340	**1.29**	1.25	0.803	**1.55**	1.955	0.512	**1.81**	2.915	0.343
0.26	0.09	11.760	**0.52**	0.21	4.712	**0.78**	0.42	2.359	**1.04**	0.762	1.312	**1.30**	1.27	0.788	**1.56**	1.987	0.503	**1.82**	2.957	0.338
0.27	0.09	11.252	**0.53**	0.22	4.576	**0.79**	0.43	2.303	**1.05**	0.778	1.285	**1.31**	1.29	0.774	**1.57**	2.019	0.495	**1.83**	3.000	0.333
0.28	0.09	10.778	**0.54**	0.22	4.445	**0.8**	0.44	2.248	**1.06**	0.794	1.259	**1.32**	1.32	0.760	**1.58**	2.052	0.487	**1.84**	3.044	0.329
0.29	0.10	10.335	**0.55**	0.23	4.319	**0.81**	0.46	2.195	**1.07**	0.811	1.233	**1.33**	1.34	0.746	**1.59**	2.085	0.480	**1.85**	3.088	0.324
0.30	0.10	9.921	**0.56**	0.24	4.197	**0.82**	0.47	2.143	**1.08**	0.828	1.208	**1.34**	1.36	0.733	**1.60**	2.118	0.472	**1.86**	3.132	0.319
0.31	0.10	9.531	**0.57**	0.25	4.080	**0.83**	0.48	2.093	**1.09**	0.845	1.183	**1.35**	1.39	0.720	**1.61**	2.152	0.465	**1.87**	3.177	0.315
0.32	0.11	9.165	**0.58**	0.25	3.968	**0.84**	0.49	2.045	**1.10**	0.862	1.160	**1.36**	1.41	0.707	**1.62**	2.187	0.457	**1.88**	3.222	0.310
0.33	0.11	8.820	**0.59**	0.26	3.859	**0.85**	0.50	1.997	**1.11**	0.880	1.136	**1.37**	1.44	0.695	**1.63**	2.221	0.450	**1.89**	3.268	0.306
0.34	0.12	8.495	**0.6**	0.27	3.754	**0.86**	0.51	1.952	**1.12**	0.898	1.114	**1.38**	1.47	0.682	**1.64**	2.256	0.443	**1.90**	3.314	0.302
0.35	0.12	8.187	**0.61**	0.27	3.652	**0.87**	0.52	1.907	**1.13**	0.92	1.09	**1.39**	1.49	0.671	**1.65**	2.292	0.436	**1.91**	3.360	0.298

Table 3-3 (Cont.) Relative Deflection and Rigidity for Cantilever Piers and Walls Without Opening (P= 100 kips, E= 1,000,000 psi & t= 1")

h/d	Δ_c	R_c	*h/d*	Δ_c	R_c	*h/d*	Δ_c	R_c	*h/d*	Δ_c	R_c	*h/d*	Δ_c	R_c	*h/d*	Δ_c	R_c	*h/d*	Δ_c	R_c
1.92	3.41	0.293	**2.18**	4.80	0.208	**2.44**	6.54	0.153	**2.70**	8.68	0.12	**2.96**	11.26	0.089	**3.22**	14.32	0.070	**3.47**	17.75	0.056
1.93	3.45	0.289	**2.19**	4.86	0.206	**2.45**	6.62	0.151	**2.71**	8.77	0.11	**2.97**	11.37	0.088	**3.23**	14.45	0.069	**3.48**	17.90	0.056
1.94	3.50	0.286	**2.2**	4.92	0.203	**2.46**	6.69	0.149	**2.72**	8.87	0.11	**2.98**	11.48	0.087	**3.24**	14.58	0.069	**3.49**	18.05	0.055
1.95	3.55	0.282	**2.21**	4.98	0.201	**2.47**	6.77	0.148	**2.73**	8.96	0.11	**2.99**	11.59	0.086	**3.25**	14.71	0.068	**3.50**	18.20	0.055
1.96	3.60	0.278	**2.22**	5.04	0.198	**2.48**	6.85	0.146	**2.74**	9.05	0.11	**3.00**	11.70	0.085	**3.26**	14.84	0.067	**3.51**	18.35	0.054
1.97	3.65	0.274	**2.23**	5.10	0.196	**2.49**	6.92	0.144	**2.75**	9.14	0.11	**3.01**	11.81	0.085	**3.27**	14.97	0.067	**3.52**	18.50	0.054
1.98	3.70	0.270	**2.24**	5.17	0.194	**2.5**	7.00	0.143	**2.76**	9.24	0.11	**3.02**	11.92	0.084	**3.28**	15.10	0.066	**3.53**	18.65	0.054
1.99	3.75	0.267	**2.25**	5.23	0.191	**2.51**	7.08	0.141	**2.77**	9.33	0.11	**3.03**	12.04	0.083	**3.29**	15.23	0.066	**3.54**	18.81	0.053
2.00	3.80	0.263	**2.26**	5.30	0.189	**2.52**	7.16	0.140	**2.78**	9.43	0.11	**3.04**	12.15	0.082	**3.30**	15.36	0.065	**3.55**	18.96	0.053
2.01	3.85	0.260	**2.27**	5.36	0.187	**2.53**	7.24	0.138	**2.79**	9.52	0.10	**3.05**	12.26	0.082	**3.31**	15.50	0.065	**3.56**	19.12	0.052
2.02	3.90	0.256	**2.28**	5.42	0.184	**2.54**	7.32	0.137	**2.80**	9.62	0.10	**3.06**	12.38	0.081	**3.32**	15.63	0.064	**3.57**	19.27	0.052
2.03	3.96	0.253	**2.29**	5.49	0.182	**2.55**	7.40	0.135	**2.81**	9.72	0.10	**3.07**	12.49	0.080	**3.33**	15.77	0.063	**3.58**	19.43	0.051
2.04	4.01	0.250	**2.3**	5.56	0.180	**2.56**	7.48	0.134	**2.82**	9.82	0.10	**3.08**	12.61	0.079	**3.34**	15.91	0.063	**3.59**	19.58	0.051
2.05	4.06	0.246	**2.31**	5.62	0.178	**2.57**	7.56	0.132	**2.83**	9.92	0.10	**3.09**	12.73	0.079	**3.35**	16.04	0.062	**3.60**	19.74	0.051
2.06	4.11	0.243	**2.32**	5.69	0.176	**2.58**	7.64	0.131	**2.84**	10.01	0.10	**3.10**	12.85	0.078	**3.36**	16.18	0.062	**3.61**	19.90	0.050
2.07	4.17	0.240	**2.33**	5.76	0.174	**2.59**	7.73	0.129	**2.85**	10.11	0.10	**3.11**	12.97	0.077	**3.37**	16.32	0.061	**3.62**	20.06	0.050
2.08	4.22	0.237	**2.34**	5.83	0.172	**2.6**	7.81	0.128	**2.86**	10.22	0.10	**3.12**	13.08	0.076	**3.38**	16.46	0.061	**3.63**	20.22	0.049
2.09	4.28	0.234	**2.35**	5.90	0.170	**2.61**	7.89	0.127	**2.87**	10.32	0.10	**3.13**	13.20	0.076	**3.39**	16.60	0.060	**3.64**	20.38	0.049
2.10	4.33	0.231	**2.36**	5.97	0.168	**2.62**	7.98	0.125	**2.88**	10.42	0.10	**3.14**	13.33	0.075	**3.40**	16.74	0.060	**3.65**	20.55	0.049
2.11	4.39	0.228	**2.37**	6.04	0.166	**2.63**	8.07	0.124	**2.89**	10.52	0.10	**3.15**	13.45	0.074	**3.41**	16.88	0.059	**3.66**	20.71	0.048
2.12	4.45	0.225	**2.38**	6.11	0.164	**2.64**	8.15	0.123	**2.90**	10.63	0.09	**3.16**	13.57	0.074	**3.42**	17.03	0.059	**3.67**	20.87	0.048
2.13	4.50	0.222	**2.39**	6.18	0.162	**2.65**	8.24	0.121	**2.91**	10.73	0.09	**3.17**	13.69	0.073	**3.43**	17.17	0.058	**3.68**	21.04	0.048
2.14	4.56	0.219	**2.4**	6.25	0.160	**2.66**	8.33	0.120	**2.92**	10.83	0.09	**3.18**	13.82	0.072	**3.44**	17.32	0.058	**3.69**	21.20	0.047
2.15	4.62	0.216	**2.41**	6.32	0.158	**2.67**	8.41	0.119	**2.93**	10.94	0.09	**3.19**	13.94	0.072	**3.45**	17.46	0.057	**3.70**	21.37	0.047
2.16	4.68	0.214	**2.42**	6.39	0.156	**2.68**	8.50	0.118	**2.94**	11.05	0.09	**3.20**	14.07	0.071	**3.46**	17.61	0.057	**3.71**	21.54	0.046
2.17	4.74	0.211	**2.43**	6.47	0.155	**2.69**	8.59	0.116	**2.95**	11.15	0.09	**3.21**	14.19	0.070	**3.47**	17.75	0.056	**3.72**	21.71	0.046

Table 3-3 (Cont.) Relative Deflection and Rigidity for Cantilever Piers and Walls Without Opening (P= 100 kips, E= 1,000,000 psi & t= 1")

h/d	Δc	R_c	*h/d*	Δc	R_c	*h/d*	Δc	R_c	*h/d*	Δc	R_c	*h/d*	Δc	R_c	*h/d*	Δc	R_c	*h/d*	Δc	R_c
3.73	21.88	0.046	**3.99**	26.61	0.038	**4.25**	31.98	0.031	**4.51**	38.05	0.03	**4.77**	44.84	0.022	**5.03**	52.414	0.019	**5.29**	60.801	0.016
3.74	22.05	0.045	**4**	26.80	0.037	**4.26**	32.20	0.031	**4.52**	38.29	0.03	**4.78**	45.12	0.022	**5.04**	52.722	0.019	**5.30**	61.141	0.016
3.75	22.22	0.045	**4.01**	27.00	0.037	**4.27**	32.42	0.031	**4.53**	38.54	0.03	**4.79**	45.40	0.022	**5.05**	53.030	0.019	**5.31**	61.482	0.016
3.76	22.39	0.045	**4.02**	27.19	0.037	**4.28**	32.65	0.031	**4.54**	38.79	0.03	**4.80**	45.68	0.022	**5.06**	53.340	0.019	**5.32**	61.824	0.016
3.77	22.56	0.044	**4.03**	27.39	0.037	**4.29**	32.87	0.030	**4.55**	39.04	0.03	**4.81**	45.96	0.022	**5.07**	53.651	0.019	**5.33**	62.167	0.016
3.78	22.74	0.044	**4.04**	27.59	0.036	**4.3**	33.09	0.030	**4.56**	39.30	0.03	**4.82**	46.24	0.022	**5.08**	53.963	0.019	**5.34**	62.511	0.016
3.79	22.91	0.044	**4.05**	27.79	0.036	**4.31**	33.32	0.030	**4.57**	39.55	0.03	**4.83**	46.52	0.021	**5.09**	54.276	0.018	**5.35**	62.857	0.016
3.80	23.09	0.043	**4.06**	27.99	0.036	**4.32**	33.54	0.030	**4.58**	39.80	0.03	**4.84**	46.80	0.021	**5.10**	54.590	0.018	**5.36**	63.204	0.016
3.81	23.27	0.043	**4.07**	28.19	0.035	**4.33**	33.77	0.030	**4.59**	40.06	0.02	**4.85**	47.09	0.021	**5.11**	54.906	0.018	**5.37**	63.553	0.016
3.82	23.44	0.043	**4.08**	28.39	0.035	**4.34**	34.00	0.029	**4.60**	40.31	0.02	**4.86**	47.37	0.021	**5.12**	55.223	0.018	**5.38**	63.902	0.016
3.83	23.62	0.042	**4.09**	28.59	0.035	**4.35**	34.23	0.029	**4.61**	40.57	0.02	**4.87**	47.66	0.021	**5.13**	55.541	0.018	**5.39**	64.253	0.016
3.84	23.80	0.042	**4.1**	28.80	0.035	**4.36**	34.46	0.029	**4.62**	40.83	0.02	**4.88**	47.95	0.021	**5.14**	55.861	0.018	**5.40**	64.606	0.015
3.85	23.98	0.042	**4.11**	29.00	0.034	**4.37**	34.69	0.029	**4.63**	41.09	0.02	**4.89**	48.24	0.021	**5.15**	56.181	0.018	**5.41**	64.959	0.015
3.86	24.16	0.041	**4.12**	29.21	0.034	**4.38**	34.93	0.029	**4.64**	41.35	0.02	**4.90**	48.53	0.021	**5.16**	56.503	0.018	**5.42**	65.314	0.015
3.87	24.35	0.041	**4.13**	29.42	0.034	**4.39**	35.16	0.028	**4.65**	41.61	0.02	**4.91**	48.82	0.020	**5.17**	56.826	0.018	**5.43**	65.670	0.015
3.88	24.53	0.041	**4.14**	29.63	0.034	**4.4**	35.39	0.028	**4.66**	41.88	0.02	**4.92**	49.11	0.020	**5.18**	57.151	0.017	**5.44**	66.028	0.015
3.89	24.71	0.040	**4.15**	29.83	0.034	**4.41**	35.63	0.028	**4.67**	42.14	0.02	**4.93**	49.41	0.020	**5.19**	57.476	0.017	**5.45**	66.386	0.015
3.90	24.90	0.040	**4.16**	30.04	0.033	**4.42**	35.87	0.028	**4.68**	42.41	0.02	**4.94**	49.70	0.020	**5.20**	57.803	0.017	**5.46**	66.747	0.015
3.91	25.08	0.040	**4.17**	30.26	0.033	**4.43**	36.10	0.028	**4.69**	42.67	0.02	**4.95**	50.00	0.020	**5.21**	58.131	0.017	**5.47**	67.108	0.015
3.92	25.27	0.040	**4.18**	30.47	0.033	**4.44**	36.34	0.028	**4.70**	42.94	0.02	**4.96**	50.30	0.020	**5.22**	58.461	0.017	**5.48**	67.471	0.015
3.93	25.46	0.039	**4.19**	30.68	0.033	**4.45**	36.58	0.027	**4.71**	43.21	0.02	**4.97**	50.60	0.020	**5.23**	58.791	0.017	**5.49**	67.835	0.015
3.94	25.65	0.039	**4.2**	30.90	0.032	**4.46**	36.82	0.027	**4.72**	43.48	0.02	**4.98**	50.90	0.020	**5.24**	59.123	0.017	**5.50**	68.200	0.015
3.95	25.84	0.039	**4.21**	31.11	0.032	**4.47**	37.07	0.027	**4.73**	43.75	0.02	**4.99**	51.20	0.020	**5.25**	59.456	0.017	**5.51**	68.567	0.015
3.96	26.03	0.038	**4.22**	31.33	0.032	**4.48**	37.31	0.027	**4.74**	44.02	0.02	**5.00**	51.50	0.019	**5.26**	59.791	0.017	**5.52**	68.935	0.015
3.97	26.22	0.038	**4.23**	31.54	0.032	**4.49**	37.55	0.027	**4.75**	44.29	0.02	**5.01**	51.80	0.019	**5.27**	60.126	0.017	**5.53**	69.304	0.014
3.98	26.41	0.038	**4.24**	31.76	0.031	**4.5**	37.80	0.026	**4.76**	44.57	0.02	**5.02**	52.11	0.019	**5.28**	60.463	0.017	**5.54**	69.675	0.014

Table 3-3 (Cont.) Relative Deflection and Rigidity for Cantilever Piers and Walls Without Opening (P= 100 kips, E= 1,000,000 psi & t= 1")

h/d	Δ_c	R_c	*h/d*	Δ_c	R_c	*h/d*	Δ_c	R_c	*h/d*	Δ_c	R_c	*h/d*	Δ_c	R_c	*h/d*	Δ_c	R_c	*h/d*	Δ_c	R_c
5.55	70.05	0.014	**5.81**	80.19	0.012	**6.06**	90.84	0.011	**6.32**	102.87	0.01	**6.59**	116.45	0.009	**6.85**	130.623	0.008	**7.11**	145.903	0.007
5.56	70.42	0.014	**5.82**	80.60	0.012	**6.07**	91.28	0.011	**6.33**	103.35	0.01	**6.60**	116.98	0.009	**6.86**	131.190	0.008	**7.12**	146.514	0.007
5.57	70.79	0.014	**5.83**	81.01	0.012	**6.08**	91.73	0.011	**6.34**	103.84	0.01	**6.61**	117.50	0.009	**6.87**	131.758	0.008	**7.13**	147.126	0.007
5.58	71.17	0.014	**5.84**	81.42	0.012	**6.09**	92.17	0.011	**6.35**	104.32	0.01	**6.62**	118.03	0.008	**6.88**	132.328	0.008	**7.14**	147.740	0.007
5.59	71.55	0.014	**5.85**	81.84	0.012	**6.1**	92.62	0.011	**6.36**	104.81	0.01	**6.63**	118.56	0.008	**6.89**	132.900	0.008	**7.15**	148.355	0.007
5.60	71.93	0.014	**5.86**	82.25	0.012	**6.11**	93.07	0.011	**6.37**	105.30	0.01	**6.64**	119.09	0.008	**6.90**	133.474	0.007	**7.16**	148.973	0.007
5.61	72.31	0.014	**5.87**	82.67	0.012	**6.12**	93.52	0.011	**6.38**	105.79	0.01	**6.65**	119.63	0.008	**6.91**	134.049	0.007	**7.17**	149.592	0.007
5.62	72.69	0.014	**5.88**	83.08	0.012	**6.13**	93.98	0.011	**6.39**	106.28	0.01	**6.66**	120.16	0.008	**6.92**	134.626	0.007	**7.18**	150.212	0.007
5.63	73.07	0.014	**5.89**	83.50	0.012	**6.14**	94.43	0.011	**6.40**	106.78	0.01	**6.67**	120.70	0.008	**6.93**	135.204	0.007	**7.19**	150.835	0.007
5.64	73.45	0.014	**5.9**	83.92	0.012	**6.15**	94.89	0.011	**6.41**	107.27	0.01	**6.68**	121.24	0.008	**6.94**	135.784	0.007	**7.20**	151.459	0.007
5.65	73.84	0.014	**5.91**	84.34	0.012	**6.16**	95.35	0.010	**6.42**	107.77	0.01	**6.69**	121.77	0.008	**6.95**	136.366	0.007	**7.21**	152.085	0.007
5.66	74.23	0.013	**5.92**	84.77	0.012	**6.17**	95.81	0.010	**6.43**	108.27	0.01	**6.70**	122.32	0.008	**6.96**	136.949	0.007	**7.22**	152.713	0.007
5.67	74.61	0.013	**5.93**	85.19	0.012	**6.18**	96.27	0.010	**6.44**	108.77	0.01	**6.71**	122.86	0.008	**6.97**	137.535	0.007	**7.23**	153.342	0.007
5.68	75.00	0.013	**5.94**	85.62	0.012	**6.19**	96.73	0.010	**6.45**	109.27	0.01	**6.72**	123.40	0.008	**6.98**	138.121	0.007	**7.24**	153.973	0.006
5.69	75.40	0.013	**5.95**	86.04	0.012	**6.2**	97.19	0.010	**6.46**	109.77	0.01	**6.73**	123.95	0.008	**6.99**	138.710	0.007	**7.25**	154.606	0.006
5.70	75.79	0.013	**5.96**	86.47	0.012	**6.21**	97.66	0.010	**6.47**	110.28	0.01	**6.74**	124.49	0.008	**7.00**	139.300	0.007	**7.26**	155.241	0.006
5.71	76.18	0.013	**5.97**	86.90	0.012	**6.22**	98.12	0.010	**6.48**	110.78	0.01	**6.75**	125.04	0.008	**7.01**	139.892	0.007	**7.27**	155.877	0.006
5.72	76.58	0.013	**5.98**	87.33	0.011	**6.23**	98.59	0.010	**6.49**	111.29	0.01	**6.76**	125.59	0.008	**7.02**	140.485	0.007	**7.28**	156.515	0.006
5.73	76.97	0.013	**5.99**	87.77	0.011	**6.24**	99.06	0.010	**6.50**	111.80	0.01	**6.77**	126.15	0.008	**7.03**	141.081	0.007	**7.29**	157.155	0.006
5.74	77.37	0.013	**6.00**	88.20	0.011	**6.25**	99.53	0.010	**6.51**	112.31	0.01	**6.78**	126.70	0.008	**7.04**	141.677	0.007	**7.30**	157.797	0.006
5.75	77.77	0.013	**6.01**	88.64	0.011	**6.26**	100.00	0.010	**6.52**	112.82	0.01	**6.79**	127.26	0.008	**7.05**	142.276	0.007	**7.31**	158.440	0.006
5.76	78.17	0.013	**6.02**	89.07	0.011	**6.27**	100.48	0.010	**6.53**	113.34	0.01	**6.80**	127.81	0.008	**7.06**	142.876	0.007	**7.32**	159.085	0.006
5.77	78.57	0.013	**6.03**	89.51	0.011	**6.28**	100.95	0.010	**6.54**	113.85	0.01	**6.81**	128.37	0.008	**7.07**	143.478	0.007	**7.33**	159.732	0.006
5.78	78.97	0.013	**6.04**	89.95	0.011	**6.29**	101.43	0.010	**6.55**	114.37	0.01	**6.82**	128.93	0.008	**7.08**	144.082	0.007	**7.34**	160.381	0.006
5.79	79.38	0.013	**6.05**	90.39	0.011	**6.3**	101.91	0.010	**6.56**	114.89	0.01	**6.83**	129.49	0.008	**7.09**	144.687	0.007	**7.35**	161.031	0.006
5.80	79.78	0.013	**6.06**	90.84	0.011	**6.31**	102.39	0.010	**6.57**	115.41	0.01	**6.84**	130.06	0.008	**7.10**	145.294	0.007	**7.36**	161.683	0.006

Table 3-4 Relative Deflection and Rigidity for Fixed Piers and Walls Without Opening
(P= 100 kips, E= 1,000,000 psi & t= 1")

h/d	Δ_F	R_F	**h/d**	Δ_F	R_F	**h/d**	Δ_F	R_F	**h/d**	Δ_F	R_F	**h/d**	Δ_F	R_F	**h/d**	Δ_F	R_F	**h/d**	Δ_F	R_F
0.10	0.03	33.223	**0.36**	0.11	8.876	**0.62**	0.21	4.766	**0.88**	0.33	3.01	**1.14**	0.49	2.040	**1.4**	0.694	1.440	**1.66**	0.955	1.047
0.11	0.03	30.181	**0.37**	0.12	8.616	**0.63**	0.21	4.673	**0.89**	0.34	2.96	**1.15**	0.50	2.012	**1.41**	0.703	1.422	**1.67**	0.967	1.034
0.12	0.04	27.645	**0.38**	0.12	8.369	**0.64**	0.22	4.583	**0.90**	0.34	2.92	**1.16**	0.50	1.984	**1.42**	0.712	1.404	**1.68**	0.978	1.022
0.13	0.04	25.497	**0.39**	0.12	8.135	**0.65**	0.22	4.495	**0.91**	0.35	2.87	**1.17**	0.51	1.956	**1.43**	0.721	1.386	**1.69**	0.990	1.010
0.14	0.04	23.655	**0.4**	0.13	7.911	**0.66**	0.23	4.410	**0.92**	0.35	2.83	**1.18**	0.52	1.929	**1.44**	0.731	1.369	**1.70**	1.001	0.999
0.15	0.05	22.057	**0.41**	0.13	7.699	**0.67**	0.23	4.328	**0.93**	0.36	2.78	**1.19**	0.53	1.903	**1.45**	0.740	1.352	**1.71**	1.013	0.987
0.16	0.05	20.657	**0.42**	0.13	7.496	**0.68**	0.24	4.247	**0.94**	0.37	2.74	**1.20**	0.53	1.877	**1.46**	0.749	1.335	**1.72**	1.025	0.976
0.17	0.05	19.421	**0.43**	0.14	7.302	**0.69**	0.24	4.169	**0.95**	0.37	2.70	**1.21**	0.54	1.851	**1.47**	0.759	1.318	**1.73**	1.037	0.965
0.18	0.05	18.321	**0.44**	0.14	7.117	**0.7**	0.24	4.093	**0.96**	0.38	2.66	**1.22**	0.55	1.826	**1.48**	0.768	1.302	**1.74**	1.049	0.953
0.19	0.06	17.335	**0.45**	0.14	6.939	**0.71**	0.25	4.019	**0.97**	0.38	2.62	**1.23**	0.56	1.802	**1.49**	0.778	1.286	**1.75**	1.061	0.943
0.20	0.06	16.447	**0.46**	0.15	6.769	**0.72**	0.25	3.948	**0.98**	0.39	2.58	**1.24**	0.56	1.777	**1.50**	0.788	1.270	**1.76**	1.073	0.932
0.21	0.06	15.643	**0.47**	0.15	6.606	**0.73**	0.26	3.877	**0.99**	0.39	2.54	**1.25**	0.57	1.753	**1.51**	0.797	1.254	**1.77**	1.086	0.921
0.22	0.07	14.911	**0.48**	0.16	6.449	**0.74**	0.26	3.809	**1.00**	0.40	2.50	**1.26**	0.58	1.730	**1.52**	0.807	1.239	**1.78**	1.098	0.911
0.23	0.07	14.242	**0.49**	0.16	6.299	**0.75**	0.27	3.743	**1.01**	0.41	2.46	**1.27**	0.59	1.707	**1.53**	0.817	1.224	**1.79**	1.111	0.900
0.24	0.07	13.627	**0.5**	0.16	6.154	**0.76**	0.27	3.678	**1.02**	0.41	2.43	**1.28**	0.59	1.684	**1.54**	0.827	1.209	**1.80**	1.123	0.890
0.25	0.08	13.061	**0.51**	0.17	6.014	**0.77**	0.28	3.615	**1.03**	0.42	2.39	**1.29**	0.60	1.662	**1.55**	0.837	1.194	**1.81**	1.136	0.880
0.26	0.08	12.538	**0.52**	0.17	5.880	**0.78**	0.28	3.553	**1.04**	0.42	2.36	**1.30**	0.61	1.640	**1.56**	0.848	1.180	**1.82**	1.149	0.870
0.27	0.08	12.053	**0.53**	0.17	5.751	**0.79**	0.29	3.493	**1.05**	0.43	2.32	**1.31**	0.62	1.619	**1.57**	0.858	1.166	**1.83**	1.162	0.861
0.28	0.09	11.602	**0.54**	0.18	5.626	**0.8**	0.29	3.434	**1.06**	0.44	2.29	**1.32**	0.63	1.597	**1.58**	0.868	1.152	**1.84**	1.175	0.851
0.29	0.09	11.181	**0.55**	0.18	5.505	**0.81**	0.30	3.377	**1.07**	0.44	2.25	**1.33**	0.63	1.577	**1.59**	0.879	1.138	**1.85**	1.188	0.842
0.30	0.09	10.787	**0.56**	0.19	5.389	**0.82**	0.30	3.321	**1.08**	0.45	2.22	**1.34**	0.64	1.556	**1.60**	0.890	1.124	**1.86**	1.201	0.832
0.31	0.10	10.419	**0.57**	0.19	5.277	**0.83**	0.31	3.266	**1.09**	0.46	2.19	**1.35**	0.65	1.536	**1.61**	0.900	1.111	**1.87**	1.215	0.823
0.32	0.10	10.073	**0.58**	0.19	5.168	**0.84**	0.31	3.213	**1.10**	0.46	2.16	**1.36**	0.66	1.516	**1.62**	0.911	1.098	**1.88**	1.228	0.814
0.33	0.10	9.747	**0.59**	0.20	5.062	**0.85**	0.32	3.160	**1.11**	0.47	2.13	**1.37**	0.67	1.497	**1.63**	0.922	1.085	**1.89**	1.242	0.805
0.34	0.11	9.440	**0.6**	0.20	4.960	**0.86**	0.32	3.109	**1.12**	0.48	2.10	**1.38**	0.68	1.478	**1.64**	0.933	1.072	**1.90**	1.256	0.796
0.35	0.11	9.150	**0.61**	0.21	4.861	**0.87**	0.33	3.060	**1.13**	0.48	2.07	**1.39**	0.69	1.459	**1.65**	0.944	1.059	**1.91**	1.270	0.788

Table 3-4 (Cont.) Relative Deflection and Rigidity for Fixed Piers and Walls Without Opening (P= 100 kips, E= 1,000,000 psi & t= 1")

h/d	Δ_F	R_F	***h/d***	Δ_F	R_F	***h/d***	Δ_F	R_F	***h/d***	Δ_F	R_F	***h/d***	Δ_F	R_F	***h/d***	Δ_F	R_F	***h/d***	Δ_F	R_F
1.92	1.28	0.779	**2.18**	1.69	0.592	**2.44**	2.18	0.458	**2.70**	2.78	0.36	**2.96**	3.48	0.287	**3.22**	4.305	0.232	**3.47**	5.219	0.192
1.93	1.30	0.770	**2.19**	1.71	0.586	**2.45**	2.21	0.453	**2.71**	2.80	0.36	**2.97**	3.51	0.285	**3.23**	4.339	0.230	**3.48**	5.258	0.190
1.94	1.31	0.762	**2.2**	1.72	0.580	**2.46**	2.23	0.449	**2.72**	2.83	0.35	**2.98**	3.54	0.282	**3.24**	4.373	0.229	**3.49**	5.298	0.189
1.95	1.33	0.754	**2.21**	1.74	0.574	**2.47**	2.25	0.445	**2.73**	2.85	0.35	**2.99**	3.57	0.280	**3.25**	4.408	0.227	**3.50**	5.338	0.187
1.96	1.34	0.746	**2.22**	1.76	0.568	**2.48**	2.27	0.441	**2.74**	2.88	0.35	**3.00**	3.60	0.278	**3.26**	4.443	0.225	**3.51**	5.377	0.186
1.97	1.36	0.738	**2.23**	1.78	0.562	**2.49**	2.29	0.437	**2.75**	2.90	0.34	**3.01**	3.63	0.275	**3.27**	4.478	0.223	**3.52**	5.417	0.185
1.98	1.37	0.730	**2.24**	1.80	0.557	**2.5**	2.31	0.432	**2.76**	2.93	0.34	**3.02**	3.66	0.273	**3.28**	4.513	0.222	**3.53**	5.458	0.183
1.99	1.39	0.722	**2.25**	1.81	0.551	**2.51**	2.33	0.428	**2.77**	2.96	0.34	**3.03**	3.69	0.271	**3.29**	4.548	0.220	**3.54**	5.498	0.182
2.00	1.40	0.714	**2.26**	1.83	0.546	**2.52**	2.36	0.424	**2.78**	2.98	0.34	**3.04**	3.72	0.269	**3.30**	4.584	0.218	**3.55**	5.539	0.181
2.01	1.42	0.707	**2.27**	1.85	0.540	**2.53**	2.38	0.420	**2.79**	3.01	0.33	**3.05**	3.75	0.267	**3.31**	4.619	0.216	**3.56**	5.580	0.179
2.02	1.43	0.699	**2.28**	1.87	0.535	**2.54**	2.40	0.417	**2.80**	3.04	0.33	**3.06**	3.78	0.264	**3.32**	4.655	0.215	**3.57**	5.621	0.178
2.03	1.45	0.692	**2.29**	1.89	0.530	**2.55**	2.42	0.413	**2.81**	3.06	0.33	**3.07**	3.81	0.262	**3.33**	4.692	0.213	**3.58**	5.662	0.177
2.04	1.46	0.684	**2.3**	1.91	0.524	**2.56**	2.45	0.409	**2.82**	3.09	0.32	**3.08**	3.85	0.260	**3.34**	4.728	0.212	**3.59**	5.704	0.175
2.05	1.48	0.677	**2.31**	1.93	0.519	**2.57**	2.47	0.405	**2.83**	3.12	0.32	**3.09**	3.88	0.258	**3.35**	4.765	0.210	**3.60**	5.746	0.174
2.06	1.49	0.670	**2.32**	1.94	0.514	**2.58**	2.49	0.401	**2.84**	3.14	0.32	**3.10**	3.91	0.256	**3.36**	4.801	0.208	**3.61**	5.788	0.173
2.07	1.51	0.663	**2.33**	1.96	0.509	**2.59**	2.51	0.398	**2.85**	3.17	0.32	**3.11**	3.94	0.254	**3.37**	4.838	0.207	**3.62**	5.830	0.172
2.08	1.52	0.656	**2.34**	1.98	0.504	**2.6**	2.54	0.394	**2.86**	3.20	0.31	**3.12**	3.97	0.252	**3.38**	4.875	0.205	**3.63**	5.872	0.170
2.09	1.54	0.649	**2.35**	2.00	0.499	**2.61**	2.56	0.390	**2.87**	3.22	0.31	**3.13**	4.01	0.250	**3.39**	4.913	0.204	**3.64**	5.915	0.169
2.10	1.56	0.643	**2.36**	2.02	0.494	**2.62**	2.58	0.387	**2.88**	3.25	0.31	**3.14**	4.04	0.248	**3.40**	4.950	0.202	**3.65**	5.958	0.168
2.11	1.57	0.636	**2.37**	2.04	0.490	**2.63**	2.61	0.383	**2.89**	3.28	0.30	**3.15**	4.07	0.246	**3.41**	4.988	0.200	**3.66**	6.001	0.167
2.12	1.59	0.629	**2.38**	2.06	0.485	**2.64**	2.63	0.380	**2.90**	3.31	0.30	**3.16**	4.10	0.244	**3.42**	5.026	0.199	**3.67**	6.044	0.165
2.13	1.61	0.623	**2.39**	2.08	0.480	**2.65**	2.66	0.377	**2.91**	3.34	0.30	**3.17**	4.14	0.242	**3.43**	5.064	0.197	**3.68**	6.088	0.164
2.14	1.62	0.617	**2.4**	2.10	0.476	**2.66**	2.68	0.373	**2.92**	3.37	0.30	**3.18**	4.17	0.240	**3.44**	5.103	0.196	**3.69**	6.131	0.163
2.15	1.64	0.610	**2.41**	2.12	0.471	**2.67**	2.70	0.370	**2.93**	3.39	0.29	**3.19**	4.20	0.238	**3.45**	5.141	0.195	**3.70**	6.175	0.162
2.16	1.66	0.604	**2.42**	2.14	0.467	**2.68**	2.73	0.366	**2.94**	3.42	0.29	**3.20**	4.24	0.236	**3.46**	5.180	0.193	**3.71**	6.219	0.161
2.17	1.67	0.598	**2.43**	2.16	0.462	**2.69**	2.75	0.363	**2.95**	3.45	0.29	**3.21**	4.27	0.234	**3.47**	5.219	0.192	**3.72**	6.264	0.160

Table 3-4 (Cont.) Relative Deflection and Rigidity for Fixed Piers and Walls Without Opening (P= 100 kips, E= 1,000,000 psi & t= 1")

h/d	Δ_F	R_F	**h/d**	Δ_F	R_F	**h/d**	Δ_F	R_F	**h/d**	Δ_F	R_F	**h/d**	Δ_F	R_F	**h/d**	Δ_F	R_F	**h/d**	Δ_F	R_F
3.73	6.31	0.159	**3.99**	7.55	0.132	**4.25**	8.95	0.112	**4.51**	10.53	0.09	**4.77**	12.28	0.081	**5.03**	14.235	0.070	**5.29**	16.391	0.061
3.74	6.35	0.157	**4**	7.60	0.132	**4.26**	9.01	0.111	**4.52**	10.59	0.09	**4.78**	12.36	0.081	**5.04**	14.314	0.070	**5.30**	16.478	0.061
3.75	6.40	0.156	**4.01**	7.65	0.131	**4.27**	9.07	0.110	**4.53**	10.65	0.09	**4.79**	12.43	0.080	**5.05**	14.394	0.069	**5.31**	16.565	0.060
3.76	6.44	0.155	**4.02**	7.70	0.130	**4.28**	9.12	0.110	**4.54**	10.72	0.09	**4.80**	12.50	0.080	**5.06**	14.473	0.069	**5.32**	16.653	0.060
3.77	6.49	0.154	**4.03**	7.75	0.129	**4.29**	9.18	0.109	**4.55**	10.78	0.09	**4.81**	12.57	0.080	**5.07**	14.553	0.069	**5.33**	16.741	0.060
3.78	6.54	0.153	**4.04**	7.81	0.128	**4.3**	9.24	0.108	**4.56**	10.85	0.09	**4.82**	12.64	0.079	**5.08**	14.634	0.068	**5.34**	16.829	0.059
3.79	6.58	0.152	**4.05**	7.86	0.127	**4.31**	9.30	0.108	**4.57**	10.92	0.09	**4.83**	12.72	0.079	**5.09**	14.714	0.068	**5.35**	16.918	0.059
3.80	6.63	0.151	**4.06**	7.91	0.126	**4.32**	9.36	0.107	**4.58**	10.98	0.09	**4.84**	12.79	0.078	**5.10**	14.795	0.068	**5.36**	17.007	0.059
3.81	6.67	0.150	**4.07**	7.96	0.126	**4.33**	9.42	0.106	**4.59**	11.05	0.09	**4.85**	12.86	0.078	**5.11**	14.876	0.067	**5.37**	17.096	0.058
3.82	6.72	0.149	**4.08**	8.02	0.125	**4.34**	9.48	0.106	**4.60**	11.11	0.09	**4.86**	12.94	0.077	**5.12**	14.958	0.067	**5.38**	17.186	0.058
3.83	6.77	0.148	**4.09**	8.07	0.124	**4.35**	9.54	0.105	**4.61**	11.18	0.09	**4.87**	13.01	0.077	**5.13**	15.040	0.066	**5.39**	17.276	0.058
3.84	6.81	0.147	**4.1**	8.12	0.123	**4.36**	9.60	0.104	**4.62**	11.25	0.09	**4.88**	13.09	0.076	**5.14**	15.122	0.066	**5.40**	17.366	0.058
3.85	6.86	0.146	**4.11**	8.18	0.122	**4.37**	9.66	0.104	**4.63**	11.31	0.09	**4.89**	13.16	0.076	**5.15**	15.204	0.066	**5.41**	17.457	0.057
3.86	6.91	0.145	**4.12**	8.23	0.122	**4.38**	9.72	0.103	**4.64**	11.38	0.09	**4.90**	13.23	0.076	**5.16**	15.287	0.065	**5.42**	17.548	0.057
3.87	6.96	0.144	**4.13**	8.28	0.121	**4.39**	9.78	0.102	**4.65**	11.45	0.09	**4.91**	13.31	0.075	**5.17**	15.370	0.065	**5.43**	17.639	0.057
3.88	7.01	0.143	**4.14**	8.34	0.120	**4.4**	9.84	0.102	**4.66**	11.52	0.09	**4.92**	13.39	0.075	**5.18**	15.453	0.065	**5.44**	17.731	0.056
3.89	7.05	0.142	**4.15**	8.39	0.119	**4.41**	9.90	0.101	**4.67**	11.59	0.09	**4.93**	13.46	0.074	**5.19**	15.537	0.064	**5.45**	17.823	0.056
3.90	7.10	0.141	**4.16**	8.45	0.118	**4.42**	9.96	0.100	**4.68**	11.65	0.09	**4.94**	13.54	0.074	**5.20**	15.621	0.064	**5.46**	17.915	0.056
3.91	7.15	0.140	**4.17**	8.50	0.118	**4.43**	10.02	0.100	**4.69**	11.72	0.09	**4.95**	13.61	0.073	**5.21**	15.705	0.064	**5.47**	18.008	0.056
3.92	7.20	0.139	**4.18**	8.56	0.117	**4.44**	10.08	0.099	**4.70**	11.79	0.08	**4.96**	13.69	0.073	**5.22**	15.790	0.063	**5.48**	18.101	0.055
3.93	7.25	0.138	**4.19**	8.61	0.116	**4.45**	10.15	0.099	**4.71**	11.86	0.08	**4.97**	13.77	0.073	**5.23**	15.875	0.063	**5.49**	18.194	0.055
3.94	7.30	0.137	**4.2**	8.67	0.115	**4.46**	10.21	0.098	**4.72**	11.93	0.08	**4.98**	13.84	0.072	**5.24**	15.960	0.063	**5.50**	18.288	0.055
3.95	7.35	0.136	**4.21**	8.72	0.115	**4.47**	10.27	0.097	**4.73**	12.00	0.08	**4.99**	13.92	0.072	**5.25**	16.045	0.062	**5.51**	18.381	0.054
3.96	7.40	0.135	**4.22**	8.78	0.114	**4.48**	10.34	0.097	**4.74**	12.07	0.08	**5.00**	14.00	0.071	**5.26**	16.131	0.062	**5.52**	18.476	0.054
3.97	7.45	0.134	**4.23**	8.84	0.113	**4.49**	10.40	0.096	**4.75**	12.14	0.08	**5.01**	14.08	0.071	**5.27**	16.217	0.062	**5.53**	18.570	0.054
3.98	7.50	0.133	**4.24**	8.89	0.112	**4.5**	10.46	0.096	**4.76**	12.21	0.08	**5.02**	14.16	0.071	**5.28**	16.304	0.061	**5.54**	18.665	0.054

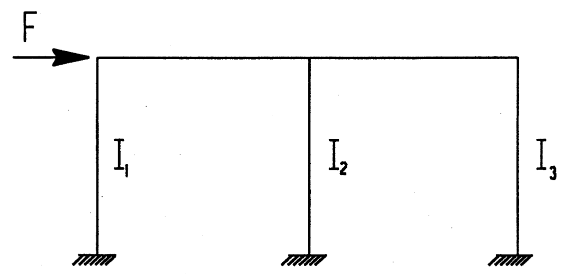
Table 3-4 (Cont.) Relative Deflection and Rigidity for Fixed Piers and Walls Without Opening (P= 100 kips, E= 1,000,000 psi & t = 1")

h/d	Δ_F	R_F	***h/d***	Δ_F	R_F	***h/d***	Δ_F	R_F	***h/d***	Δ_F	R_F	***h/d***	Δ_F	R_F	***h/d***	Δ_F	R_F	***h/d***	Δ_F	R_F
5.55	18.76	0.053	**5.81**	21.36	0.047	**6.06**	24.07	0.042	**6.32**	27.14	0.04	**6.59**	30.60	0.033	**6.85**	34.197	0.029	**7.11**	38.076	0.026
5.56	18.86	0.053	**5.82**	21.46	0.047	**6.07**	24.19	0.041	**6.33**	27.26	0.04	**6.60**	30.73	0.033	**6.86**	34.341	0.029	**7.12**	38.230	0.026
5.57	18.95	0.053	**5.83**	21.56	0.046	**6.08**	24.30	0.041	**6.34**	27.39	0.04	**6.61**	30.86	0.032	**6.87**	34.485	0.029	**7.13**	38.386	0.026
5.58	19.05	0.052	**5.84**	21.67	0.046	**6.09**	24.41	0.041	**6.35**	27.51	0.04	**6.62**	31.00	0.032	**6.88**	34.630	0.029	**7.14**	38.541	0.026
5.59	19.14	0.052	**5.85**	21.78	0.046	**6.1**	24.53	0.041	**6.36**	27.63	0.04	**6.63**	31.13	0.032	**6.89**	34.775	0.029	**7.15**	38.698	0.026
5.60	19.24	0.052	**5.86**	21.88	0.046	**6.11**	24.64	0.041	**6.37**	27.76	0.04	**6.64**	31.27	0.032	**6.90**	34.921	0.029	**7.16**	38.854	0.026
5.61	19.34	0.052	**5.87**	21.99	0.045	**6.12**	24.76	0.040	**6.38**	27.88	0.04	**6.65**	31.40	0.032	**6.91**	35.067	0.029	**7.17**	39.011	0.026
5.62	19.44	0.051	**5.88**	22.09	0.045	**6.13**	24.87	0.040	**6.39**	28.01	0.04	**6.66**	31.54	0.032	**6.92**	35.213	0.028	**7.18**	39.169	0.026
5.63	19.53	0.051	**5.89**	22.20	0.045	**6.14**	24.99	0.040	**6.40**	28.13	0.04	**6.67**	31.68	0.032	**6.93**	35.360	0.028	**7.19**	39.326	0.025
5.64	19.63	0.051	**5.9**	22.31	0.045	**6.15**	25.11	0.040	**6.41**	28.26	0.04	**6.68**	31.81	0.031	**6.94**	35.508	0.028	**7.20**	39.485	0.025
5.65	19.73	0.051	**5.91**	22.42	0.045	**6.16**	25.22	0.040	**6.42**	28.39	0.04	**6.69**	31.95	0.031	**6.95**	35.655	0.028	**7.21**	39.644	0.025
5.66	19.83	0.050	**5.92**	22.52	0.044	**6.17**	25.34	0.039	**6.43**	28.51	0.04	**6.70**	32.09	0.031	**6.96**	35.803	0.028	**7.22**	39.803	0.025
5.67	19.93	0.050	**5.93**	22.63	0.044	**6.18**	25.46	0.039	**6.44**	28.64	0.03	**6.71**	32.22	0.031	**6.97**	35.952	0.028	**7.23**	39.962	0.025
5.68	20.03	0.050	**5.94**	22.74	0.044	**6.19**	25.57	0.039	**6.45**	28.77	0.03	**6.72**	32.36	0.031	**6.98**	36.101	0.028	**7.24**	40.122	0.025
5.69	20.13	0.050	**5.95**	22.85	0.044	**6.2**	25.69	0.039	**6.46**	28.90	0.03	**6.73**	32.50	0.031	**6.99**	36.250	0.028	**7.25**	40.283	0.025
5.70	20.23	0.049	**5.96**	22.96	0.044	**6.21**	25.81	0.039	**6.47**	29.03	0.03	**6.74**	32.64	0.031	**7.00**	36.400	0.027	**7.26**	40.444	0.025
5.71	20.33	0.049	**5.97**	23.07	0.043	**6.22**	25.93	0.039	**6.48**	29.15	0.03	**6.75**	32.78	0.031	**7.01**	36.550	0.027	**7.27**	40.605	0.025
5.72	20.43	0.049	**5.98**	23.18	0.043	**6.23**	26.05	0.038	**6.49**	29.28	0.03	**6.76**	32.92	0.030	**7.02**	36.701	0.027	**7.28**	40.767	0.025
5.73	20.53	0.049	**5.99**	23.29	0.043	**6.24**	26.17	0.038	**6.50**	29.41	0.03	**6.77**	33.06	0.030	**7.03**	36.852	0.027	**7.29**	40.929	0.024
5.74	20.63	0.048	**6.00**	23.40	0.043	**6.25**	26.29	0.038	**6.51**	29.54	0.03	**6.78**	33.20	0.030	**7.04**	37.003	0.027	**7.30**	41.092	0.024
5.75	20.74	0.048	**6.01**	23.51	0.043	**6.26**	26.41	0.038	**6.52**	29.67	0.03	**6.79**	33.34	0.030	**7.05**	37.155	0.027	**7.31**	41.255	0.024
5.76	20.84	0.048	**6.02**	23.62	0.042	**6.27**	26.53	0.038	**6.53**	29.80	0.03	**6.80**	33.48	0.030	**7.06**	37.308	0.027	**7.32**	41.418	0.024
5.77	20.94	0.048	**6.03**	23.73	0.042	**6.28**	26.65	0.038	**6.54**	29.93	0.03	**6.81**	33.63	0.030	**7.07**	37.460	0.027	**7.33**	41.582	0.024
5.78	21.04	0.048	**6.04**	23.85	0.042	**6.29**	26.77	0.037	**6.55**	30.07	0.03	**6.82**	33.77	0.030	**7.08**	37.613	0.027	**7.34**	41.747	0.024
5.79	21.15	0.047	**6.05**	23.96	0.042	**6.3**	26.89	0.037	**6.56**	30.20	0.03	**6.83**	33.91	0.029	**7.09**	37.767	0.026	**7.35**	41.912	0.024
5.80	21.25	0.047	**6.06**	24.07	0.042	**6.31**	27.02	0.037	**6.57**	30.33	0.03	**6.84**	34.05	0.029	**7.10**	37.921	0.026	**7.36**	42.077	0.024

3-11 SAMPLE PROBLEMS

Sample Problem 3-1: Distribution of Force Among Columns

Given:
A force acting at the top of a building frame as shown. The supporting columns are of equal height and are fixed at the base. The modulus of elasticity, E , is the same for each column. Assuming the top girder is rigid, and $I_1 = 1/3\ I_2 = 1/6\ I_3$.



Find: The shear carried by the first column is most nearly:

Answers: A) 1/10 F B) 1/5 F C) 2/3 F D) 1/3 F

Solution:
Since the top girder is rigid, all columns will be treated as fixed-fixed.

From Table 3-2, the stiffness is given as $K = \frac{12EI}{h^3}$ and since E, I and h are the same for all columns, the force F will be distributed in proportional to I.

$$K_1 = \frac{12E_1 I_1}{h_1^{\ 3}}$$

$$V_1 = F\frac{K_1}{K_1 + K_2 + K_3} = F\frac{I_1}{I_1 + I_2 + I_3} = F\frac{I_1}{I_1 + 3I_1 + 6I_1} = F/10$$

NOTE:

1- The lateral force distributed among columns and frames in proportional to their stiffnesses.
2- The lateral force distributed among shear walls in proportional to their absolute or relative rigidities (if rigid diaphragm).

Answer: A ⇐

Sample Problem 3-2: Deflection vs. Rigidity

Given:
Deflection is the inverse of which of the following?

A) Flexibility
B) Stiffness
C) Rigidity
D) Compliance

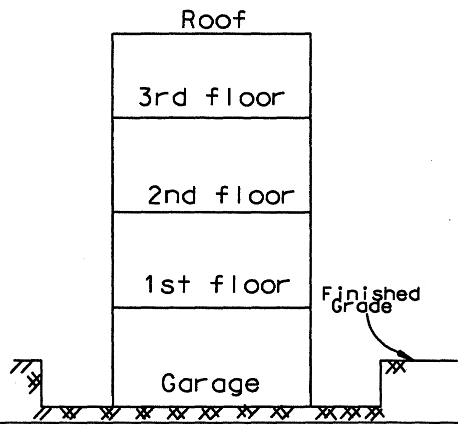
Solution:
Flexibility is the reciprocal of stiffness and compliance is the same as flexibility. Deflection is the inverse of rigidity. $R = 1/\Delta$

Answer: C ⇐

Sample Problem 3-3: Base, Level & Story

Given: For the building shown, what level is considered the base?

A) Garage floor
B) First floor
C) Second floor
D) Third floor



Solution:
Base is defined as the level at which the seismic force is transmitted (imparted) to the structure.§11.2 ASCE 7-06 (Definitions)

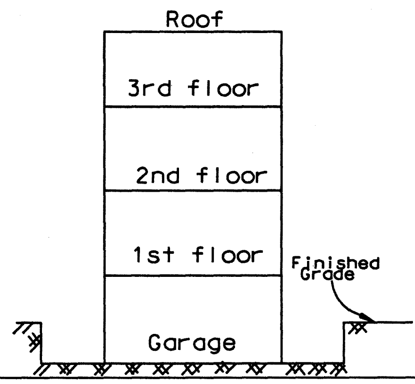
Answer: A ⇐

Sample Problem 3-4: Base, Level & Story

Given:

The second level of the structure shown is located at:

A) The first floor
B) The second floor
C) The garage floor
D) The roof



Solution:
Level "i" is the level of the structure referred to by subscript "i". "i" = 1 designates the first level above the base. §11.3 ASCE 7-05 (NOTATION).

Answer: B ⇐

Sample Problem 3-5:Base, Level & Story

Given: For the structure shown ABOVE (Prob. 3-3), the first story is the story immediately:

A) Above the first floor
B) Above the second floor
C) Below the roof
D) Below the first floor

Solution:
Story is the space between levels. Story x is the story below level x above the base. §11.2 ASCE7-05 (DEFINITIONS).

Answer: D ⇐

Sample Problem 3-6: Damping Ratio

Given: Damping ratio is defined as the ratio of:

A) Critical damping coefficient to actual damping coefficient
B) Actual soil site period to building period
C) Actual damping coefficient to critical damping coefficient
D) Base shear to total seismic dead load

Solution:

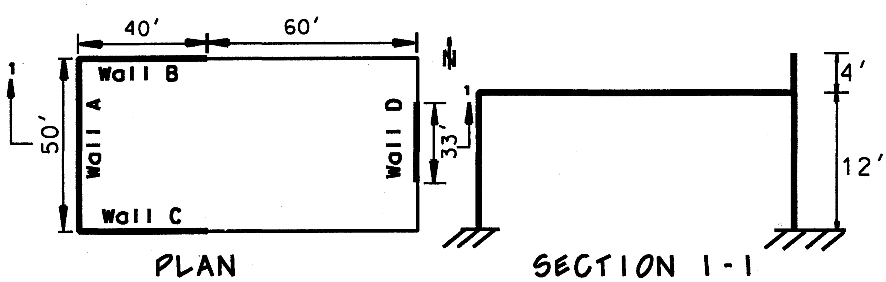
The ratio between the damping coefficient and the critical damping is known as the damping ratio.

$$\xi = \frac{B}{B_{critical}} \qquad (3-7)$$

Answer: C ⇐

Sample Problem 3-7: Relative Rigidity of Solid Walls

Given: The building as shown has four concrete shear walls. The plan view and cross section show the arrangement and height of the walls.



Find: The relative rigidity of wall D is most nearly

Answer: A) 8.876 B) 7.895 C) 6.449 D) 5.312

Solution:
The connection between the roof and the wall is not given. Therefore, it is conservative to assume that all walls are fixed at the bottom free at top (i.e.) cantilever. Wall D has a parapet of 4 feet. Normally these concrete wall are precast and assembled at the job site (tilt up construction). Even if the wall is cast-in-place, the parapet is going to be a continuous part of the wall.

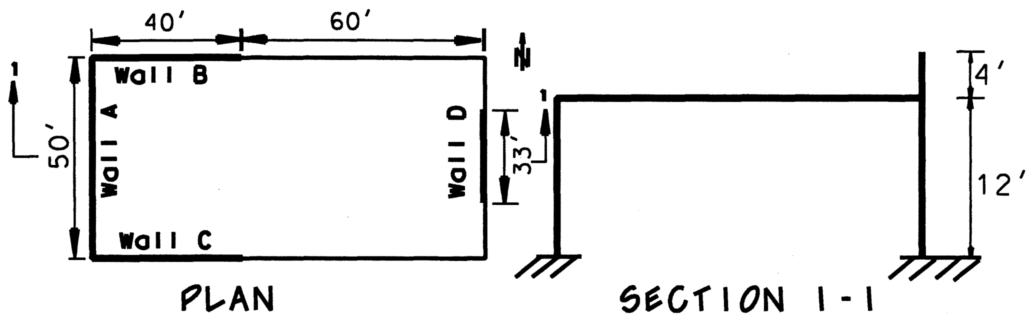
$$h/d = \frac{16}{33} = 0.4848$$

Using Table 3-2 and for the value of $h/d = 0.4848$, the rigidity $R_c = 5.312$
Answer (A) is for fixed-fixed for h/d = 12/33 = 0.3636, the rigidity $R_F = 8.876$
Answer (B) is for h/d = 16/33 = 0.4848, the rigidity $R_F = 6.449$
Answer (D) is for cantilever of h/d =12/33=0.3636, the rigidity $R_c = 7.895$

Answer: D ⇐

Sample Problem 3-8: Center of Rigidity (CR)

Given:
The building shown has four concrete shear walls as shown. The plan view and cross section show the arrangement and height of the walls.



Find:
The center of rigidity (CR) is

Answers:

A) Closer to wall "A"
B) Closer to wall "D"
C) Half-way between walls "B" & "C"
D) Answers A & C

Solution:

The center of rigidity (CR) is always closer to wall(s) that has a higher rigidity value(s). Wall "A" has higher rigidity because smaller h/d ratio (12/50) = 0.24. Also, the CR is midway between walls "B" & "C" because both of them have the same *h/d* ratio.

Answer: D⇐

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Chapter 4

Seismic Code Requirements for Building Structures
2007 CBC Chapter 16/ ASCE 7-05 Chapter 12

Topics to be covered

- Basis for Design
- Actual and Design Seismic Force
- Selection of Lateral Force Procedure
- Seismic Design Parameters
- Base Shear Calculations
- Distribution of Base Shear
- Redundancy Factor
- Combinations of Structural Systems
- Drift and Building Separations
- P-Delta ($P-\Delta$) Effects

10 Example Problems

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Chapter 4- Building Structures

4.1 2007 CBC/ ASCE 7-05 STRUCTURAL DESIGN BASIS

§11.1.1 of the ASCE 7-05 set the purpose of the Seismic Design Criteria as:

11.1.1 Purpose : The specified earthquake loads are based upon post-elastic energy dissipation in the structure, and because of this fact, the requirements for design, detailing, and construction shall be satisfied even for structures and members for which load combinations that do not contain earthquake loads indicate larger demands than combinations that include earthquake loads. The purpose of the earthquake provisions is primarily to safeguard against major structural failures and loss of life, not to limit damage or maintain function.

Structures designed in conformance with the seismic design provisions prescribed by the current Code should be able to:

1. Resist minor ground motion without damage.
2. Resist moderate ground motion without structural damage but with some nonstructural damage.
3. Resist major ground motion without collapse but with possible structural and nonstructural damage.
4. The seismic provisions in the ASCE 7-05 will consider the potential geological and seismic hazards in Seismic Design Categories C through F from:

    - Slope instability
    - Liquefaction
    - Differential settlement
    - Surface displacement due to faulting or lateral spreading.

How does the building code attempt to accomplish this objective when resisting earthquake ground motion ?

1- Design the structure for forces less than those corresponding to elastic response generated by the design earthquake.
2- Rely on ductility and detailing to prevent collapse
3- Allow the energy imparted by the earthquake to be absorbed by the structure without destroying it.
4- It is assumed that the input energy is absorbed (dissipated) upon post-elastic energy dissipation in the structure.

It should be noted that the amount or level of structural damage depends of the following primary parameters:

TABLE 4-1 Parameters Affecting Structural Damage

Earthquake Parameters	Site Parameters	Structural Parameters
➢ Magnitude ➢ Duration ➢ Frequency ➢ Length of fault	➢ Soil characteristics ➢ Distance to fault(s) ➢ Natural period of the site and its relation to structural period (resonance) CH 3-7	➢ Natural period of the building ➢ Building configuration (regular vs. irregular) ➢ Type of lateral-force resisting system (LFRS) and detailing (MRF, braced frames, shear walls) ➢ Construction material (steel, concrete, wood, masonry) ➢ Quality control of construction

As stated in the ASCE 7-05 that the basic requirements of the seismic design is "The seismic analysis and design procedures to be used in the design of building structures and their components shall be as prescribed in this section. The building structure shall include complete lateral and vertical force-resisting systems capable of providing adequate strength, stiffness, and energy dissipation capacity to withstand the design ground motions within the prescribed limits of deformation and strength demand. The design ground motions shall be assumed to occur along any horizontal direction of a building structure. The adequacy of the structural systems shall be demonstrated through the construction of a mathematical model and evaluation of this model for the effects of design ground motions."

11.1.3 Applicability:

Structures and their nonstructural components shall be designed and constructed in accordance with the requirement of the following sections based on the type of structure or component:

a. Buildings: Chapter 12 (ASCE 7-05)
b. Nonstructural Components: Chapter 13 (ASCE 7-05)
c. Nonbuilding Structures: Chapter 15 (ASCE 7-05)
d. Seismically Isolated Structures: Chapter 17 (ASCE 7-05)
e. Structures with Damping Systems: Chapter 18 (ASCE 7-05)

There are major processes that will be included in the design for seismic requirements taken into consideration the seismic design category (SDC), design procedure (dynamic, equivalent lateral force procedure (ELF) or simplified static) as follows:

- Distribution of horizontal shear
- Stability against overturning
- Anchorage

The following is a list of major milestones in the process of lateral force design:

Calculate Loads (Seismic Force in Ch. 12 of ASCE 7 is at SD Level) → Determine the most severe load combination SD/LRFD or ASD (Ch. 2 ASCE 7) or Ch 16 CBC → Continuous load path to transfer loads from origin to LFRS
- Distribution
- Stability (OTM)
- Anchorage

Figure 4-1 Major Steps in Design for Lateral Loads

4.2 STRUCTURAL SYSTEMS

In Chapter 1 of this manual, loads were classified as vertical (gravity) and lateral loads (horizontal). The classification of the structural systems in the current Code is based on which portion of the system is responsible of resisting (carrying) the vertical load and which portion is responsible of carrying the lateral load. The following abbreviations will be used in relation to classification of structural systems:

SMF	= Special moment frame
IMF	= Intermediate moment frame
MMWF	= Masonry moment wall frame
OMF	= Ordinary moment frame
EBF	= Eccentrically braced frame
CBF	= Concentrically braced frame

Section §11.2 of ASCE 7 gives the definitions of the different structural systems as follows:

1- **Building Frame System:** A structural system with an essentially complete space frame providing support for vertical loads. Seismic force resistance is provided by shear walls or braced frames.

2- **Dual System**: A structural system with an essentially complete space frame providing support for vertical loads. Seismic force resistance is provided by moment resisting frames and shear walls or braced frames as prescribed in Section 12.2.5.1.

3- **Shear Wall-Frame Interactive System**: A structural system that uses combinations of ordinary reinforced concrete shearwalls and ordinary reinforced concrete moment frames designed to resist lateral forces in proportion to their rigidities considering interaction between shear walls and frames on all levels.

4- **Space Frame System**: A 3-D structural system composed of interconnected members, other than bearing walls, that is capable of supporting vertical loads and, where designed for such an application, is capable of providing resistance to seismic forces.

Also, the different types of frames are defined as follows:

1- **Braced Frame**: An essentially vertical truss, or its equivalent, of the concentric or eccentric type that is provided in a building frame system or dual system to resist seismic forces.

2- **Concentrically Braced Frame (CBF):** A braced frame in which the members are subjected primarily to axial forces. CBFs are categorized as ordinary concentrically braced frames (OCBF) or special concentrically braced frames (SCBF).

3- **Eccentrically Braced Frame (EBF)**: A diagonally braced frame in which at least one end of each brace frames into a beam a short distance from a beam-column or from another diagonal brace.

4- **Moment Frame:** A frame in which members and joints resist lateral forces by flexure as well as along the axis of the members. Moment frames are categorized as intermediate moment frames (IMF), ordinary moment frames (OMF), and special moment frames (SMF).

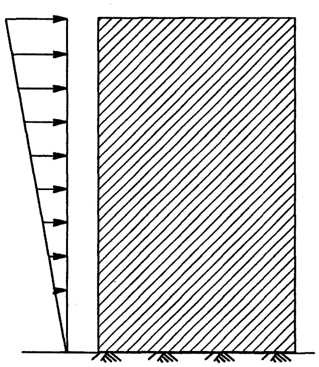
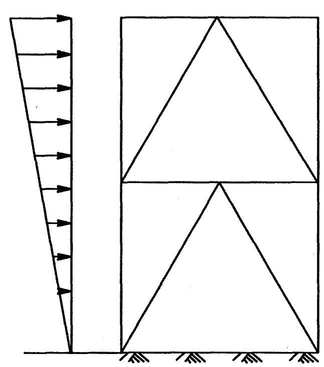
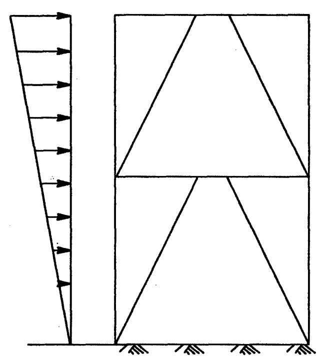
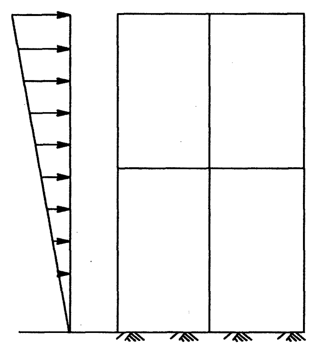
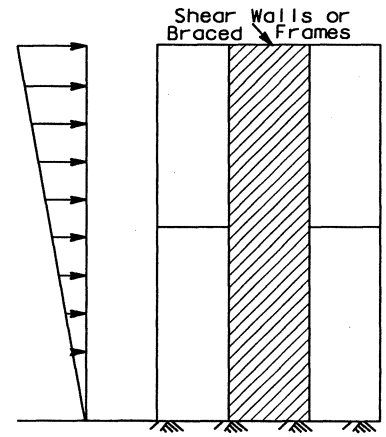
Table 12.2-1 of ASCE 7 lists the design coefficients and factors for EIGHT categories [A through H] total of 84 systems of the force resisting systems.

The 7 Rules

1- The seismic base shear (V) acts at the center of mass (CM)
2- The resultant of resisting elements (R) acts at the center of rigidity (CR)
3- All masses will be used to calculate the center of mass (CM)
4- Certain (not necessarily all) rigidities (or stiffnesses) will be used to calculate the center of rigidity (CR)
5- The seismic force is distributed among resisting elements in proportional to their rigidities (or stiffnesses) if the diaphragm is a rigid.
6- The seismic force is distributed among resisting elements in proportional to their tributary area if the diaphragm is a flexible.
7- Seismic shear is resisted by CERTAIN (not necessarily all) walls while torsional moment is resisted by ALL walls

In general, structural systems can be categorized as follows in regard to resisting lateral forces:

TABLE 4-2 LATERAL-FORCE RESISTING SYSTEMS (LRFS)

Shear Wall System	Frame System			Dual System
	Braced		Unbraced (UBF)	
	Concentric (CBF)	Eccentric (EBF)		
➢ A system where the shear wall is designed to resists lateral forces parallel to the plane of the wall ➢ Also known as vertical diaphragm or structural wall system	➢ A vertical truss system to resist lateral forces ➢ The members are subjected primarily to axial forces.	➢ A vertical truss system to resist lateral forces	➢ A system consists of beams and columns to carry gravity (vertical) loads ➢ The lateral forces are resisted either by shear walls or by the members and joints (flexure action)	➢ The total seismic force resistance is to be provided by the combination of the moment frames and the shear walls or braced frames in proportion to their rigidities. ➢ For a dual system, the moment frames shall be capable of resisting at least 25 percent of the design seismic forces.
				

A- Bearing Wall System: A structural system <u>without a complete vertical load-carrying space frame</u>. Bearing walls or bracing systems provide support for all or most gravity loads. Resistance to lateral load is provided by shear walls or braced frames.

The following table shows the difference between the shear walls and braced frames

TABLE 4-3 BEARING WALL SYSTEMS

A-Bearing Wall Systems (1-15 in Table 12.2-1)
➢ Carry vertical (gravity) loads ➢ Resist horizontal (lateral) loads ➢ <u>Examples</u>: concrete and masonry shear walls, light-framed walls with shear panels.

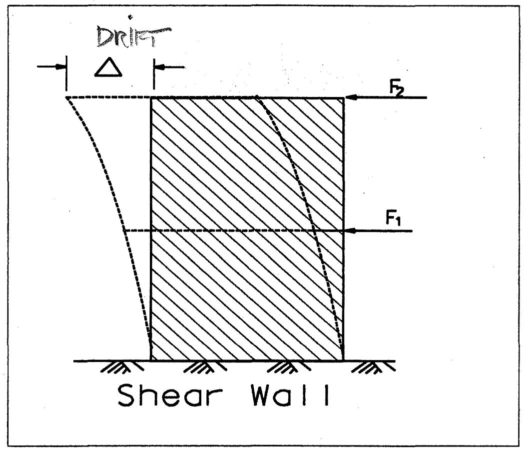


Figure 4-2 Bearing Wall System

The following table shows the height limitations of some of the bearing wall systems listed in Table 12.2-1 for the SIX DESIGN CATEGORIES (SDC):

TABLE 4-3 BEARING WALL SYSTEMS (Systems 1 through 15 in Table 12.2-1)

A-Bearing Wall Systems	Height limitation per "SDC", ft				
BEST	A* or B	C	D[d]	E[d]	F[e] WORST
1. Special reinforced concrete shear walls	NL	NL	160	160	100
2. Ordinary reinforced concrete shear walls	NL	NL	NP	NP	NP
7. Special reinforced masonry shear walls	NL	NL	160	160	100
8. Intermediate reinforced masonry shear walls	NL	NL	NP	NP	NP
9. Ordinary reinforced masonry shear walls	NL	160	NP	NP	NP
13. Light frame walls with wood structural panels	NL	NL	65	65	65

NL = No limit NP = Not permitted

[d] See Section 12.2.5.4 for a description of building systems limited to buildings with a height of 240 ft (73.2 m) or less.

[e] See Section 12.2.5.4 for building systems limited to buildings with a height of 160 ft (48.8 m) or less.

* SDC "A" is not listed in Table 12.2-1 because in this SDC the $S_{DS} < 0.167g$ and $S_{D1} < 0.067g$ which indicates that the structures in this SDC are the least vulnerable to earthquake forces compared to other SDC. Therefore, "NL" for SDC "B" also means "NL" for SDC "A"

The following are some conclusions could be drawn from the above table:

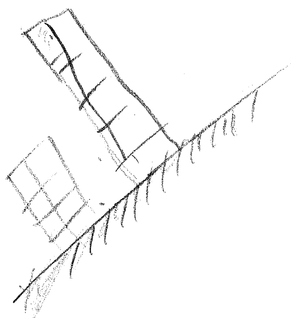
- Concrete and masonry shear walls used in SDC "D, E & F" are required to be specially detailed reinforced walls.
- The maximum height for special reinforced concrete and masonry shear walls when used in SDC "D, E " is 160 ft and 100 ft for SDC "F".
- Light frame walls with wood structural panels could be used in SDC " D, E & F" with 65 ft height limit.

B- Building Frame Systems: A structural system with an essentially complete space frame providing support for gravity loads. Resistance to lateral load is provided by shear walls or braced frames.

TABLE 4-5 BUILDING FRAME SYSTEMS

Shear Walls	Braced Frames
➢ Carry vertical (gravity) loads ➢ Resist horizontal (lateral) loads ➢ <u>Examples</u>: concrete and masonry shear walls, light-framed walls with shear panels.	➢ Carry <u>some</u> gravity loads ➢ Resist horizontal (lateral) loads ➢ <u>Examples</u>: Steel or timber braced frames where the bracing carries gravity loads and light steel –framed bearing walls with tension only braces

$T_a = C_t (h_n)^x$



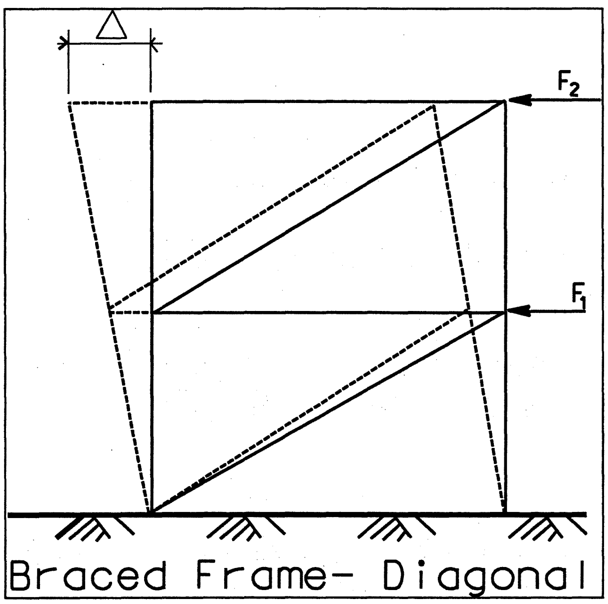


Figure 4-3 Building Frame System

There are 27 different systems listed in Table 12.2-1 of the ASCE7 under the building frame systems category. Below is a brief list of this category and the height limitations for the SIX SDC where these systems could be used.

TABLE 4-6 BUILDING FRAME SYSTEMS (Systems 1 through 27 of Table 12.2-1)

B-Building Frame Systems	Height limitation per "SDC", ft				
	A* or B	C	D[d]	E[d]	F[e]
1.Steel eccentrically braced frames, moment resisting connections at columns away from links	NL	NL	160	160	100
2. Steel eccentrically braced frames, non-moment-resisting connections at columns away from links	NL	NL	160	160	100
3. Special steel concentrically braced frames	NL	NL	160	160	100
4. Ordinary steel concentrically braced frames	NL	NL	35[j]	35[j]	NP[j]
5. Special reinforced concrete shear walls	NL	NL	160	160	100

NL = No limit **NP = Not permitted**

[d] See Section 12.2.5.4 for a description of building systems limited to buildings with a height of 240 ft (73.2 m) or less.

[e] See Section 12.2.5.4 for building systems limited to buildings with a height of 160 ft (48.8 m) or less.

[j] Steel ordinary concentrically braced frames are permitted in single-story buildings up to a height of 60 ft (18.3 m) where the dead load of the roof does not exceed 20 psf (0.96 kN/m2) and in penthouse structures

* SDC "A" is not listed in Table 12.2-1 because in this SDC the $S_{DS}<0.167g$ *and* $S_{D1}<0.067g$ which indicate that the structures in this SDC are the least vulnerable to earthquake forces compared to other SDC. Therefore, "NL" for SDC "B" also means "NL" for SDC "A"

The following are some conclusions could be drawn from the above table:

- For structures assigned to Seismic Design Categories D, E, and F, specially detailed concrete and masonry shear walls, as specified for bearing wall systems, may be utilized and these are limited to a maximum height of 100 feet for category F structures and 160 feet for category D and E structures.

- Steel braced frames in Seismic Design Categories D, E, and F may be special concentrically braced.

- Ordinary reinforced concrete shear walls and intermediate reinforced masonry shear walls may be used in Seismic Design Categories A, B, and C without limitations on their height.

C- Moment-Resisting Frame Systems (Systems 1 to 11 in Table 12.2-1 of ASCE 7): A structural system with an essentially complete space frame providing support for gravity loads. Moment-resisting frames are specially detailed to provide resistance to lateral load primarily by flexural action of members.

In SDC "D, E, and F", special reinforced concrete and structural steel moment-resisting frames are required to be detailed to satisfy ACI sections 2 1.2 through 21.5 No restrictions are placed on the height of these systems. Moment-resisting frames have the advantage of affording unlimited free access in a building. In addition, a high degree of redundancy can be provided and the system has an excellent inelastic response capacity. Large lateral displacements may be developed while the gravity load carrying capacity remains intact. The large displacements, however, may cause damage to nonstructural elements.

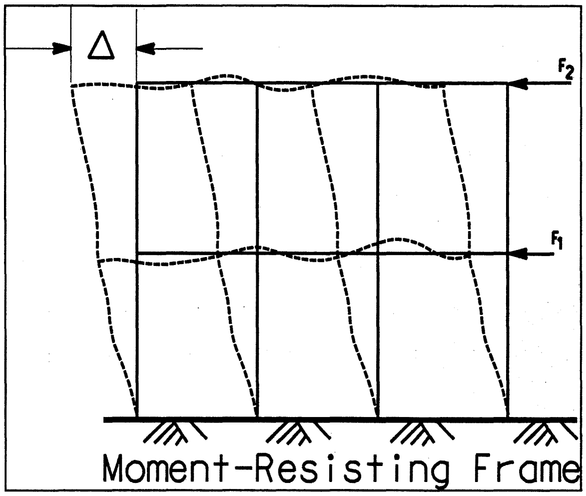


Figure 4-4 Moment-Resisting Frame System

Below is a list of this category and the height limitations for the SIX SDC.

TABLE 4-7 MOMENT-RESISTING FRAME SYSTEMS (1 through 11 of Table 12.2-1)

C-Moment-Resisting Frame Systems	Height limitation per "SDC", ft				
	A* or B	C	D[d]	E[d]	F[e]
1. Special steel moment frames	NL	NL	NL	NL	NL
2. Special steel truss moment frames	NL	NL	160	100	NP
3. Intermediate steel moment frames	NL	NL	35[h]	NP[h]	NP[i]
4. Ordinary steel moment frames	NL	NL	NP[h]	NP[h]	NP[i]
5. Special reinforced concrete moment frames	NL	NL	NL	NL	NL
6. Intermediate reinforced concrete moment frames	NL	NL	NP	NP	NP
7. Ordinary reinforced concrete moment frames	NL	NP	NP	NP	NP
8. Special composite steel and concrete moment frames	NL	NL	NL	NL	NL
9. Intermediate composite moment frames	NL	NL	NP	NP	NP
10. Composite partially restrained moment frames	160	160	100	NP	NP
11. Ordinary composite moment frames	NL	NP	NP	NP	NP

NL = No limit NP = Not permitted

[d] See Section 12.2.5.4 for a description of building systems limited to buildings with a height of 240 ft (73.2 m) or less.

[e] See Section 12.2.5.4 for building systems limited to buildings with a height of 160 ft (48.8 m) or less.

[h] See Sections 12.2.5.6 and 12.2.5.7 for limitations for steel OMFs and IMFs in structures assigned to Seismic Design Category D or E.

[i] See Sections 12.2.5.8 and 12.2.5.9 for limitations for steel OMFs and IMFs in structures assigned to Seismic Design Category F.

* SDC "A" is not listed in Table 12.2-1 because in this SDC the $S_{DS}<0.167g$ *and* $S_{D1}<0.067g$ which indicate that the structures in this SDC are the least vounlerable to earthquake forces compared to other SDC. Therefore, "NL" for SDC "B" also means "NL" for SDC "A"

The following are some conclusions could be drawn from the above table:

- Intermediate and ordinary steel moment frames and intermediate reinforced concrete moment frames may be used in Seismic Design Categories A, B, and C without limitations on their height.

D- Dual System with Special Moment Resisting Frames Capable of Resisting at Least 25% of Prescribed Forces (Systems 1 through 13 of the ASCE 7):

§ 12.2.5.1 Dual System. For a dual system, the moment frames shall be capable of resisting at least 25 percent of the design seismic forces. The total seismic force resistance is to be provided by the combination of the moment frames and the shear walls or braced frames in proportion to their rigidities.

A structural system with the following features:

1. An essentially complete space frame that provides support for gravity loads.
2. Resistance to lateral load is provided by shear walls or braced frames and moment-resisting frames (SMF, IMF, MMWF or steel OMF). The moment-resisting frames shall be designed to independently resist at least 25 percent of the design base shear.
3. The two systems shall be designed to resist the total design base shear in proportion to their relative rigidities considering the interaction of the dual system at all levels.

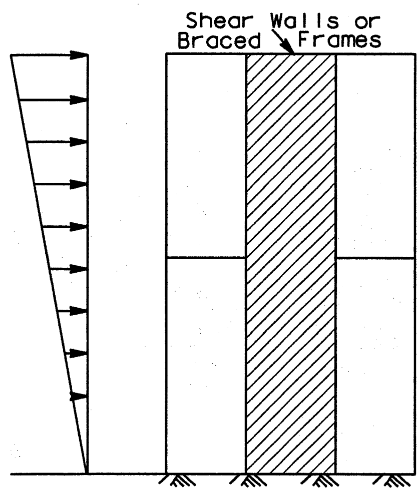


Figure 4-5 Dual System

The following Table has a list of this category and the height limitations for the SIX SDC.

TABLE 4-8 Dual System with Special Moment Resisting Frames Capable of Resisting at Least 25% of Prescribed Forces (Systems 1 through 13 of the ASCE 7)

D- Dual Systems	Height limitation per "SDC", ft				
	A or B	C	D[d]	E[d]	F[e]
1. Steel eccentrically braced frames	NL	NL	NL	NL	NL
2. Special steel concentrically braced frames	NL	NL	NL	NL	NL
3. Special reinforced concrete shear walls	NL	NL	NL	NL	NL
4. Ordinary reinforced concrete shear walls	NL	NL	NP	NP	NP
5. Composite steel and concrete eccentrically braced frames	NL	NL	NL	NL	NL
6. Composite steel and concrete concentrically braced frames	NL	NL	NL	NL	NL
7. Composite steel plate shear walls	NL	NL	NL	NL	NL
8. Special composite reinforced concrete shear walls with steel elements	NL	NL	NL	NL	NL
9. Ordinary composite reinforced concrete shear walls with steel elements	NL	NL	NP	NP	NP
10. Special reinforced masonry shear walls	NL	NL	NL	NL	NL
11. Intermediate reinforced masonry shear walls	NL	NL	NP	NP	NP
12. Buckling-restrained braced frame	NL	NL	NL	NL	NL
13. Special steel plate shear walls	NL	NL	NL	NL	NL

NL = No limit NP = Not permitted

* SDC "A" is not listed in Table 12.2-1 because in this SDC the $S_{DS} < 0.167g$ and $S_{D1} < 0.067g$ which indicates that the structures in this SDC are the least vulnerable to earthquake forces compared to other SDC. Therefore, "NL" for SDC "B" also means "NL" for SDC "A"

E- Table 4-9 Dual System with Intermediate Moment Resisting Frames Capable of Resisting at Least 25% of Prescribed Forces (1 through 8 in Table 12.2-1 of ASCE7)

E- Dual Systems	Height limitation per "SDC", ft				
	A or B	C	D[d]	E[d]	F[e]
1. Special steel concentrically braced frames[f]	NL	NL	35	NP	NP[h,k]
2. Special reinforced concrete shear walls	NL	NL	160	100	100
3. Ordinary reinforced masonry shear walls	NL	160	NP	NP	NP
4. Intermediate reinforced masonry shear walls	NL	NL	NP	NP	NP
5. Composite steel and concrete concentrically braced frames	NL	NL	160	100	NP
6. Ordinary composite braced frames	NL	NL	NP	NP	NP
7. Ordinary composite reinforced concrete shear walls with steel elements	NL	NL	NP	NP	NP
8. Ordinary reinforced concrete shear walls	NL	NL	NP	NP	NP

[h] See Sections 12.2.5.6 and 12.2.5.7 for limitations for steel OMFs and IMFs in structures assigned to Seismic Design Category D or E.

[k] Increase in height to 45 ft (13.7 m) is permitted for single story storage warehouse facilities.

§12.2.5.2 Cantilever Column Systems. Cantilever column systems are permitted as indicated in Table 12.2-1 and as follows:

- The axial load on individual cantilever column elements calculated in accordance with the load combinations of Section 2.3 shall not exceed 15 percent of the design strength of the column to resist axial loads alone, or for allowable stress design, the axial load stress on individual cantilever column elements, calculated in accordance with the load combinations of Section 2.4 shall not exceed 15 percent of the permissible axial stress.
- Foundation and other elements used to provide overturning resistance at the base of cantilever column elements shall have the strength to resist the load combinations with over strength factor of Section 12.4.3.2.

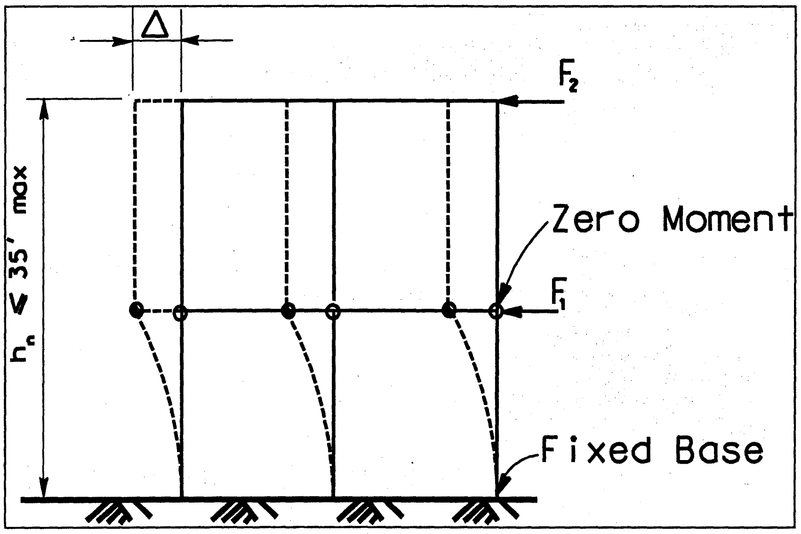


Figure 4-6 Cantilevered-Column System

Table 4-10 CANTILEVERED COLUMN SYSTEMS (Systems 1-7)

G. CANTILEVERED COLUMN SYSTEMS DETAILED TO CONFORM TO THE REQUIREMENTS FOR:	Height limitation per "SDC", ft				
	A or B	C	D[d]	E[d]	F[e]
1. Special steel moment frames	35	35	35	35	35
2. Intermediate steel moment frames	35	35	35h	NP[h]	NP[i]
3. Ordinary steel moment frames	35	35	NP	NP[h]	NP[i]
4. Special reinforced concrete moment frames	35	35	35	35	35
5. Intermediate concrete moment frames	35	35	NP	NP	NP
6. Ordinary concrete moment frames	35	NP	NP	NP	NP
7. Timber frames	35	35	35	NP	NP

§12.2.5.3 Inverted Pendulum-Type Structures. Regardless of the structural system selected, inverted pendulums as defined in Section 11.2 as: "**INVERTED PENDULUM-TYPE STRUCTURES**: Structures in which more than 50 percent of the structure's mass is concentrated at the top of a slender, cantilevered structure and in which stability of the mass at the top of the structure relies on rotational restraint to the top of the cantilevered element", Supporting columns or piers of inverted pendulum-type structures shall be designed for the bending moment calculated at the base determined using the procedures given in Section 12.8 and varying uniformly to a moment at the top equal to one-half the calculated bending moment at the base.

4.3 HEIGHT LIMITS

Table 12.2-1 gives the height limits for the structural systems listed and in the SIX Seismic Design Categories (SDC). Note that "**NL**" means no limit and "**NP**" means not permitted.

12.2.5.4 Increased Building Height Limit for Steel Braced Frames and Special Reinforced Concrete Shear Walls. The height limits in Table 12.2-1 are permitted to be increased from 160 ft (50 m) to 240 ft (75 m) for structures assigned to Seismic Design Categories D or E and from 100 ft (30 m) to 160 ft (50 m) for structures assigned to Seismic Design Category F that have steel braced frames or special reinforced concrete cast-in-place shear walls and that meet both of the following requirements:

1- The structure shall not have an extreme torsional irregularity as defined in Table 12.2-1 (horizontal structural irregularity Type 1b).

2- The braced frames or shear walls in any one plane shall resist no more than 60 percent of the total seismic forces in each direction, neglecting accidental torsional effects.

4.4 2007 CBC/ ASCE 7-05 CLASSIFICATION OF STRUCTURES

12.3.2 Irregular and Regular Classification. Structures shall be classified as regular or irregular based upon the criteria in this section. Such classification shall be based on horizontal and vertical configurations.

§12.3.2.1 Horizontal Irregularity. Structures having one or more of the irregularity types listed in Table 12.3-1 shall be designated as having horizontal structural irregularity. Such structures assigned to the seismic design categories listed in Table 12.3-1 shall comply with the requirements in the sections referenced in that table.

TABLE 4-11 REGULAR AND IRREGULAR STRUCTURES

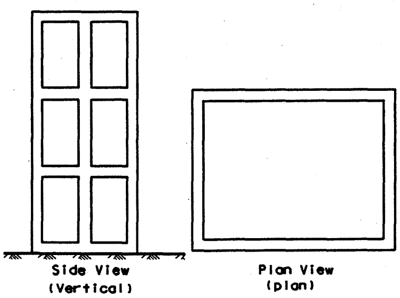
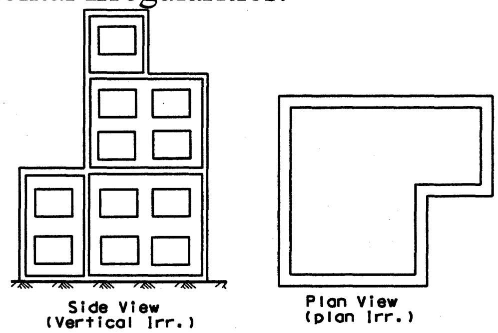
Regular Structures	Irregular Structures
➢ Regular structures have no significant physical discontinuities in plan or vertical configuration or in their lateral-force-resisting systems such as the irregular features  ➢ Vertical irregularities: Examine the side view of the structure	➢ Irregular structures have significant physical discontinuities in configuration or in their lateral-force-resisting systems. Irregular features. There are **SEVEN** Vertical and **SIX** Horizontal Irregularities.  ➢ Plan (Horizontal) irregularities: Examine the plan view of the structure

TABLE 12.3-1 HORIZONTAL STRUCTURAL IRREGULARITIES

	Irregularity Type and Description
1a.	**Torsional Irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.
1b.	**Extreme Torsional Irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.
2.	**Reentrant Corner Irregularity** is defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction.
3.	**Diaphragm Discontinuity Irregularity** is defined to exist where there are diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50% of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50% from one story to the next.
4.	**Out-of-Plane Offsets Irregularity** is defined to exist where there are discontinuities in a lateral force-resistance path, such as out-of-plane offsets of the vertical elements.
5.	**Nonparallel Systems-Irregularity** is defined to exist where the vertical lateral force-resisting elements are not parallel to or symmetric about the major orthogonal axes of the seismic force–resisting system.

TABLE 4-12 PLAN IRREGULARITIES

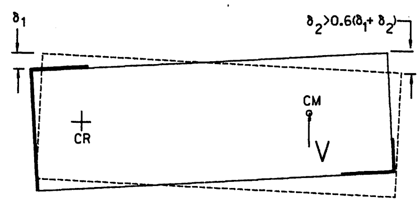
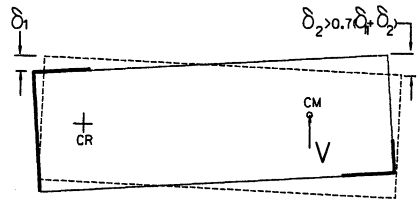
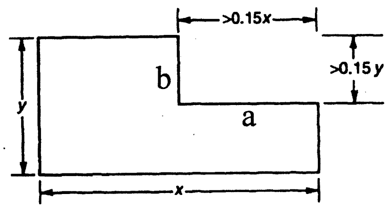
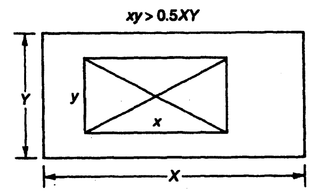
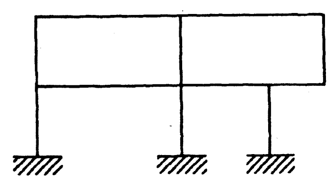
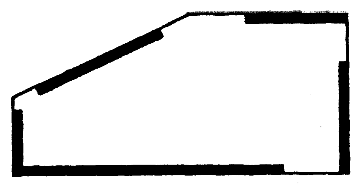
#	Type	Definition	Illustration
1a	***Torsional Irregularity***	➢ The maximum story drift ate one end is more than 1.2 times the average of the story drifts of the two end of the structure. $\delta_2 \rangle 1.2(\frac{\delta_1+\delta_2}{2}) = 0.6(\delta_1+\delta_2)$	 Ia. Torsional
1b	***Extreme Torsional Irregularity***	➢ The maximum story drift at one end is more than 1.4 times the average of the story drifts of the two end of the structure. $\delta_2 \rangle 1.4\left(\frac{\delta_1+\delta_2}{2}\right) = 0.7(\delta_1+\delta_2)$	 Ib. Torsional
2	***Re-entrant corners***	➢ Where both projections of the structure beyond a re-entrant corner are greater than 15 % of the plan dimension of the structure in the given direction. projection "a" > 0.15 x, and projection "b" > 0.15 y	 2. Reentrant Corner
3	***Diaphragm Discontinuity***	➢ Area of opening "xy" > 0.5 of the area "XY", **OR** ➢ Changes in effective diaphragm stiffness >50% from one story to the next.	 3. Diaphragm Discontinuity
4	***Out-of- plane Offset***	➢Discontinuities in lateral-force-resisting path, such as out-of-plane offsets of vertical elements.	 4. Offset
5	***Nonparallel systems***	➢Vertical lateral-force-resisting elements are not parallel to **OR** symmetric about the major orthogonal axes of the lateral-force-resisting system.	 5. Non Parallel System

TABLE 4-13 ADDITIONAL REQUIREMENTS FOR PLAN IRREGULARITIES

Type	Irregularity	Seismic Design Category (SDC)	Additional design requirements
1a	Torsional (Applies only to structures with rigid or semirigid diaphrams)	B, C, D, E, F	➢Design required using a three-dimensional representation (12.7.3)
		C, D, E, F	➢Amplification of torsion required (12.8.4.3). ➢Story drift is determined from the largest difference in deflection along the top and bottom edges (12.12.1)
		D, E, F	➢Equivalent lateral force procedure **NOT** permitted (12.6). ➢An increase of 25% is required in the calculated design forces for connections of diaphragms to collectors and vertical elements, and for connections of collectors to vertical elements (12.3.3.4).
lb	Extreme torsional (Applies only to Structures with rigid or semirigid diaphragms)	B, C, D	➢Design required using a three-dimensional representation (12.7.3)
		C, D	➢Amplification of torsion (A_x) required (12.8.4.3). ➢Story drift is determined from the largest difference in deflection along the top and bottom edges (12.12.1).
		D	➢Equivalent lateral force procedure **NOT** permitted (12.6). ➢An increase of 25% is required in the calculated design forces for connections of diaphragms to collectors and vertical elements, and for connections of collectors to vertical elements (12.3.3.4).
		E, F	➢Not permitted (12.3.3.1).
2	Re-entrant comers*	D, E, F	➢An increase of 25% is required in the calculated design forces for connections of diaphragms to collectors and vertical elements, and for connections of collectors to vertical
3	Diaphragm discontinuity*	D, E, F	➢An increase of 25% is required in the calculated design forces for connections of diaphragms to collectors and vertical elements, and for connections of collectors to vertical elements (12.3.3.4).

TABLE 4-13 ADDITIONAL REQUIREMENTS FOR PLAN IRREGULARITIES (Cont.)

4	Out-of-plane offsets*	B, C, D, E, F	➢ Columns supporting discontinuous vertical elements shall be designed for the special seismic load combinations of ASCE Section 12.4.3.2 (12.3.3.3).
		B, C, D, E, F	➢ Design required using a three-dimensional representation (12.7.3)
		D, E, F	➢ An increase of 25% is required in the calculated design forces for connections of diaphragms to collectors and vertical elements, and for connections of collectors to vertical elements (12.3.3.4).
5	Nonparallel system*	B, C, D, E, F	➢ Design required using a three-dimensional representation(12.7.3)
		C, D, E ,F	➢ Design required for 100% of forces for one direction plus 30% of the forces for the perpendicular direction (12.5.3).

* Per §12.6 ASCE 7-05, a modal response analysis procedure (§12.9) is required for buildings in SDC "D", "E" and "F" if $T \geq 3.5T_s$

§12.3.2.2 Vertical Irregularity. Structures having one or more of the irregularity types listed in Table 12.3-2 shall be designated as having vertical irregularity. Such structures assigned to the seismic design categories listed in Table 12.3-2 shall comply with the requirements in the sections referenced in that table.

EXCEPTIONS:

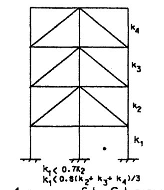
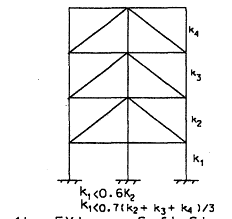
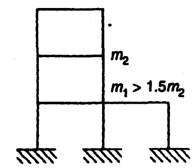
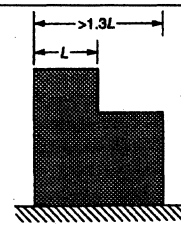
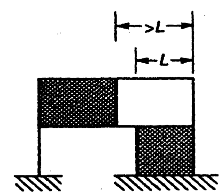
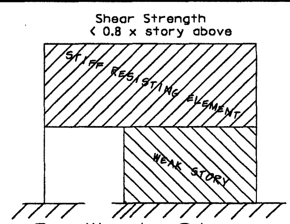
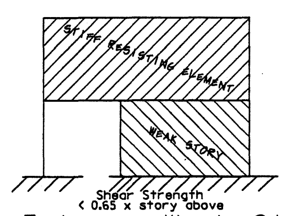
1. *Vertical structural irregularities of Types 1a, 1b, or 2 in Table 12.3-2 do not apply where no story drift ratio under design lateral seismic force is greater than 130 percent of the story drift ratio of the next story above. Torsional effects need not be considered in the calculation of story drifts. The story drift ratio relationship for the top two stories of the structure are not required to be evaluated.*

2. *Irregularities Types 1a, 1b, and 2 of Table 12.3-2 are not required to be considered for one-story buildings in any seismic design category or for two-story buildings assigned to Seismic Design Categories B, C, or D.*

TABLE 12.3-2 VERTICAL STRUCTURAL IRREGULARITIES

	Irregularity Type and Description
1a.	**Stiffness-Soft Story Irregularity** is defined to exist where there is a story in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above.
1b.	**Stiffness-Extreme Soft Story Irregularity** is defined to exist where there is a story in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.
2.	**Weight (Mass) Irregularity** is defined to exist where the effective mass of any story is more than 150% of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.
3.	**Vertical Geometric Irregularity** is defined to exist where the horizontal dimension of the seismic force–resisting system in any story is more than 130% of that in an adjacent story.
4.	**In-Plane Discontinuity in Vertical Lateral Force-Resisting Element Irregularity** is defined to exist where an in-plane offset of the lateral force-resisting elements is greater than the length of those elements or there exists a reduction in stiffness of the resisting element in the story below.
5a.	**Discontinuity in Lateral Strength–Weak Story Irregularity** is defined to exist where the story lateral strength is less than 80% of that in the story above. The story lateral strength is the total lateral strength of all seismic-resisting elements sharing the story shear for the direction under consideration.
5b.	**Discontinuity in Lateral Strength–Extreme Weak Story Irregularity** is defined to exist where the story lateral strength is less than 65% of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.

TABLE 4-14 VERTICAL IRREGULARITIES

#	Type	Definition	Illustration
1a	***Stiffness irregularity-soft story***	Soft story exists when: ➢ lateral stiffness < 70% of story <u>above</u>, **OR** ➢ lateral stiffness < 80% of average stiffness of <u>3 stories above.</u>	 1a. soft Story
1b	***Stiffness irregularity-Extreme Soft Story***	Soft story exists when: ➢ lateral stiffness < 60% of story <u>above</u>, **OR** ➢ lateral stiffness < 70% of average stiffness of <u>3 stories above.</u>	 1b. EXtreme Soft Story
2	***Weight (mass irregularity)***	➢ Story mass > 150% (adjacent story mass) (***<u>exception</u>:*** a roof that is lighter than the floor below need not considered)	 2. Mass Irregularity
3	***Vertical geometric irregularity***	➢ Horizontal dimension of lateral –force-resisting system > 130 % of that in adjacent story. (***<u>exception</u>:*** one story penthouses need not be considered)	 3. Vertical Irregularity
4	***In-plane discontinuity in vertical lateral - force-resisting element)***	➢ In-plane offset of lateral-force-resisting element > lengths of those elements, or reduction in stiffness of resisting elements in story below.	 4. Discontinuity
5a	***Discontinuityin lateral strength Weak Story***	➢ Weak story strength < 80 % of story strength <u>above.</u> ➢ Story strength = total strength of seismic-force-resisting elements sharing story shear for direction under consideration.	 5a. Weak Story
5b	***Discontinuity in lateral Extreme Weak Story***	➢ Weak story strength < 65 % of story strength <u>above.</u> ➢ Story strength = total strength of seismic-force-resisting elements sharing story shear for direction under consideration.	 5b. Extreme Weak Story

§12.3.3 Limitations and Additional Requirements for Systems with Structural Irregularities.

§12.3.3.1 Prohibited Horizontal and Vertical Irregularities for Seismic Design Categories D through F. Structures assigned to Seismic Design Category E or F having horizontal irregularity Type 1b of Table 12.3-1 or vertical irregularities Type 1b, 5a, or 5b of Table 12.3-2 shall not be permitted. Structures assigned to Seismic Design Category D having vertical irregularity Type 5b of Table 12.3-2 shall not be permitted.

12.3.3.2 Extreme Weak Stories. Structures with a vertical irregularity Type 5b as defined in Table 12.3-2, shall not be over two stories or 30 ft (9 m) in height.

EXCEPTION: The limit does not apply where the "weak" story is capable of resisting a total seismic force equal to Ω_0 times the design force prescribed in Section 12.8.

§12.3.3.3 Elements Supporting Discontinuous Walls or Frames. Columns, beams, trusses, or slabs supporting discontinuous walls or frames of structures having horizontal irregularity Type 4 of Table 12.3-1 or vertical irregularity Type 4 of Table 12.3-2 shall have the design strength to resist the maximum axial force that can develop in accordance with the load combinations with overstrength factor of Section 12.4.3.2. The connections of such discontinuous elements to the supporting members shall be adequate to transmit the forces for which the discontinuous elements were required to be designed.

§12.3.3.4 Increase in Forces Due to Irregularities for Seismic Design Categories D through F. For structures assigned to Seismic Design Category D, E, or F and having a horizontal structural irregularity of Type 1a, 1b, 2, 3, or 4 in Table 12.3-1 or a vertical structural irregularity of Type 4 in Table 12.3-2, the design forces determined from Section 12.8.1 shall be increased 25 percent for connections of diaphragms to vertical elements and to collectors and for connections of collectors to the vertical elements. Collectors and their connections also shall be designed for these increased forces unless they are designed for the load combinations with overstrength factor of Section 12.4.3.2, in accordance with Section 12.10.2.1.

TABLE 4-15 ADDITIONAL DESIGN REQUIREMENTS FOR VERTICAL IRREGULARITIES

Type	Irregularity	Seismic Design Category (SDC)	Additional design requirements
1a	Soft story	D, E, F	Equivalent lateral force procedure **NOT** permitted (§12.6 ASCE 7-05).
1b	Extreme soft story	D	Equivalent lateral force procedure **NOT** permitted (§12.6 ASCE 7-05)
		E, F	**NOT** permitted (12.3.3.1).
2	Mass	D, E, F	Equivalent lateral force procedure **NOT** permitted (§12.6 ASCE 7-05)
3	Geometric	D, E, F	Equivalent lateral force procedure **NOT** permitted (§12.6 ASCE 7-05)
4	Discontinuity	B, C, D, E, F	Columns supporting discontinuous vertical elements shall be designed for the special seismic load combinations of ASCE Section 12.4.3.2 (12.3.3.3).
		D, E, F	➢ An increase of 25% is required in the calculated design forces for connections of diaphragms to collectors and vertical elements, and for connections of collectors to vertical elements (12.3.3.4 ASCE 7-05). ➢ Equivalent lateral force procedure **NOT** permitted for structures with $T \geq 3.5\ T_S$ (§12.6 ASCE 7-05).
5a	Weak story	D	➢ Equivalent lateral force procedure **NOT** permitted for structures with $T \geq 3.5\ T_S$ (§12.6 ASCE 7-05).
		E, F	➢ **NOT** permitted (12.3.3.1 ASCE 7-05).
5b	Extreme weak story	D, E, F	➢ **NOT** permitted (12.3.3.1 ASCE 7-05)
		B, C	➢ Maximum height two stories unless designed for Ω_0 forces (12.3.3.2 ASCE 7-05)

4.5 SELECTION OF LATERAL FORCE PROCEDURE

The selection of the lateral force procedure permitted by the ASCE 7-05 is based on:

i) The Seismic Design Category (SDC) assigned to the structure,
ii) The structural characteristics which include:

a) Structure period
b) Number of stories
c) Structural irregularities (horizontal or vertical)
d) Construction type
e) Occupancy Category

The following table lists the permitted analytical procedures.

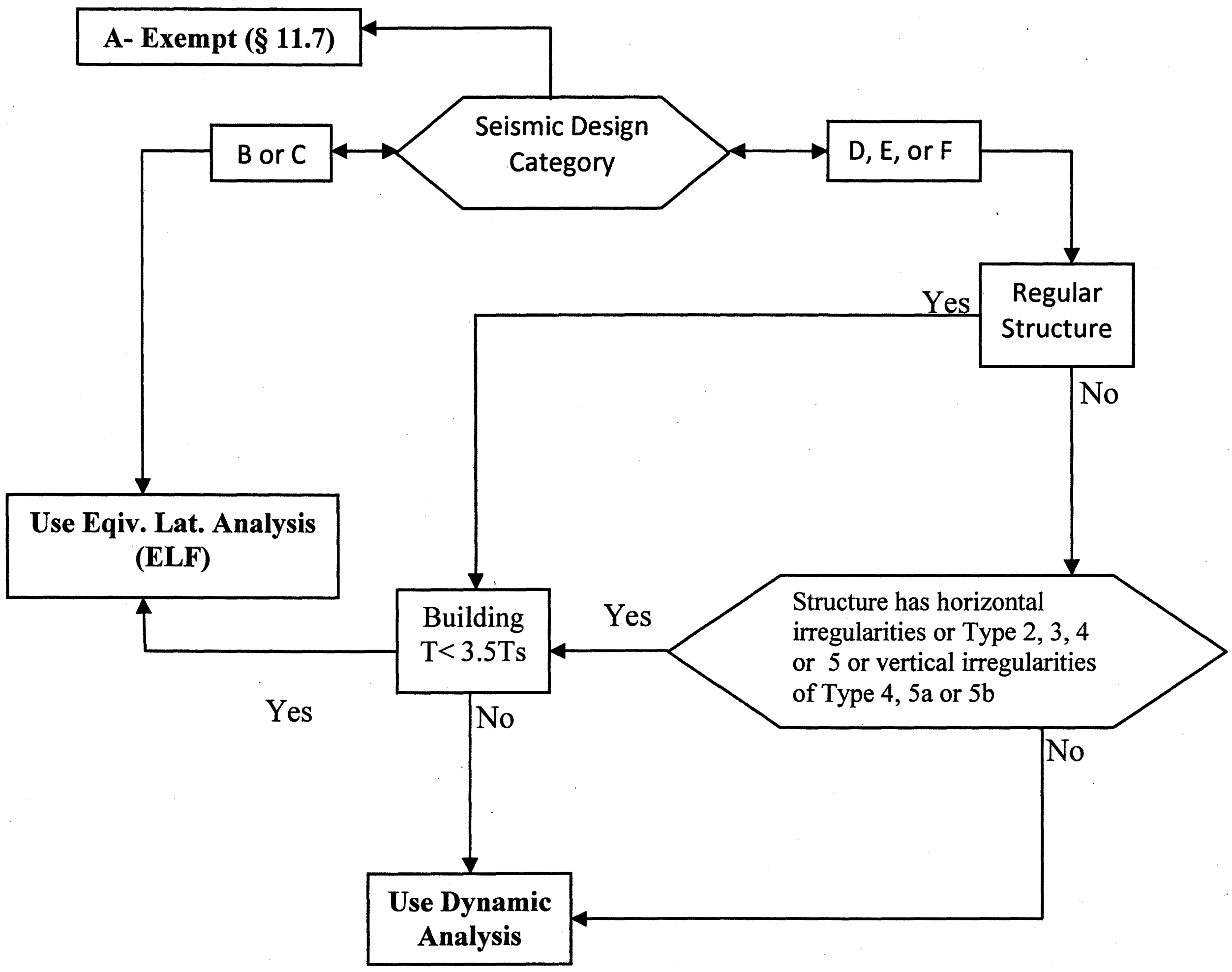
TABLE 12.6-1 PERMITTED ANALYTICAL PROCEDURES

Seismic Design Category	Structural Characteristics	Equivalent Lateral Force Analysis (ELF) Section 12.8	Modal Response Spectrum Analysis Section 12.9	Seismic Response History Procedures Chapter 16
B, C	Occupancy Category I or II buildings of light-framed construction not exceeding 3 stories in height	P	P	P
	Other Occupancy Category I or II buildings not exceeding 2 stories in height	P	P	P
	All other structures	P	P	P
D, E, F	Occupancy Category I or II buildings of light-framed construction not exceeding 3 stories in height	P	P	P
	Other Occupancy Category I or II buildings not exceeding 2 stories in height	P	P	P
	Regular structures with $T < 3.5T_s$ and all structures of light frame construction	P	P	P
	Irregular structures with $T < 3.5T_s$ and having only horizontal irregularities Type 2, 3, 4, or 5 of Table 12.3-1 or vertical irregularities Type 4, 5a, or 5b of Table 12.3-2	P	P	P
	All other structures	NP	P	P

***NOTE*: P: Permitted; NP: Not Permitted**

Selection of the Equivalent Lateral Force – ELF and Dynamic Analysis procedures

Sources: *Table 12.6-1 (Permitted Analytical Procedures), Table 12.3-1 (Horizontal Irregularities) & Table 12.3.2 (Vertical Irregularities)*



Type	Description of Horizontal Irregularity
2	Reentrant Corner
3	Diaphragm Discontinuity
4	Out-of-plane Offsets
5	Nonparallel Systems

Type	Description of Vertical Irregularity
4	In-Plane Discontinuity
5a	Discontinuity in Lateral Strength (Weak Story)
5b	Discontinuity in Lateral Strength (Extreme Weak Story)

$$T_S = \frac{S_{D1}}{S_{DS}}$$

4.6- ROAD MAP FOR EQUIVALENT LATERAL FORCE (ELF) PROCEDURE

1- Site classification characteristics: A, B, C, D, E & F Tables 1613.5.2 & 1613.5.5

2- Maximum considered earthquake spectral response accelerations S_1 & S_s
Three sources: 1) usgs.gov 2)NEHRP 3)Maps-2007CBC, 2006 IBC& ASCE7-05

3- Site Coefficients F_a & F_v Tables 1613.5.3(1) & 1613.5.3(2)

4- Adjusted maximum considered earthquake spectral response accelerations

$S_{MS} = F_a S_s$ (Equation 16-37)

$S_{M1} = F_v S_1$ (Equation 16-38)

5- Design spectral response acceleration parameters

$S_{DS} = 2/3\ S_{MS}$ (Equation 16-39)

$S_{D1} = 2/3\ S_{M1}$ (Equation 16-40)

6- Period of the structure T
Equations: 12.8-7, 12.8-8, 12.8-9, 15.4-6 with the limitation

7- Occupancy importance factor, I, Table 11.5-1 ASCE 7-05

8-Seismic design category (SDC), Tables 1613.5.6(1) & 1613.5.6(2)

9- Lateral-force-resisting systems (LFRS), § 11.2 & Table 12.2-1, ASCE 7-05

10- Response modification coefficient, R, Table 12.2-1, ASCE 7-05

11- Seismic dead load, W, § 12.7-2, ASCE 7-05

12- Seismic Response Coefficient, C_s, §12.8 ASCE7-05

13- Seismic Base Shear, V §12.8 ASCE7-05

14- Vertical Distribution of Base Shear, F_x §12.8.3 ASCE7-05

15- Horizontal and Vertical Components of E, E_h & E_v §12.4 ASCE7-05

16- Redundancy Factor, ρ, §12.3.4 ASCE 7-05

17- Overstrength Factor : Ω_0, Table 12.2-1, ASCE 7-05

18- Deflection Control : $\delta_x = C_d \delta_{xe}/I$, Table 12.2-1 & §12.12.1

19- Combinations of Systems Table 12.2-1 & §12.2.3.1 & 12.2.3.2

1- Site classification characteristics: A, B, C, D, E & F Table 1613.5.2 (CBC)

§1613.5.2 Site class definitions. Based on the site soil properties, the site shall be classified as either Site Class A, B, C, D, E or F in accordance with Table 1613.5.2. When the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used unless the building official or geotechnical data determines that Site Class E or F soil is likely to be present at the site. There are three possibilities that the site can be given in the problem statement as follows:

1- The site class will be stated in the problem statement e.g. " the site class is C".
2- Properties are given as shear wave velocity, SPT, undrained shear strength and the Table shown below will be used to determine the soil type (site class)
3- Say nothing or "the soil properties are not know in sufficient detail", then site "D" is used unless the building official or geotechnical data determines that Site Class E or F soil is likely to be present at the site.

TABLE 1613.5.2 SITE CLASS DEFINITIONS

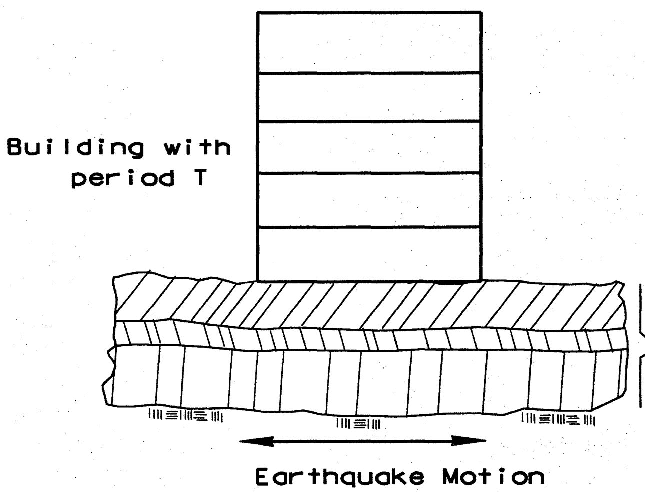
SITE CLASS	SOIL PROFILE NAME	AVERAGE PROPERTIES IN TOP 100 feet, SEE SECTION 1613.5.5		
		Soil shear wave velocity, $\bar{V}_s,(ft/s)$	Standard penetration resistance, $\bar{N}$	Soil undrained shear strength, $\bar{S}_u,(psf)$
A	Hard rock	$\bar{V}_s > 5{,}000$	N/A	N/A
B	Rock	$2{,}500 < \bar{V}_s \le 5{,}000$	N/A	N/A
C	Very dense soil and soft rock	$1{,}200 < \bar{V}_s \le 2{,}500$	$\bar{N} > 50$	$\bar{S}_u \ge 2{,}000$
D	Stiff soil profile	$600 \le \bar{V}_s \le 1{,}200$	$15 \le \bar{N} \le 50$	$1{,}000 \le \bar{S}_u \le 2{,}000$
E	Soft soil profile	$\bar{V}_s < 600$	$\bar{N} < 15$	$\bar{S}_u < 1{,}000$
E	------	Any profile with more than 10 feet of soil having the following characteristics: 1. Plasticity index PI > 20, 2. Moisture content w ≥ 40%, and 3. Undrained shear strength $\bar{S}_u < 500\,psf$		
F	------	Any profile containing soil having one or more of the following characteristics: 1. Soil vulnerable to potential failure or collapse under seismic loading such as liquefiable, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays (H >10 feet of peat and /or highly organic clay where H= thickness of soil) 3. Very high plasticity clays (H > 25 feet with plasticity index PI >75) 4. Very thick soft/medium stiff clays (H >120 feet)		

➢ ASCE 7-05 Section 20.1 includes the following statement: *"Where site-specific data are not available to a depth of 100 feet, appropriate soil properties are permitted to be estimated by the registered design professional preparing the soils report based on known geologic conditions.*

Three important parameters related to the site and soil type should be discussed:

1- Site period
2- Soil liquefaction
3- General response spectrum for soils (see Figure 4-8)

1- Site period: Each site has its own period which depends on many factors including the type of the soil. The site period has a direct and significant effect on the damage of a structure due to an earthquake. In fact if the frequency of the site coincide with the frequency of an earthquake, site movement can be increased many folds which increase the damage of any structure. This phenomenon known as resonance. Therefore, the soil-structure resonance is the term used to refer to the amplification of earthquake effects caused by local soil conditions. In 1985 of Mexico City earthquake and 1989 Loma Prieta earthquake, the resonance was a major factor of the level of damage occurred in both locations.



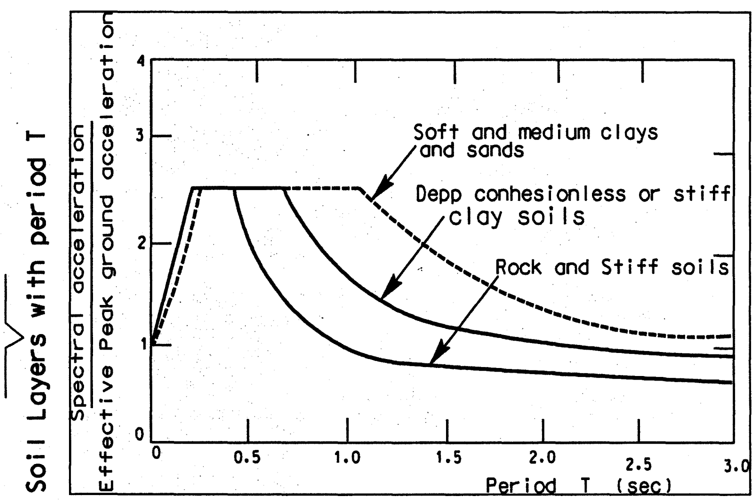


Figure 4-7 Soil-Structure Interaction **Figure 4-8** Normalized Response Spectra

*2- Soil liquefaction: Liquefaction is a process by which sediment below water the table temporarily lose strength and behave as viscous liquid rather than a solid. The type of sediments most susceptible are clay-free sand (cohesionless) and silts. When the seismic waves, primarily shear waves, passing through saturated granular layers, distort granular structure, and cause loosely packed groups of particles to collapse. These collapses increase the pore-water pressure between the grains if drainage cannot occur. If the pore-water pressure rises to a level approaching the weight of the overlying soil, the granular layer temporarily behaves as a viscous liquid rather than a solid.

2- Maximum considered earthquake spectral response accelerations S_1 & S_s

Three sources: 1) usgs.gov 2)NEHRP 3)Maps-2007CBC, 2006 IBC& ASCE7-05

The mapped maximum considered earthquake (MCE) spectral response accelerations at short periods, S_S, and at 1-second period, S_1. These values can be determined using either:

1) USGS website at http://earthquake.usgs.gov/research/hazmaps/. The U.S. Geological Survey (USGS) has prepared an Internet calculation tool for obtaining seismic design parameters using the same data that was used to prepare the ground motion maps published in the 2006 IBC, ASCE 7-05. By inputting the longitude and latitude of the building location, this method provides for a more accurate and reliable determination of S_S and S_1.

2) *2003 NEHRP Provision*- FEMA 450 CD also contains this calculation tool.

3) 2006 IBC Figures 1613.5(1) through 1613.5(14)[ASCE 7-05 Figures 22-1 through 22-20]

3- Site Coefficients F_a & F_v Tables 1613.5.3(1) & 1613.5.3(2)

Site Coefficients F_a & F_v will be determined according the following two tables

VELOCITY
ACCELERATION

TABLE 1613.5.3 (1)
VALUES OF SITE COEFFICIENT, $F^a{}_a$

Site Class	Mapped Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \ge 1.25$
BEST 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
WORST F	Note b	Note b	Note b	Note b	Note b

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at short periods, S_S

b. Values shall be determined in accordance with Section 11.4.7 of ASCE 7.

§11.4.7 Site-Specific Ground Motion Procedures. The site specific ground motion procedures set forth in Chapter 21 are permitted to be used to determine ground motions for any structure. A site response analysis shall be performed in accordance with Section 21.1 for structures on Site Class F sites, unless the exception to Section 20.3.1 is applicable. For seismically isolated structures and for structures with damping systems on sites with S_1 greater than or equal to 0.6, a ground motion hazard analysis shall be performed in accordance with Section 21.2.

TABLE 1613.5.3(2)
VALUES OF SITE COEFFICIENT, F^a_v

Site Class	Mapped Spectral Response Acceleration Parameter at 1-Second Period				
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \ge 0.5$
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	Note b	Note b	Note b	Note b	Note b

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at 1-second period, S_1 .
b. Values shall be determined in accordance with Section 11.4.7of ASCE 7.

4- Adjusted maximum considered earthquake spectral response accelerations, S_{MS} & S_{M1}

§1613.5.3 Site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters. The maximum considered earthquake spectral response acceleration for short periods, S_{MS} and at 1-second period, S_{M1} adjusted for site class effects shall be determined by Equations 16-37 and 16-38, respectively:

$$S_{MS} = F_a S_s \quad \text{(Equation 16-37)}$$

$$S_{M1} = F_v S_1 \quad \text{(Equation 16-38)}$$

where:
F_a = Site coefficient defined in Table 1613.5.3(1)
F_v = Site coefficient defined in Table 1613.5.3(2)
S_s = The mapped spectral accelerations for short periods as determined in Section 1613.5.1.
S_1 = The mapped spectral accel. for a 1- second period as determined in Section 1613.5.1.

5- Design spectral response acceleration parameters S_{DS} & S_{D1}

§1613.5.4 Design spectral response acceleration parameters. Five-percent damped design spectral response acceleration at short periods, S_{DS} and at 1- second period, S_{D1} shall be determined from Equations 16-39 and 16-40, respectively:

$$S_{DS} = 2/3\, S_{MS} \quad \text{(Equation 16-39)}$$

$$S_{D1} = 2/3\, S_{M1} \quad \text{(Equation 16-40)}$$

Where:
S_{MS} = The maximum considered earthquake spectral response accelerations for short period as determined in Section 1613.5.3
S_{M1} = The maximum considered earthquake spectral response accelerations for 1-second period as determined in Section 1613.5.3

Table 4-18 Design Spectral Acceleration Parameter for 0.2 -Second Period (S_{DS})

S_s	Site Class					
	A	B	C	D	E	F
0.05	0.027	0.033	0.040	0.050	0.080	Site-Specific Ground Motion Procedure Required-Chapter 21-ASCE 7-05
0.10	0.053	0.067	0.080	0.110	0.170	
0.15	0.080	0.100	0.120	0.160	0.250	
0.20	0.107	0.133	0.160	0.210	0.330	
0.25	0.133	0.167	0.200	0.270	0.420	
0.30	0.160	0.200	0.240	0.310	0.450	
0.35	0.187	0.233	0.280	0.350	0.480	
0.40	0.213	0.267	0.320	0.390	0.510	
0.45	0.240	0.300	0.360	0.430	0.540	
0.50	0.267	0.333	0.400	0.470	0.570	
0.55	0.293	0.367	0.430	0.490	0.570	
0.60	0.320	0.400	0.460	0.520	0.580	
0.65	0.347	0.433	0.490	0.550	0.590	
0.70	0.373	0.467	0.520	0.570	0.590	
0.75	0.400	0.500	0.550	0.600	0.600	
0.80	0.427	0.533	0.570	0.630	0.600	
0.85	0.453	0.567	0.600	0.650	0.600	
0.90	0.480	0.600	0.620	0.680	0.600	
0.95	0.507	0.633	0.640	0.710	0.600	
1.00	0.533	0.667	0.670	0.730	0.600	
1.05	0.560	0.700	0.700	0.750	0.630	
1.10	0.587	0.733	0.730	0.770	0.660	
1.15	0.613	0.767	0.770	0.790	0.690	
1.20	0.640	0.800	0.800	0.810	0.720	
1.25	0.667	0.833	0.830	0.830	0.750	
1.30	0.693	0.867	0.870	0.870	0.780	
1.35	0.720	0.900	0.900	0.900	0.810	
1.40	0.747	0.933	0.930	0.930	0.840	
1.45	0.773	0.967	0.970	0.970	0.870	
1.50	0.800	1.000	1.000	1.000	0.900	
1.55	0.827	1.033	1.030	1.030	0.930	
1.60	0.853	1.067	1.070	1.070	0.960	
1.65	0.880	1.100	1.100	1.100	0.990	
1.70	0.907	1.133	1.130	1.130	1.020	
1.75	0.933	1.167	1.170	1.170	1.050	
1.80	0.960	1.200	1.200	1.200	1.080	
1.85	0.987	1.233	1.230	1.230	1.110	
1.90	1.013	1.267	1.270	1.270	1.140	
2.00	1.067	1.333	1.330	1.330	1.200	
2.10	1.120	1.400	1.400	1.400	1.260	
2.20	1.173	1.467	1.470	1.470	1.320	
2.30	1.227	1.533	1.530	1.530	1.380	
2.40	1.280	1.600	1.600	1.600	1.440	
2.50	1.333	1.667	1.670	1.670	1.500	
2.60	1.387	1.733	1.730	1.730	1.560	
2.70	1.440	1.800	1.800	1.800	1.620	
2.80	1.493	1.867	1.870	1.870	1.680	
2.90	1.547	1.933	1.930	1.930	1.740	
3.00	1.600	2.000	2.000	2.000	1.800	

Table 4-19 Design Spectral Acceleration Parameter for 1-Second Period (S_{D1})

S_1	Site Class					
	A	B	C	D	E	F
0.10	*0.053*	*0.067*	*0.110*	*0.160*	*0.230*	Site-Specific Ground Motion Procedure Required-Chapter 21-ASCE 7-05
0.12	0.064	0.080	0.130	0.180	0.270	
0.14	0.075	0.093	0.150	0.200	0.310	
0.16	0.085	0.107	0.170	0.220	0.350	
0.18	0.096	0.120	0.190	0.250	0.390	
0.20	*0.107*	*0.133*	*0.210*	*0.270*	*0.430*	
0.22	0.117	0.147	0.230	0.290	0.450	
0.24	0.128	0.160	0.250	0.300	0.480	
0.26	0.139	0.173	0.270	0.320	0.510	
0.28	0.149	0.187	0.280	0.340	0.530	
0.30	*0.160*	*0.200*	*0.300*	*0.360*	*0.560*	
0.32	0.171	0.213	0.310	0.370	0.580	
0.34	0.181	0.227	0.330	0.390	0.590	
0.36	0.192	0.240	0.340	0.400	0.610	
0.38	0.203	0.253	0.360	0.410	0.620	
0.40	*0.213*	*0.267*	*0.370*	*0.430*	*0.640*	
0.42	0.224	0.280	0.390	0.440	0.670	
0.44	0.235	0.293	0.400	0.460	0.700	
0.46	0.245	0.307	0.410	0.470	0.740	
0.48	0.256	0.320	0.420	0.490	0.770	
0.50	*0.267*	*0.333*	*0.430*	*0.500*	*0.800*	
0.52	0.277	0.347	0.450	0.520	0.830	
0.54	0.288	0.360	0.470	0.540	0.860	
0.56	0.299	0.373	0.490	0.560	0.900	
0.58	0.309	0.387	0.500	0.580	0.930	
0.60	*0.320*	*0.400*	*0.520*	*0.600*	*0.960*	
0.65	0.347	0.433	0.560	0.650	1.040	
0.70	0.373	0.467	0.610	0.700	1.120	
0.75	0.400	0.500	0.650	0.750	1.200	
0.80	0.427	0.533	0.690	0.800	1.280	
0.85	*0.453*	*0.567*	*0.740*	*0.850*	*1.360*	
0.90	0.480	0.600	0.780	0.900	1.440	
0.95	0.507	0.633	0.820	0.950	1.520	
1.00	0.533	0.667	0.870	1.000	1.600	
1.05	0.560	0.700	0.910	1.050	1.680	
1.10	*0.587*	*0.733*	*0.950*	*1.100*	*1.760*	
1.15	0.613	0.767	1.000	1.150	1.840	
1.20	0.640	0.800	1.040	1.200	1.920	
1.25	0.667	0.833	1.080	1.250	2.000	
1.30	*0.693*	*0.867*	*1.130*	*1.300*	*2.080*	

6- Period of the structure T

Equations: 12.8-7, 12.8-8, 12.8-9, 15.4-6 with the limitation

The period of the structure "T" is the length of time (in seconds) that it takes for one complete cycle of vibration as shown and described in Figure 4-9. The period is a unique characteristic of each structure and its a function of mass and stiffness of the system. It represents the fundamental period of vibration (first mode) of the structure in the direction of analysis under consideration.

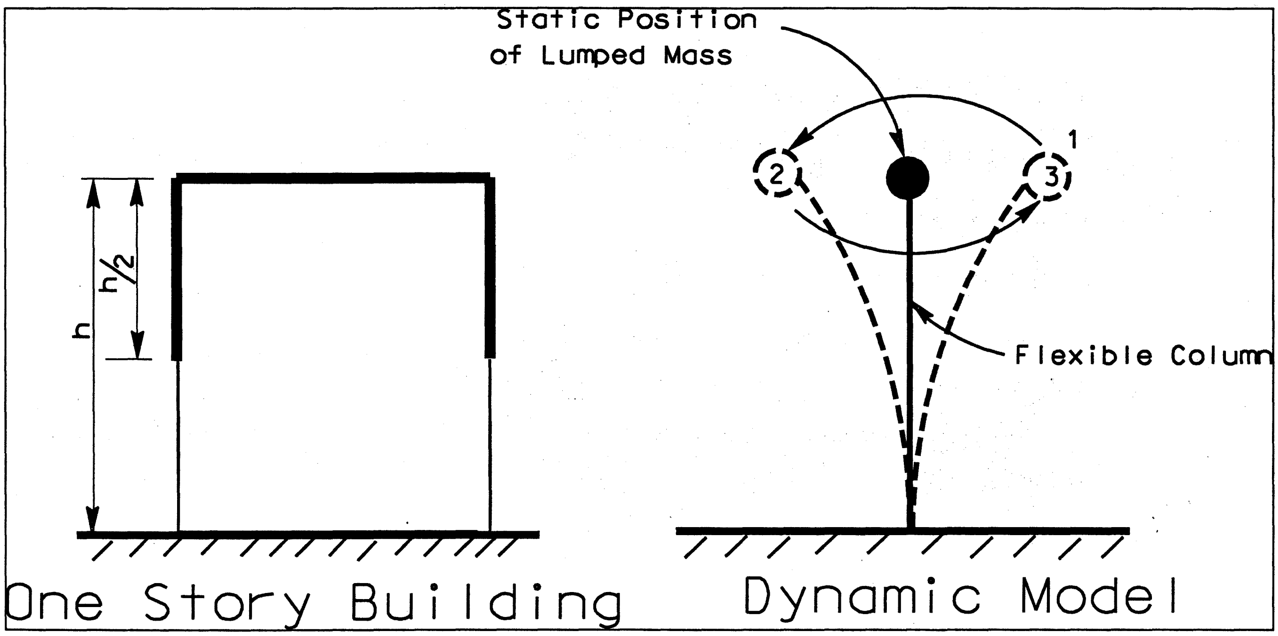


Figure 4-9 Structural Period "T"

§12.8.2 Period Determination. The fundamental period of the structure, T, in the direction under consideration shall be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The fundamental period, T, shall not exceed the product of the coefficient for upper limit on calculated period (C_u) from Table 12.8-1 and the approximate fundamental period, T_a , determined in accordance with section 12.8.2.1. As an alternative to performing an analysis to determine the fundamental period, T, it is permitted to use the approximate building period, T_a, calculated in accordance with Section 12.8.2.1, directly.

§12.8.2.1 Approximate Fundamental Period. The approximate fundamental period (T_a), in s, shall be determined from the following equation:

$$T_a = C_t h_n^x \qquad (12.8\text{-}7)$$

where h_n is the height in ft above the base to the highest level of the structure and the coefficients C_t and x are determined from Table 12.8-2.

TABLE 12.8-2 VALUES OF APPROXIMATE PERIODPARAMETERS C_t AND x

Structure Type	C_t	x
Moment-resisting frame systems in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting where subjected to seismic forces:		
Steel moment-resisting frames	0.028 (0.0724)[a]	0.8
Concrete moment-resisting frames	0.016 (0.0466)[a]	0.9
Eccentrically braced steel frames	0.03 (0.0731)[a]	0.75
All other structural systems	0.02 (0.0488)[a]	0.75

[a]metric equivalents are shown in parentheses.

Alternatively, it is permitted to determine the approximate fundamental period (T_a), in s, from the following equation for structures not exceeding 12 stories in height in which the seismic force–resisting system consists entirely of concrete or steel moment resisting frames and the story height is at least 10 ft (3 m):

$$T_a = 0.1\,N \qquad (12.8\text{-}8)$$

where N = number of stories.

The approximate fundamental period, T_a, in s for masonry or concrete shear wall structures is permitted to be determined from Eq. 12.8-9 as follows:

$$T_a = \frac{0.0019}{\sqrt{C_w}} h_n \qquad (12.8\text{-}9)$$

where h_n is as defined in the preceding text and C_w is calculated from Eq. 12.8-10 as follows:

$$C_w = \frac{100}{A_B} \sum_{i=1}^{x} \left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{\left[1 + 0.83\left(\frac{h_i}{D_i}\right)^2\right]} \qquad (12.8-10)$$

where

A_B = area of base of structure, ft^2

A_i = web area of shear wall "i" in ft^2

D_i = length of shear wall "i" in ft

h_i = height of shear wall "i" in ft

x = number of shear walls in the building effective in resisting lateral forces in the direction under consideration

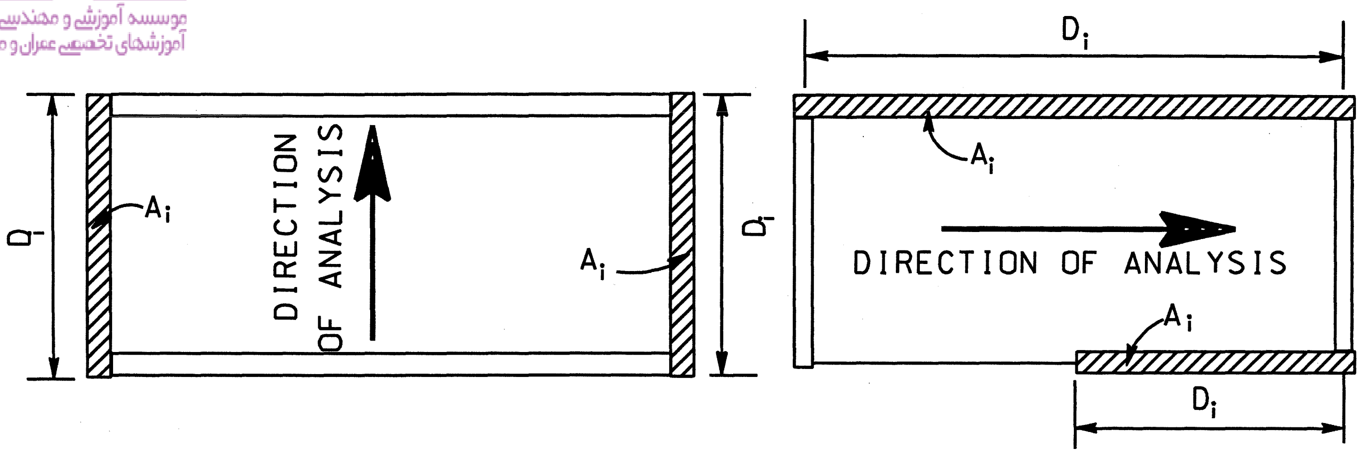


Figure 4-10 Parameters of Equation 12.8-10

The fundamental period "T" may be computed by using the following formula (Rayleigh Formula):

$$T = 2\pi \sqrt{\left(\sum_{i=1}^{n} w_i \delta_i^2\right) \div \left(g \sum_{i=1}^{n} f_i \delta_i\right)} \qquad (15.4-6)$$

It should be noted that Rayleigh Formula for determining the structural period has the following two characteristics:

1- It represents the first mode of vibration
2- For a single story structure the equation becomes:

$$T = 2\pi \sqrt{\frac{W}{gK}} \qquad (Eq.\#13 - Non\,Code\,Eq.\ Summary)$$

$$\boxed{T = 2\pi \sqrt{\left(\sum_{i=1}^{n} W_i \delta_i^2\right) \div \left(g \sum_{i=1}^{n} f_i \delta_i\right)} = 2\pi \sqrt{\frac{W.\delta^2}{g.f.\delta}} = 2\pi \sqrt{\frac{W\delta}{g.f}} = 2\pi \sqrt{\frac{W}{gK}}}$$

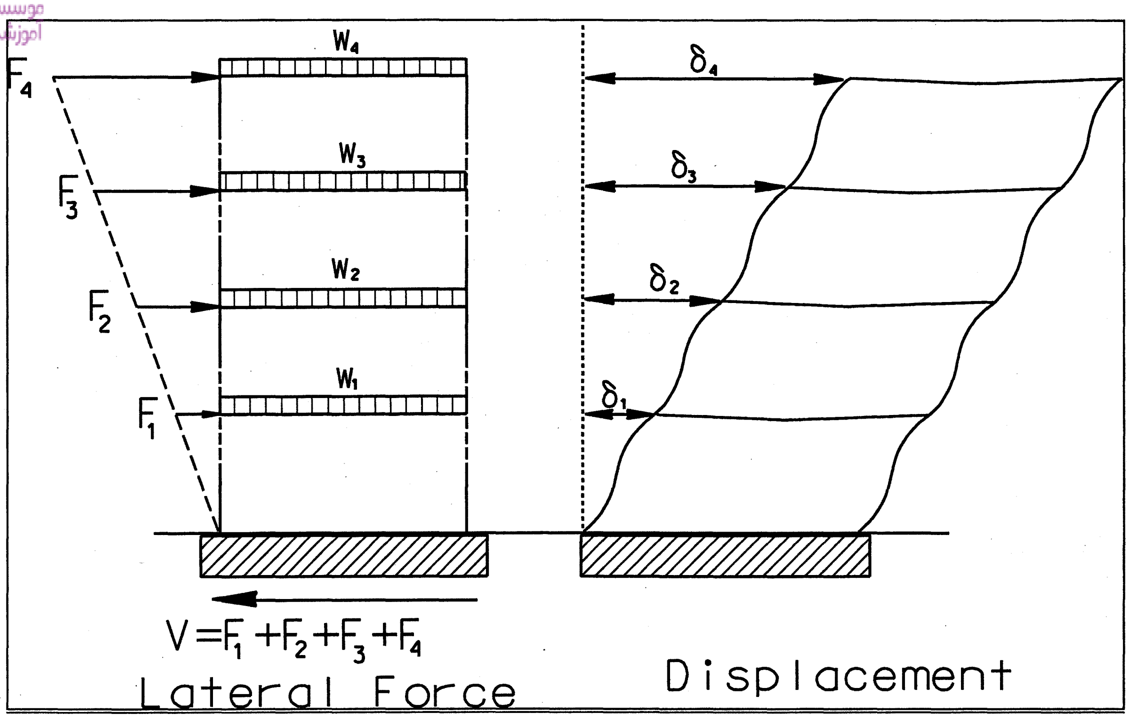


Figure 4-11 Rayleigh Formula Parameters

3-The fundamental period, T, shall not exceed the product of the coefficient for upper limit on calculated period (C_u) from Table 12.8-1 and the approximate fundamental period, T_a , determined in accordance with section 12.8.2.1

$$T \leq C_U T_a \qquad (4\text{-}1)$$

NOTE: The above limit of T is applicable only for strength calculations. For drift calculation, the actual value of T will be used without checking the limit of Equation (4-1)

TABLE 12.8-1 COEFFICIENT FOR UPPER LIMITON CALCULATED PERIOD

Design Spectral Response Acceleration Parameter at 1s, S_{D1}	Coefficient C_u
≥ 0.4	1.4
0.3	1.4
0.2	1.5
0.15	1.6
≤ 0.1	1.7

From the above Table, the relationship between fundamental period, T and the approximate fundamental period T_a as follows:

$$T \leq 1.4T_a \ \ where\ S_{D1} \geq 0.3$$

$$T \leq 1.5T_a \ \ where\ S_{D1} = 0.2$$

$$T \leq 1.6T_a \ \ where\ S_{D1} = 0.15$$

$$T \leq 1.7T_a \ \ where\ S_{D1} \leq 0.1$$

7- Occupancy importance factor, I, Table 11.5-1 ASCE 7-05

11.5.1 Importance Factor. An importance factor, I, shall be assigned to each structure in accordance with Table 11.5-1 based on the Occupancy Category from Table 1-1 [Table 1604.5 of CBC]

TABLE 1604.5
OCCUPANCY CATEGORY OF BUILDINGS AND OTHER STRUCTURES

OCCUPANCY CATEGORY	NATURE OF OCCUPANCY
I	Buildings and other structures that represent a low hazard to human life in the event of failure, including but not limited to: • Agricultural facilities. • Certain temporary facilities. • Minor storage facilities.
II	•Buildings and other structures except those listed in Occupancy Categories I, III and IV
III	Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to: • Covered structures whose primary occupancy is public assembly with an occupant load greater than 300. • Buildings and other structures with elementary school, secondary school or day care facilities with an occupant load greater than 250. • Buildings and other structures with an occupant load greater than 500 for colleges or adult education facilities. • Health care facilities with an occupant load of 50 or more resident patients, but not having surgery or emergency treatment facilities. • Jails and detention facilities. • Any other occupancy with an occupant load greater than 5,000. • Power-generating stations, water treatment for potable water, waste water treatment facilities and other public utility facilities not included in Occupancy Category IV. • Buildings and other structures not included in Occupancy Category IV containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released.
IV	Buildings and other structures designated a essential facilities, including but not limited to: • Hospitals and health care facilities having surgery or emergency treatment facilities. • Fire, rescue and police stations and emergency vehicle garages. • Designated earthquake, hurricane or other emergency shelters. • Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response. IV • Power-generating stations and other public utility facilities required as emergency backup facilities for Occupancy Category IV structures. • Structures containing highly toxic materials as defined by Section 307 where the quantity of the material exceeds the maximum allowable quantities of Table 307.1(2) • Aviation control towers, air traffic control centers and emergency aircraft hangars. • Buildings and other structures having critical national defense functions • Water treatment facilities required to maintain water pressure for fire suppression.

11.5.2 Protected Access for Occupancy Category IV. Where operational access to an Occupancy Category IV structure is required through an adjacent structure, the adjacent structure shall conform to the requirements for Occupancy Category IV structures. Where operational access is less than 10 ft from an interior lot line or another structure on the same lot, protection from potential falling debris from adjacent structures shall be provided by the owner of the Occupancy Category IV structure.

TABLE 11.5-1 IMPORTANCE FACTORS

Occupancy Category	I
I or II	1.0
III	1.25
IV	1.5

8- Seismic design category (SDC), Tables 1613.5.6(1) & 1613.5.6(2)

Occupancy Category I, II, or III structures located where the mapped spectral response acceleration parameter at 1-s period, S_1, is greater than or equal to 0.75 shall be assigned to Seismic Design Category E. Occupancy Category IV structures located where the mapped spectral response acceleration parameter at 1- s period, S_1, is greater than or equal to 0.75 shall be assigned to Seismic Design Category F.

TABLE 4-16 SDC "E" and "F" Determination

Value of S_1	Occupancy Category		
	I or II	III	IV
$S_1 \geq 0.75$	E	E	F

All other structures shall be assigned to a Seismic Design Category based on their Occupancy Category and the design spectral response acceleration parameters, S_{DS} and S_{D1}, determined in accordance with Section 11.4.4. Each building and structure shall be assigned to the more severe Seismic Design Category in accordance with Table 11.6-1 or 11.6-2, irrespective of the fundamental period of vibration of the structure, T. (select worst situation)

TABLE 11.6-1
SEISMIC DESIGN CATEGORY BASED ON SHORT PERIOD RESPONSE ACCELERATION PARAMETER

Value of S_{DS}	Occupancy Category		
	I or II	III	IV
$S_{DS} < 0.167$	A	A	A
$0.167 \leq S_{DS} < 0.33$	B	B	C
$0.33 \leq S_{DS} < 0.50$	C	C	D
$0.50 \leq S_{DS}$	D	D	D

TABLE 11.6-2
SEISMIC DESIGN CATEGORY BASED ON 1-S PERIOD RESPONSE ACCELERATION PARAMETER

Value of S_{D1}	Occupancy Category		
	I or II	III	IV
$S_{D1} < 0.067$	A	A	A
$0.067 \leq S_{D1} < 0.133$	B	B	C
$0.133 \leq S_{D1} < 0.20$	C	C	D
$0.20 \leq S_{D1}$	D	D	D

Where S_1, is less than 0.75, the Seismic Design Category is permitted to be determined from Table 11.6-1 alone where all of the following apply:

1. In each of the two orthogonal directions, the approximate fundamental period of the structure, T_a, determined in accordance with Section 12.8.2.1 is less than $0.8T_S$, where T_S is determined in accordance with Section 11.4.5.

2. In each of two orthogonal directions, the fundamental period of the structure used to calculate the story drift is less than T_S.
3. Eq. 12.8-2 is used to determine the seismic response coefficient C_S.
4. The diaphragms are rigid as defined in Section 12.3.1 or for diaphragms that are flexible, the distance between vertical elements of the seismic force-resisting system does not exceed 40 ft. Where the alternate simplified design procedure of Section 12.14 is used, the Seismic Design Category is permitted to be determined from Table 11.6-1 alone, using the value of S_{DS} determined in Section 12.14.8.1.

The following is a brief description of the classification basis used by the current Code:

1- SDC "F": This SDC includes Occupancy Category IV structures located close to a major active fault where $S_1 \geq 0.75$. This is the most vulnerable category to seismic damage. There are many severe restrictions on the systems and method of analysis for the structures that will be built in this SDC.

2- SDC "E": This SDC includes Occupancy Category I, II & III structures located close to a major active fault where $S_1 \geq 0.75$. This is also vulnerable category to seismic damage. There are many severe restrictions on the systems and method of analysis for the structures that will be located in this SDC.

3- SDC "D": This SDC includes Occupancy Category I, II , III & IV structures located in high seismic area but not close to a major active fault. There are some restrictions on the structural systems and method of analysis for the structures that will be built in this SDC. For example, dynamic analysis is required for irregular structures.

4- SDC "C": This SDC includes Occupancy Category IV structures located in moderate seismic areas and Occupancy Categories I, II and III in somewhat severe areas.

5- SDC "B": This SDC includes Occupancy Categories I, II & III structures located in moderate seismic areas.

6- SDC "A": This SDC includes Occupancy Categories I, II, III & IV structures located in least vulnerable seismic area where $S_{DS} < 0.167$ & $S_{D1} < 0.067$ where a minor ground movement is expected.

Determination of "SDC" At a Glance

Step 1: Determine if exceptions apply §1613.1 (4 exceptions)
Step 2: Determine S_s & S_1 (*www.USGS.GOV or NEHRP or Maps*)
Step 3: Determine Soil type (Table 1613.5.2 "A" through "F")
Step 4: Determine S_{DS} & S_{D1} (Equations 16-39 & 16-40)
Step 5: Determine Occupancy Category (I, II, III & IV from Table 1604.5)
Step 6: SDC "E" and "F" will be determined according to the following Table:

Value of S_1	Occupancy Category		
	I or II	III	IV
$S_1 \geq 0.75$	E	E	F

Step 7: Determine SDC "A" through "D" from both Tables 11.6-1 & 11.6-2 or Table 11.6-1 alone if $S_1 < 0.75$ and the four requirements are met (§11.6 ASCE 7-05)

11.7 DESIGN REQUIREMENTS FOR SEISMIC DESIGN CATEGORY A

11.7.1 Applicability of Seismic Requirements for Seismic Design Category A Structures.

Structures assigned to Seismic Design Category A need only comply with the requirements of Section 11.7. The effects on the structure and its components due to the forces prescribed in this section shall be taken as E and combined with the effects of other loads in accordance with the load combinations of Section 2.3 or 2.4. For structures with damping systems, see Section 18.2.1.

§11.7.2 Lateral Forces. Each structure shall be analyzed for the effects of static lateral forces applied independently in each of two orthogonal directions. In each direction, the static lateral forces at all levels shall be applied simultaneously. For purposes of analysis, the force at each level shall be determined using Eq. 11.7-1 as follows:

$$F_x = 0.01w_x \qquad (11.7\text{-}1)$$

where
F_x = the design lateral force applied at story x, and
w_x = the portion of the total dead load of the structure, D, located or assigned to Level x

11.7.3 Load Path Connections. All parts of the structure between separation joints shall be interconnected to form a continuous path to the lateral force-resisting system, and the connections shall be capable of transmitting the lateral forces induced by the parts being connected. Any smaller portion of the structure shall be tied to the remainder of the structure with elements having design strength of not less than 5 percent of the portion's weight. This connection force does not apply to the overall design of the lateral force-resisting system. Connection design forces need not exceed the maximum forces that the structural system can deliver to the connection.

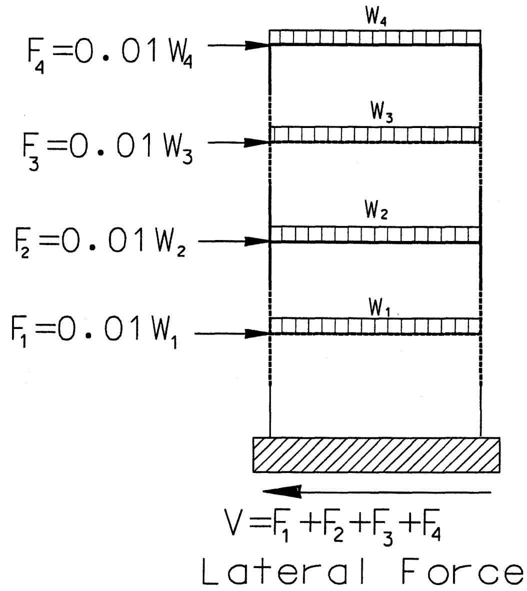


Figure 4-12 Vertical Distribution of Shear for SDC "A"

$$F_E \geq 0.05w_E \qquad (4\text{-}2)$$

w_E = weight of the smaller portion

11.7.4 Connection to Supports. A positive connection for resisting a horizontal force acting parallel to the member shall be provided for each beam, girder, or truss either directly to its supporting elements, or to slabs designed to act as diaphragms. Where the connection is through a diaphragm, then the member's supporting element must also be connected to the diaphragm. The connection shall have a minimum design strength of 5 percent of the dead plus live load reaction.

$$F_E \geq 0.05(dead\ load + live\ load)\ reaction \qquad (4\text{-}3)$$

11.7.5 Anchorage of Concrete or Masonry Walls. Concrete and masonry walls shall be anchored to the roof and all floors and members that provide lateral support for the wall or that are supported by the wall. The anchorage shall provide a direct connection between the walls and the roof or floor construction. The connections shall be capable of resisting the horizontal forces specified in Section 11.7.3, but not less than a minimum strength level horizontal force of 280 lb/linear ft (4.09 kN/m) of wall substituted for E in the load combinations of Section 2.3 or 2.4.

9- Lateral-force-resisting systems (LFRS), § 11.2 & Table 12.2-1, ASCE 7-05

Structural Systems:

i) **Building Frame System:** A structural system with an essentially complete space frame providing support for vertical loads. Seismic force resistance is provided by shear walls or braced frames.

ii) **Dual System**: A structural system with an essentially complete space frame providing support for vertical loads. Seismic force resistance is provided by moment resisting frames and shear walls or braced frames as prescribed in Section 12.2.5.1.

iii) **Shear Wall-Frame Interactive System**: A structural system that uses combinations of ordinary reinforced concrete shearwalls and ordinary reinforced concrete moment frames designed to resist lateral forces in proportion to their rigidities considering interaction between shear walls and frames on all levels.

iv) **Frame:**

1- **Braced Frame**: An essentially vertical truss, or its equivalent, of the concentric or eccentric type that is provided in a building frame system or dual system to resist seismic forces.

2- **Concentrically Braced Frame (CBF):** A braced frame in which the members are subjected primarily to axial forces. CBFs are categorized as ordinary concentrically braced frames (OCBF) or special concentrically braced frames (SCBF).

3- **Eccentrically Braced Frame (EBF)**: A diagonally braced frame in which at least one end of each brace frames into a beam a short distance from a beam-column or from another diagonal brace.

4- **Moment Frame**: A frame in which members and joints resist lateral forces by flexure as well as along the axis of the members. Moment frames are categorized as intermediate moment frames (IMF), ordinary moment frames (OMF), and special moment frames (SMF).

10- Response modification coefficient, *R*, Table 12.2-1, ASCE 7-05

The *R*-value represents a relative rating of the ability of a structural system to resist severe earthquake ground motion without collapse. It is also the reduction in seismic force demand in proportion to the perceived ductility of a give structural system (ductility is the ability of a structure to continue to carry gravity loads as it deforms laterally beyond the stage of elastic response).

TABLE 12.2-1 DESIGN COEFFICIENTS AND FACTORS FOR SEISMIC FORCE–RESISTING SYSTEMS

Seismic Force–Resisting System	Response Modification Coefficient, R^a	System Overstrength Factor, Ω_0^g	Deflection Amplification Factor, C_d^b	Structural System Limitations and Building Height (ft) Limit[c]				
				Seismic Design Category				
				B	C	D[d]	E[d]	F[e]
A. BEARING WALL SYSTEMS								
1. Special reinforced concrete shear walls	5	2 ½	5	NL	NL	160	160	100
2. Ordinary reinforced concrete shear walls	4	2 ½	4	NL	NL	NP	NP	NP
3. Detailed plain concrete shear walls	2	2 ½	2	NL	NP	NP	NP	NP
4. Ordinary plain concrete shear walls	1 ½	2 ½	1 ½	NL	NP	NP	NP	NP
5. Intermediate precast shear walls	4	2 ½	4	NL	NL	40[k]	40[k]	40[k]
6. Ordinary precast shear walls	3	2 ½	3	NL	NP	NP	NP	NP
7. Special reinforced masonry shear walls	5	2 ½	3 ½	NL	NL	160	160	100
8. Intermediate reinforced masonry shear walls	3 ½	2 ½	2 ¼	NL	NL	NP	NP	NP
9. Ordinary reinforced masonry shear walls	2	2 ½	1 ¾	NL	160	NP	NP	NP
10. Detailed plain masonry shear walls	2	2 ½	1 ¾	NL	NP	NP	NP	NP
11. Ordinary plain masonry shear walls	1 ½	2 ½	1 ¼	NL	NP	NP	NP	NP
12. Prestressed masonry shear walls	1 ½	2 ½	1 ¾	NL	NP	NP	NP	NP
13. Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	6 ½	3	4	NL	NL	65	65	65
14. Light-framed walls with shear panels of all other materials	2	2 ½	2	NL	NL	35	NP	NP
15. Light-framed wall systems using flat strap bracing	4	2	3 ½	NL	NL	65	65	65

Seismic Force–Resisting System	Response Modification Coefficient, R^a	System Overstrength Factor, Ω_0^g	Deflection Amplification Factor, C_d^b	Structural System Limitations and Building Height (ft) Limit[c]				
				Seismic Design Category				
				B	C	D[d]	E[d]	F[e]
B. BUILDING FRAME SYSTEMS								
1. Steel eccentrically braced frames, moment resisting connections at columns away from links	8	2	4	NL	NL	160	160	100
2. Steel eccentrically braced frames, non-moment-resisting, connections at columns away from links	7	2	4	NL	NL	160	160	100
3. Special steel concentrically braced frames	6	2	5	NL	NL	160	160	100
4. Ordinary steel concentrically braced frames	3 ¼	2	3 ¼	NL	NL	35[j]	35[j]	NP[j]
5. Special reinforced concrete shear walls	6	2 ½	5	NL	NL	160	160	100
6. Ordinary reinforced concrete shear walls	5	2 ½	4 ¼	NL	NL	NP	NP	NP
7. Detailed plain concrete shear walls	2	2 ½	2	NL	NP	NP	NP	NP
8. Ordinary plain concrete shear walls	1 ½	2 ½	1 ½	NL	NP	NP	NP	NP
9. Intermediate precast shear walls	5	2 ½	4 ½	NL	NL	40[k]	40[k]	40[k]
10. Ordinary precast shear walls	4	2 ½	4	NL	NP	NP	NP	NP
11. Composite steel and concrete eccentrically braced frames	8	2 ½	4	NL	NL	160	160	100
12. Composite steel and concrete concentrically braced frames	5	2	4 ½	NL	NL	160	160	100
13. Ordinary composite steel and concrete braced frames	3	2	3	NL	NL	NP	NP	NP
14. Composite steel plate shear walls	6 ½	2 ½	5 ½	NL	NL	160	160	100
15. Special composite reinforced concrete shear walls with steel elements	6	2 ½	5	NL	NL	160	160	100
16. Ordinary composite reinforced concrete shear walls with steel elements	5	2 ½	4 ½	NL	NL	NP	NP	NP
17. Special reinforced masonry shear walls	5 ½	2 ½	4	NL	NL	160	160	100
18. Intermediate reinforced masonry shear walls	4	2 ½	4	NL	NL	NP	NP	NP
19. Ordinary reinforced masonry shear walls	2	2 ½	2	NL	160	NP	NP	NP
20. Detailed plain masonry shear walls	2	2 ½	2	NL	NP	NP	NP	NP
21. Ordinary plain masonry shear walls	1 ½	2 ½	1 ¼	NL	NP	NP	NP	NP
22. Prestressed masonry shear walls	1 ½	2 ½	1 ¾	NL	NP	NP	NP	NP
23. Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	7	2 ½	4 ½	NL	NL	65	65	65
24. Light-framed walls with shear panels of all other materials	2 ½	2 ½	2 ½	NL	NL	35	NP	NP
25. Buckling-restrained braced frames, non-moment-resisting beam-column connections	7	2	5 ½	NL	NL	160	160	100
26. Buckling-restrained braced frames, moment-resisting beam-column connections	8	2 ½	5	NL	NL	160	160	100
27. Special steel plate shear wall	7	2	6	NL	NL	160	160	100

Seismic Force–Resisting System	Response Modification Coefficient, R^a	System Overstrength Factor, Ω_0^g	Deflection Amplification Factor, C_d^b	Structural System Limitations and Building Height (ft) Limit[c]				
				Seismic Design Category				
				B	C	D[d]	E[d]	F[e]
C. MOMENT- RESISTING FRAME SYSTEMS								
1. Special steel moment frames	8	3	5 ½	NL	NL	NL	NL	NL
2. Special steel truss moment frames	7	3	5 ½	NL	NL	160	100	NP
3. Intermediate steel moment frames	4.5	3	4	NL	NL	35[h]	NP[h]	NP[i]
4. Ordinary steel moment frames	3.5	3	3	NL	NL	NP*h*	NP*h*	NP*i*
5. Special reinforced concrete moment frames	8	3	5 ½	NL	NL	NL	NL	NL
6. Intermediate reinforced concrete moment frames	5	3	4 ½	NL	NL	NP	NP	NP
7. Ordinary reinforced concrete moment frames	3	3	2 ½	NL	NP	NP	NP	NP
8. Special composite steel and concrete moment frames	8	3	5 ½	NL	NL	NL	NL	NL
9. Intermediate composite moment frames	5	3	4 ½	NL	NL	NP	NP	NP
10. Composite partially restrained moment frames	6	3	5 ½	160	160	100	NP	NP
11. Ordinary composite moment frames	3	3	2 ½	NL	NP	NP	NP	NP
D. DUAL SYSTEMS WITH SPECIAL MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES								
1. Steel eccentrically braced frames	8	2 ½	4	NL	NL	NL	NL	NL
2. Special steel concentrically braced frames	7	2 ½	5 ½	NL	NL	NL	NL	NL
3. Special reinforced concrete shear walls	7	2 ½	5 ½	NL	NL	NL	NL	NL
4. Ordinary reinforced concrete shear walls	6	2 ½	5	NL	NL	NP	NP	NP
5. Composite steel and concrete eccentrically braced frames	8	2 ½	4	NL	NL	NL	NL	NL
6. Composite steel and concrete concentrically braced frames	6	2 ½	5	NL	NL	NL	NL	NL
7. Composite steel plate shear walls	7 ½	2 ½	6	NL	NL	NL	NL	NL
8. Special composite reinforced concrete shear walls with steel elements	7	2 ½	6	NL	NL	NL	NL	NL
9. Ordinary composite reinforced concrete shear walls with steel elements	6	2 ½	5	NL	NL	NP	NP	NP
10. Special reinforced masonry shear walls	5 ½	3	5	NL	NL	NL	NL	NL
11. Intermediate reinforced masonry shear walls	4	3	3 ½	NL	NL	NP	NP	NP
12. Buckling-restrained braced frame	8	2 ½	5	NL	NL	NL	NL	NL
13. Special steel plate shear walls	8	2 ½	6 ½	NL	NL	NL	NL	NL

Seismic Force–Resisting System	Response Modification Coefficient, R^a	System Overstrength Factor, Ω_0^g	Deflection Amplification Factor, C_d^b	Structural System Limitations and Building Height (ft) Limit[c] — Seismic Design Category B	C	D[d]	E[d]	F[e]
E. DUAL SYSTEMS WITH INTERMEDIATE MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES								
1. Special steel concentrically braced frames	6	2 ½	5	NL	NL	35	NP	NP[h,k]
2. Special reinforced concrete shear walls	6 ½	2 ½	5	NL	NL	160	100	100
3. Ordinary reinforced masonry shear walls	3	3	2 ½	NL	160	NP	NP	NP
4. Intermediate reinforced masonry shear walls	3 ½	3	3	NL	NL	NP	NP	NP
5. Composite steel and concrete concentrically braced frames	5 ½	2 ½	4 ½	NL	NL	160	100	NP
6. Ordinary composite braced frames	3 ½	2 ½	3	NL	NL	NP	NP	NP
7. Ordinary composite reinforced concrete shear walls with steel elements	5	3	4 ½	NL	NL	NP	NP	NP
8. Ordinary reinforced concrete shear walls	5 ½	2 ½	4 ½	NL	NL	NP	NP	NP
F. SHEAR WALL-FRAME INTERACTIVE SYSTEM WITH ORDINARY REINFORCED CONCRETE MOMENT FRAMES AND ORDINARY REINFORCED CONCRETE SHEAR WALLS	4 ½	2 ½	4	NL	NP	NP	NP	NP
G. CANTILEVERED COLUMN SYSTEMS DETAILED TO CONFORM TO THE REQUIREMENTS FOR:								
1. Special steel moment frames	2 ½	1 ¼	2 ½	35	35	35	35	35
2. Intermediate steel moment frames	1 ½	1 ¼	1 ½	35	35	35[h]	NP[h,i]	NP[h,i]
3. Ordinary steel moment frames	1 ¼	1 ¼	1 ¼	35	35	NP	NP[h,i]	NP[h,i]
4. Special reinforced concrete moment frames	2 ½	1 ¼	2 ½	35	35	35	35	35
5. Intermediate concrete moment frames	1 ½	1 ¼	1 ½	35	35	NP	NP	NP
6. Ordinary concrete moment frames	1	1 ¼	1	35	NP	NP	NP	NP
7. Timber frames	1 ½	1 ½	1 ½	35	35	35	NP	NP
H. STEEL SYSTEMS NOT SPECIFICALLY DETAILED FOR SEISMIC RESISTANCE, EXCLUDING CANTILEVER COLUMN SYSTEMS	3	3	3	NL	NL	NP	NP	NP

[a] Response modification coefficient, R, for use throughout the standard. Note R reduces forces to a strength level, not an allowable stress level.

[b] Reflection amplification factor, C_d, for use in Sections 12.8.6, 12.8.7, and 12.9.2
[c] NL = Not Limited and NP = Not Permitted. For metric units use 30.5 m for 100 ft and use 48.8 m for 160 ft. Heights are measured from the base of the structure as defined in Section 11.2.
[d] See Section 12.2.5.4 for a description of building systems limited to buildings with a height of 240 ft (73.2 m) or less.
[e] See Section 12.2.5.4 for building systems limited to buildings with a height of 160 ft (48.8 m) or less.
[f] Ordinary moment frame is permitted to be used in lieu of intermediate moment frame for Seismic Design Categories B or C.
[g] The tabulated value of the overstrength factor, Ω_o, is permitted to be reduced by subtracting one-half for structures with flexible diaphragms, but shall not be taken as less than 2.0 for any structure.
[h] See Sections 12.2.5.6 and 12.2.5.7 for limitations for steel OMFs and IMFs in structures assigned to Seismic Design Category D or E.
[i] See Sections 12.2.5.8 and 12.2.5.9 for limitations for steel OMFs and IMFs in structures assigned to Seismic Design Category F.
[j] Steel ordinary concentrically braced frames are permitted in single-story buildings up to a height of 60 ft (18.3 m) where the dead load of the roof does not exceed 20 psf (0.96 kN/m2) and in penthouse structures
[k] Increase in height to 45 ft (13.7 m) is permitted for single story storage warehouse facilities.

11- Seismic dead load, *W*, § 12.7-2, ASCE 7-05

§12.7.2 Effective Seismic Weight. The effective seismic weight, W, of a structure shall include the total dead load and other loads listed below:

1. In areas used for storage, a minimum of 25 percent of the floor live load (*floor live load in public garages and open parking structures need not be included*).
2. Where provision for partitions is required by Section 4.2.2 in the floor load design, the actual partition weight or a minimum weight of 10 psf (0.48 kN/m^2) of floor area, whichever is greater.
3. Total operating weight of permanent equipment (e.g. boilers, air conditioners)
4. Where the flat roof snow load, P_f, exceeds 30 psf (1.44 kN/m2), 20 percent of the uniform design snow load, regardless of actual roof slope.

Table 4-20 Effective Seismic Weight Per §12.7.2 (ASCE 7-05)

	Description	Load
1	Areas of storage (*other than public garages and open parking garages)*	➢ 25 percent floor live load
2	Buildings with partitions	➢ 10 psf or actual weight, whichever is greater
3	Permanent Equipment	➢ 100 percent of operating weight
4	Building with roofs designed for snow	➢ Where flat roof snow loads (P_f) are greater than 30 psf, 20 % of the design snow load needs to be included, regardless of actual roof slope

12- Seismic Response Coefficient, C_s, §12.8 ASCE7-05

§12.8.1.1 Calculation of Seismic Response Coefficient. The seismic response coefficient, C_s, shall be determined in accordance with Eq. 12.8-2.

$$C_S = \frac{S_{DS}}{\left(\frac{R}{I}\right)} \qquad (12.8\text{-}2)$$

where

S_{DS} = the design spectral response acceleration parameter in the short period range as determined from Section 11.4.4

R = the response modification factor in Table 12.2-1

I = the occupancy importance factor determined in accordance with Section 11.5.1

The value of C_s computed in accordance with Eq. 12.8-2 need not exceed the following:

$$C_S = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} \quad for \quad T \le T_L \qquad (12.8\text{-}3)$$

$$C_S = \frac{S_{D1}\,T_L}{T^2\left(\frac{R}{I}\right)} \quad for \quad T > T_L \qquad (12.8\text{-}4)$$

C_s shall not be less than

$$C_s = 0.01 \qquad (12.8\text{-}5)$$

In addition, for structures located where S_1 is equal to or greater than 0.6g, C_s shall not be less than

$$C_S = \frac{0.5\,S_1}{\left(\frac{R}{I}\right)} \qquad (12.8\text{-}6)$$

where I and R are as defined in Section 12.8.1.1 and

S_{D1} = the design spectral response acceleration parameter at a period of 1.0 s, as determined from Section 11.4.4

T = the fundamental period of the structure (s) determined in Section 12.8.2

T_L = long-period transition period (s) determined in Section 11.4.5

S_1 = the mapped maximum considered earthquake spectral response acceleration parameter determined in accordance with Section 11.4.1

12.8.1.3 Maximum S_S Value in Determination of C_S. For regular structures five stories or less in height and having a period, T, of 0.5 s or less, C_s is permitted to be calculated using a value of 1.5 for S_S.

13- Seismic Base Shear, V §12.8 ASCE7-05

12.8.1 Seismic Base Shear. The seismic bases hear, V, in a given direction shall be determined in accordance with the following equation:

$$V = C_s W \qquad (12.8\text{-}1)$$

where

C_s = the seismic response coefficient determined in accordance with Section 12.8.1.1

W = the effective seismic weight per Section 12.7.2.

Steps of the Equivalent Lateral Force (ELF) Procedure

Step 1: Calculate the structural period "T" using:

i) Approximate Method: $T_a = C_t h_n^x$ ii) $T_a = 0.1 N$ iii) $T_a = \dfrac{0.0019}{\sqrt{C_w}} h_n$

ii) Rayleigh Method (**Note:** Always check limit if applicable $T \le C_U T_a$)

$$T = 2\pi \sqrt{\left(\sum_{i=1}^{n} w_i \delta_i^2\right) \div \left(g \sum_{i=1}^{n} f_i \delta_i\right)}$$

Step 2: Calculate the period "T_S" $T_S = \dfrac{S_{D1}}{D_{DS}}$ (T_S is not part of any base shear equations, its used for comparison only)

Step 3: Calculate the period "T_0" $T_0 = 0.2 \dfrac{S_{D1}}{D_{DS}}$ (T_0 is not part of any base shear equations, its used for comparison only)

Step 4: If $T < T_0$, the acceleration is given as $S_a = S_{DS}\left(0.4 + 0.6 \dfrac{T}{T_0}\right)$ and the base shear will be as follows: $V = \dfrac{S_a W}{R/I}$

Step 5: If $T_0 \le T < T_S$, Eq. (12.8-2) governs and no need to check the minimum values of (Eq. 12.8-5) & (Eq. 12.8-6).

Step 6: If $T_S < T \le T_L$, Eq. (12.8-3) governs and the minimum values of (Eq. 12.8-5) & (Eq. 12.8-6) should be checked.

Step 7: If $T > T_L$, Eq.(12.8-4) governs

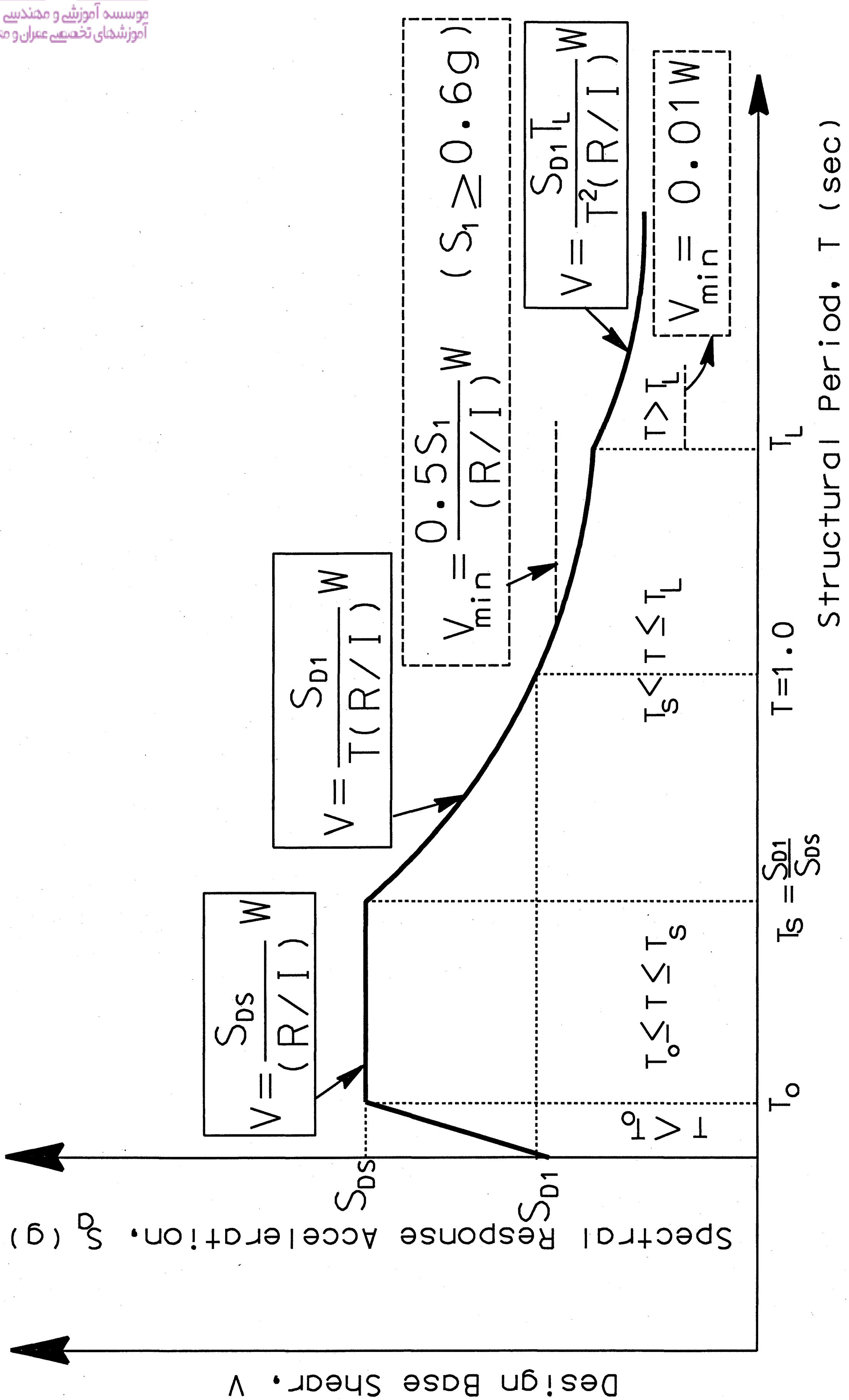


Figure 4-13 Equivalent Lateral Force Procedure

14- Vertical Distribution of Base Shear, F_x §12.8.3 ASCE7-05

§12.8.3 Vertical Distribution of Seismic Forces. The lateral seismic force (F_x) (kip or kN) induced at any level shall be determined from the following equations:

$$F_x = C_{vx} V \qquad (12.8\text{-}11)$$

and

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad (12.8-12)$$

Where:
C_{vx} = vertical distribution factor,
V = total design lateral force or shear at the base of the structure (kip or kN)
w_i and w_x = the portion of the total effective seismic weight of the structure (W) located or assigned to Level i or x
h_i and h_x = the height (ft or m) from the base to Level i or x

k = an exponent related to the structure period as follows:
for structures having a period of 0.5 s or less, $k = 1$ for structures having a period of 2.5s or more, $k = 2$ for structures having a period between 0.5 and 2.5 s, k shall be 2 or shall be determined by linear interpolation between 1 and 2

Table 4-21 Values of the exponent "k"

T	$\le 0.5\,\text{sec}$	$0.5\,\text{sec} < T < 2.5\,\text{sec}$	$\ge 2.5\,\text{sec}$
k	1.0	2.0 OR Linear Interpolation between 1 & 2	2.0

12.8.4 Horizontal Distribution of Forces. The seismic design story shear in any story (V_x) (kip or kN) shall be determined from the following equation:

$$V_x = \sum_{i=x}^{n} F_i \qquad (12.8-13)$$

Where:
F_i = the portion of the seismic base shear (V) (kip or kN) induced at Level i.

The seismic design story shear (V_x) (kip or kN) shall be distributed to the various vertical elements of the seismic force– resisting system in the story under consideration based on the relative lateral stiffness of the vertical resisting elements and the diaphragm.

Table 4-22 DISTRIBUTION OF BASE SHEAR (V)

Simplified Static Procedure

- To be used if the base shear was calculated using Eq. (12.14-11)

$$V = \frac{FS_{DS}}{R} W \qquad (12.14-11)$$

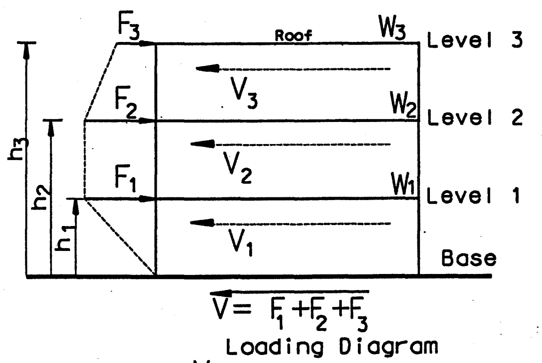
F = 1.0 for one-story buildings
 = 1.1 for two-story buildings
 = 1.2 for three-story buildings

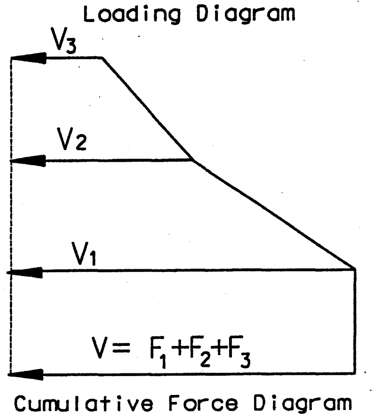
R=the response modification factor from Table 12.14-1

- The distribution is based on the following equation:

$$F_x = \frac{w_x}{W} V \qquad (12.14-12)$$

- Distribution is nonlinear and is not a function of the story height (h_x)





ELF Procedure

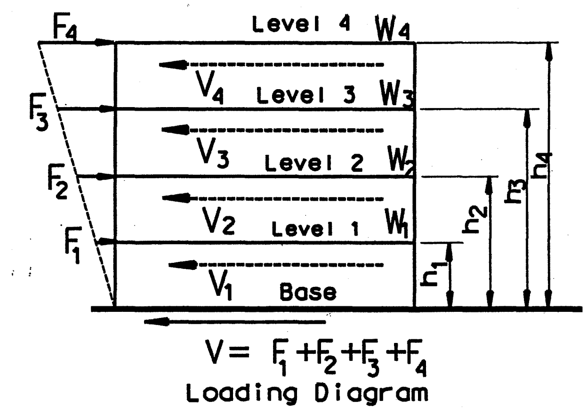
- To be used if the base shear was calculated using Eq. (12.8-1)

$$V = C_s W \qquad (12.8\text{-}1)$$

- The distribution is based on the following equation:

$$F_x = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} V \qquad (12.8-11)$$

- Distribution is linear (with certain assumptions) and is a function of the story height (h_x)



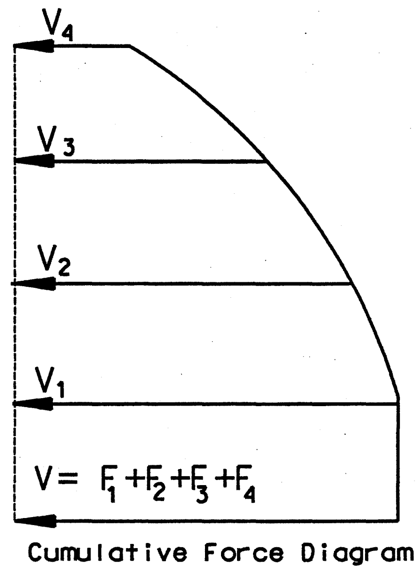


Table 4-23 COMPARISON BETWEEN w_x & w_{px}

<table>
<tr><th>w_x</th><th>w_{px}</th></tr>
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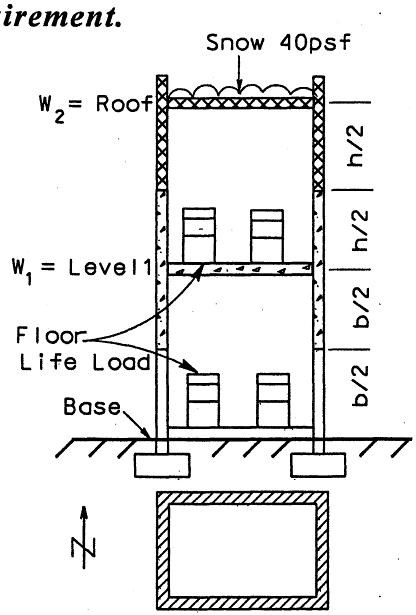
➢ Seismic dead load to be used in calculating the distribution of the base shear F_x

$$F_x = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} V \qquad (12.8-11)$$

➢ It consist of the dead load of the level (floor or roof) plus the exterior tributary walls dead load plus any portion of the FOUR items mentioned above

➢ It includes all walls (parallel and perpendicular) because all walls contribute to the distribution of the base shear.

➢ All foundation weight and half of the first story weight are ignored. ***This is a common practice NOT Code requirement.***



W_1 = Dead load of the floor + Applicable portion(s) of the FOUR items §12.7.2 + (b/2 +h/2) (building perimeter)(unit weight of the wall)

W_2 = Dead load of the roof + Applicable portion(s) of the FOUR items §12.7.2 + (h/2) (building perimeter)(unit weight of the wall)

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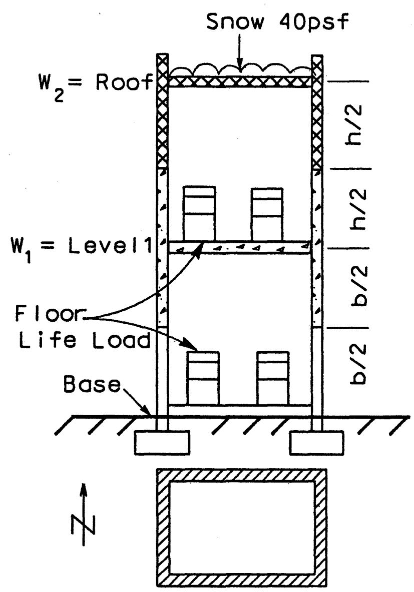
➢ Seismic dead load to be used in calculating the diaphragm force F_{px} (Eq. 12.10-1)

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \qquad (12.10-1)$$

➢ It consist of the dead load of the diaphragm plus the exterior tributary perpendicular (normal) walls dead load plus any portion of the FOUR items mentioned above

➢ Does not include walls parallel to the direction of analysis because it do not contribute to the load on the diaphragm.

➢ All foundation weight and half of the first story weight are ignored. ***This is a common practice NOT Code requirement.***



W_{p1} = Dead load of the floor + Applicable portion(s) of the FOUR items §12.7.2 + (b/2 +h/2) (walls ⊥ direction of analysis)(unit weight of the wall)

W_{p2} = Dead load of the roof + Applicable portion(s) of the FOUR items §12.7.2 + (b/2 +h/2) (walls ⊥ direction of analysis)(unit weight of the wall)

</td></tr>
</table>

15- Horizontal and Vertical Components of E, E_h & E_v §12.4 ASCE7-05

§12.4.2 Seismic Load Effect. The seismic load effect, E, shall be determined in accordance with the following:

1. For use in load combination 5 in Section 2.3.2 or load combination 5 and 6 in Section 2.4.1, E shall be determined in accordance with Eq. 12.4-1 as follows:

$$E = E_h + E_v \qquad (12.4\text{-}1)$$

2. For use in load combination 7 in Section 2.3.2 or load combination 8 in Section 2.4.1, E shall be determined in accordance with Eq. 12.4-2 as follows:

$$E = E_h - E_v \qquad (12.4\text{-}2)$$

where

E = seismic load effect

E_h = effect of horizontal seismic forces as defined in Section 12.4.2.1

E_v = effect of vertical seismic forces as defined in Section 12.4.2.2

12.4.2.1 Horizontal Seismic Load Effect. The horizontal seismic load effect, E_h, shall be determined in accordance with Eq. 12.4-3 as follows:

$$E_h = \rho\, Q_E \qquad (12.4\text{-}3)$$

where

Q_E = effects of horizontal seismic forces from V or F_P. Where required in Sections 12.5.3 and 12.5.4, such effects shall result from application of horizontal forces simultaneously in two directions at right angles to each other.

ρ = redundancy factor, as defined in Section 12.3.4

12.4.2.2 Vertical Seismic Load Effect. The vertical seismic load effect, E_v, shall be determined in accordance with Eq. 12.4-4 as follows:

$$E_v = 0.2 S_{DS} D \qquad (12.4\text{-}4)$$

where

S_{DS} = design spectral response acceleration parameter at short periods obtained from Section 11.4.4

D = effect of dead load

EXCEPTIONS: The vertical seismic load effect, E_v, is permitted to be taken as zero for either of the following conditions:

1. In Eqs. 12.4-1, 12.4-2, 12.4-5, and 12.4-6 where SDS is equal to or less than 0.125.
2. In Eq. 12.4-2 where determining demands on the soil-structure interface of foundations.

16- Redundancy Factor: ρ, §12.3.4, ASCE 7-05

Redundancy factor "ρ" is multiple paths of resistance (load paths) to earthquake forces. More redundancy means better reliability because these is increased opportunity for inelastic deformations.

Redundancy factor "ρ" is used to encourage the designer to provide a reasonable number and distribution of lateral-force-resisting elements (LFRE). This could be done by providing sufficient shear walls of reasonable length, bracing systems, moment frames or a combination of such systems. The Code is penalizing the structures that have few LFRE by assigning a large value (grater than 1) of the redundancy factor.

It should be noted that the redundancy factor "ρ" is to be calculated separately for each of the principle directions of lateral loading (i.e. N-S, E-W).

§12.3.4 Redundancy. A redundancy factor, ρ, shall be assigned to the seismic force–resisting system in each of two orthogonal directions for all structures in accordance with this section.

§12.3.4.1 Conditions Where Value of ρ is 1.0. The value of ρ is permitted to equal 1.0 for the following:

1. Structures assigned to Seismic Design Category B or C.
2. Drift calculation and P-delta effects.
3. Design of nonstructural components.
4. Design of nonbuilding structures that are not similar to buildings.
5. Design of collector elements, splices, and their connections for which the load combinations with overstrength factor of Section 12.4.3.2 are used.
6. Design of members or connections where the load combinations with overstrength of Section 12.4.3.2 are required for design.
7. Diaphragm loads determined using Eq. 12.10-1.
8. Structures with damping systems designed in accordance with Section 18.

§12.3.4.2 Redundancy Factor, ρ, for Seismic Design Categories D through F. For structures assigned to Seismic Design Category D, E, or F, ρ shall equal 1.3 unless one of the following two conditions is met, whereby ρ is permitted to be taken as 1.0:

a. Each story resisting more than 35 percent of the base shear in the direction of interest shall comply with Table 12.3-3.
b. Structures that are regular in plan at all levels provided that the seismic force–resisting systems consist of at least two bays of seismic force–resisting perimeter framing on each side of the structure in each orthogonal direction at each story resisting more than 35 percent of the base shear. The number of bays for a shear wall shall be calculated as the

length of shear wall divided by the story height or two times the length of shear wall divided by the story height for light-framed construction.

17- Overstrength Factor : Ω_0 , Table 12.2-1, ASCE 7-05

The seismic force amplification factor Ω_0 is a factor greater than "1 " and was incorporated in the Code Equation 12.4-7(ASCE 7-05) to account for structural overstrength. Overstrength could be defined as a characteristics of structures where the actual strength is greater than the design strength.

$$E_{mh} = \Omega_0 \; Q_E \qquad (12.4\text{-}7)$$

$$Maximum\ Force = \Omega_0 \times Design\ \ Force$$

$$V_M = \Omega_0 \times V_s$$

The magnitude of overstrength factor, Ω_0 depends on the type of the structural system and the material used for constructing such system. This is obvious in the values listed in Table 12.2-1 for building structure (from 1¼ to 3.0). As explained before that the design philosophy of the current Code is based on the inelastic behavior of structures. To account for this behavior, the Code is using this factor.

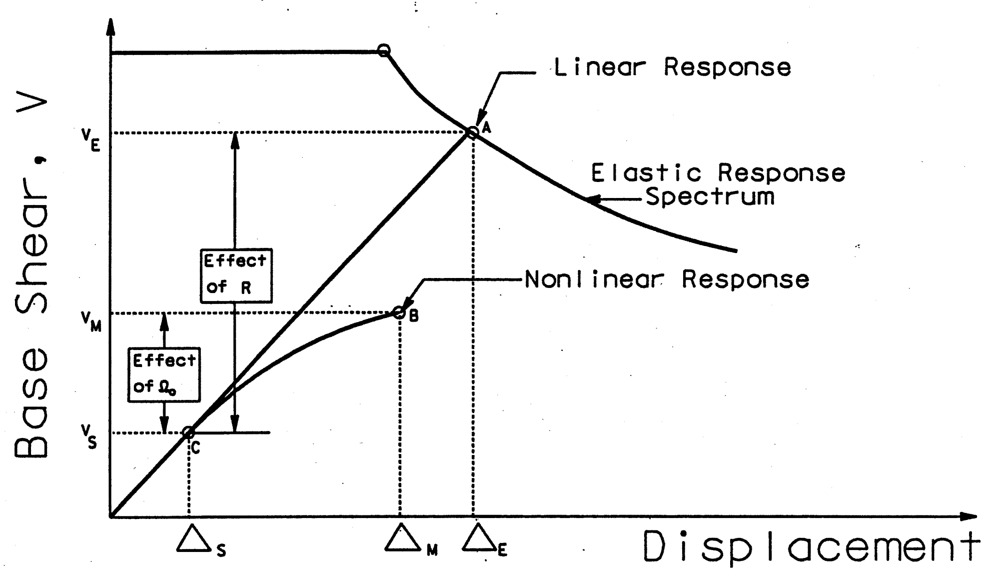


Figure 4-14 Building Earthquake Response

NOTE: From the above Figure and the discussion for applying the overstrength factor Ω_0 , the following relationships between V_E, V_M & V_S as follows:

$$V_E = (R/I) \times C_s \times W \qquad \Leftarrow Elastic\ \ Curve$$

$$V_M = \Omega_0 \times C_S \times W \qquad \Leftarrow Inelastic\ Curve$$

$$V = C_S \times W \qquad \Leftarrow ELF\,(ASCE\,7-05\,/\,IBC\,/\,CBC)$$

18- Deflection Control & Drift: $\delta_x = C_d\delta_{xe}/I$, §12.12.1 &Table 12.2-1

Drift is defined as the horizontal (lateral) deflection at the top of the story relative to the bottom of the story. Total drift of a structure is the lateral (horizontal) displacement of level "n" relative to the base of the structure. Three terms associated with the drift should be addressed:

i) story drift: the lateral displacement (deflection) at the top of the story relative to the bottom of the story.

ii) story drift ratio: (story drift/ story height)-this will be used in the "$P\Delta$ effects"

iii) total drift: the lateral (horizontal) displacement of level "n" relative to the base of the structure

The amount of drift either story or total should be within the specified limits stated in the Code due to the following reasons:

a) to prevent structure and nonstructural damage resulting from excessive drift
b) to prevent adjacent buildings from pounding against each other (building separation)
c) to prevent the encroachment on an adjacent private or public property (set back)
d) to prevent (or minimize) the psychological effects on the occupants.

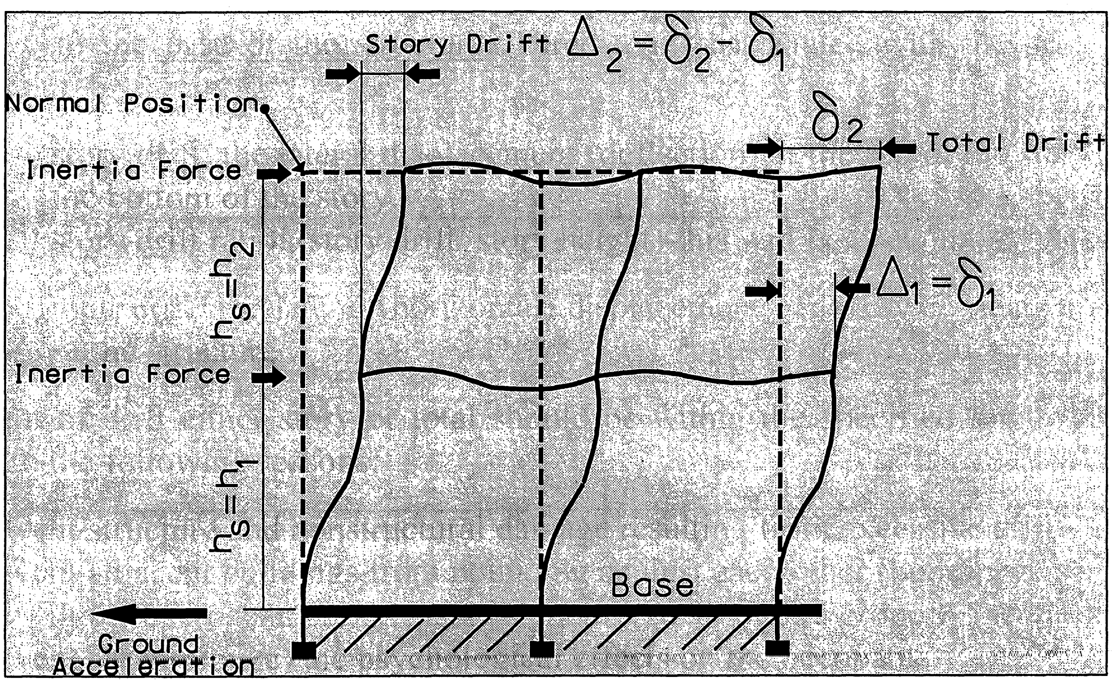


Figure 4-15 Total Drift, Story Drift and Story Drift Ratio

Story drift at any level is:

$$\Delta_{Sx} = \delta_x - \delta_{x-1} \quad \text{(Eq. 4-4)}$$

where δ_x is the design total lateral displacement with respect to the base

12.8.6 Story Drift Determination. The design story drift (Δ) shall be computed as the difference of the deflections at the centers of mass at the top and bottom of the story under consideration as shown in Fig. 12.8-2. Where allowable stress design is used, shall be computed using the strength level seismic forces specified in Section 12.8 without reduction for allowable stress design.

The deflections of Level x at the center of the mass (δ_x) (in. or mm) shall be determined in accordance with the following equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I} \qquad (12.8-15)$$

Where:

C_d = the deflection amplification factor in Table 12.2-1

δ_{xe} = the deflections determined by an elastic analysis

I = the importance factor determined in accordance with Section 11.5.1

12.8.6.1 Minimum Base Shear for Computing Drift. The elastic analysis of the seismic force–resisting system shall be made using the prescribed seismic design forces of Sec. 12.8.

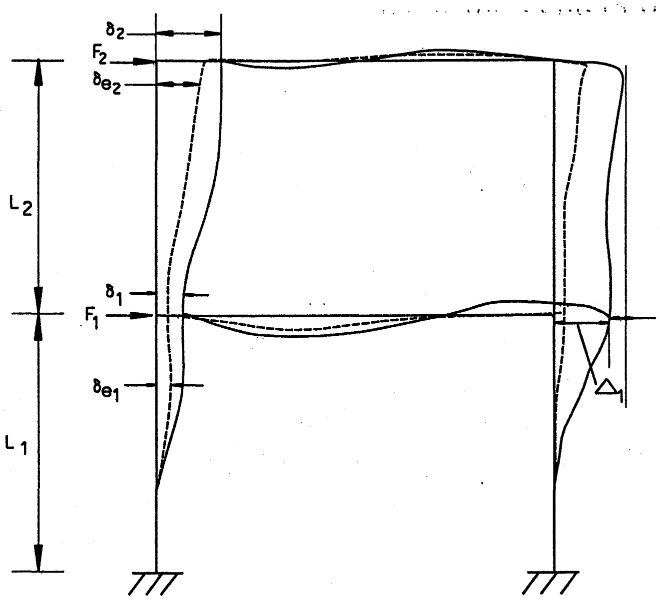


FIGURE 12.8-2 STORY DRIFT DETERMINATION

Story Level 2

F_2 = strength –level design earthquake force

δ_{e2} = elastic displacement computed under strength-level earthquake force.

$\delta_2 = C_d \delta_{e2} / I_E$ = amplified displacement

$\Delta_2 = (\delta_{e2} - \delta_{e1}) C_d / I_E \le \Delta_a$ (Table 12.12-1)

Story Level 1

F_1 = strength –level design earthquake force

δ_{e1} = elastic displacement computed under strength-level earthquake force.

$\delta_1 = C_d \delta_{e1} / I_E$ = amplified displacement

$\Delta_1 = \delta_1 \le \Delta_a$ (Table 12.12-1)

Δ_1 = *Story Drift*

Δ_1 / L_1 = *Story Drift Ratio*

δ_2 = *Total Displacement*

12.8.6.2 Period for Computing Drift. For determining compliance with the story drift limits of Section 12.12.1, it is permitted to determine the elastic drifts, (δ_{xe}), using seismic design forces based on the computed fundamental period of the structure without the upper limit ($C_u T_a$) specified in Section 12.8.2.

§12.12 DRIFT AND DEFORMATION

12.12.1 Story Drift Limit. The design story drift (Δ)as determined in Sections 12.8.6, 12.9.2, or 16.1, shall not exceed the allowable story drift(Δ_a)as obtained from Table 12.12-1 for any story. For structures with significant torsional deflections, the maximum drift shall include torsional effects. For structures assigned to Seismic Design Category C, D, E, or F having horizontal irregularity Types 1a or 1b of Table 12.3-1, the design story drift, Δ, shall be computed as the largest difference of the deflections along any of the edges of the structure at the top and bottom of the story under consideration.

12.12.1.1 Moment Frames in Structures Assigned to Seismic Design Categories D through F. For seismic force–resisting systems comprised solely of moment frames in structures assigned to Seismic Design Categories D, E, or F, the design story drift (Δ) shall not exceed Δ_a/ρ for any story. ρ shall be determined in accordance with Section 12.3.4.2.

TABLE 12.12-1 ALLOWABLE STORY DRIFT, $\Delta_a^{a,b}$

Structure	Occupancy Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}{}^{c}$	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures [d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

[a] h_{sx} is the story height below Level x.

[b] For seismic force–resisting systems comprised solely of moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.

[c] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived.

[d] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

19- Combinations of Structural Systems Table 12.2-1 & §12.2.3.1 & 12.2.3.2

§12.2.3.1 *R*, C_d, and Ω_o Values for Vertical Combinations. The value of the response modification coeficient, R, used for design at any story shall not exceed the lowest value of R that is used in the same direction at any story above that story. Likewise, deflection amplification factor, C_d, and the system over strength factor, Ω_o, used for the design at any story shall not be less than the largest value of this factor that is used in the same direction at any story above that story.

EXCEPTIONS:

1. Rooftop structures not exceeding two stories in height and 10 percent of the total structure weight.
2. Other supported structural systems with a weight equal to or less than 10 percent of the weight of the structure.
3. Detached one-and two-family dwellings of light-frame construction.

A two-stage equivalent lateral force procedure is permitted to be used for structures having a flexible upper portion above a rigid lower portion, provided that the design of the structure complies with the following:

a. The stiffness of the lower portion must be at least 10 times the stiffness of the upper portion.
b. The period of the entire structure shall not be greater than 1.1 times the period of the upper portion considered as a separate structure fixed at the base.
c. The flexible upper portion shall be designed as a separate structure using the appropriate values of R and ρ.
d. The rigid lower portion shall be designed as a separate structure using the appropriate values of R and ρ. The reactions from the upper portion shall be those determined from the analysis of the upper portion amplified by the ratio of the R/ρ of the upper portion over R/ρ of the lower portion. This ratio shall not be less than 1.0.

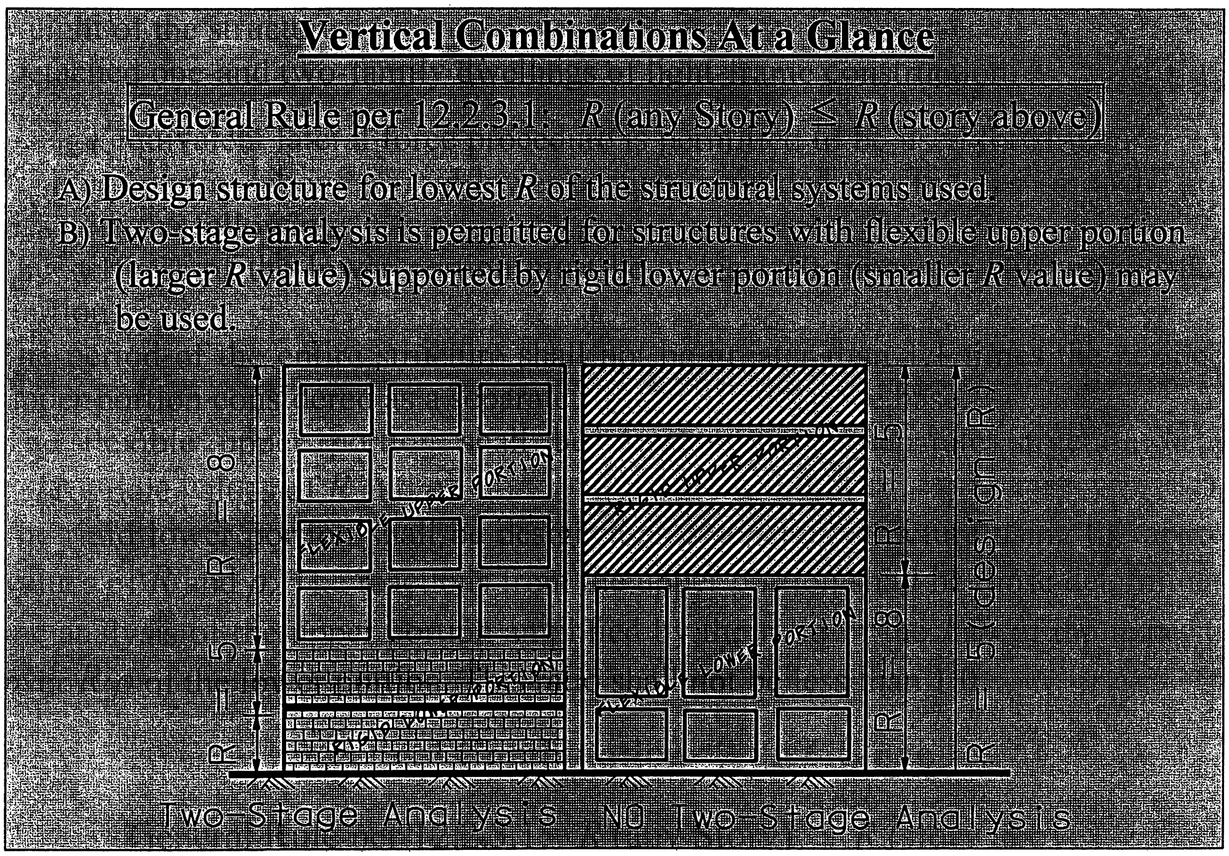


Figure 4-16 Vertical Combinations

§12.2.3.2 R, C_d ,and Ω_o Values for Horizontal Combinations.

Where a combination of different structural systems is utilized to resist lateral forces in the same direction, the value of R used for design in that direction shall not be greater than the least value of R for any of the systems utilized in that direction. Resisting elements are permitted to be designed using the least value of R for the different structural systems found in each independent line of resistance if the following three conditions are met:

(1) Occupancy Category I or II building,
(2) two stories or less in height, and
(3) use of light-frame construction or flexible diaphragms.

The value of R used for design of diaphragms in such structures shall not be greater than the least value for any of the systems utilized in that same direction.
The deflection amplification factor, C_d , and the system over strength factor, Ω_o ,in the direction under consideration at any story shall not be less than the largest value of this factor for the R factor used in the same direction being considered.

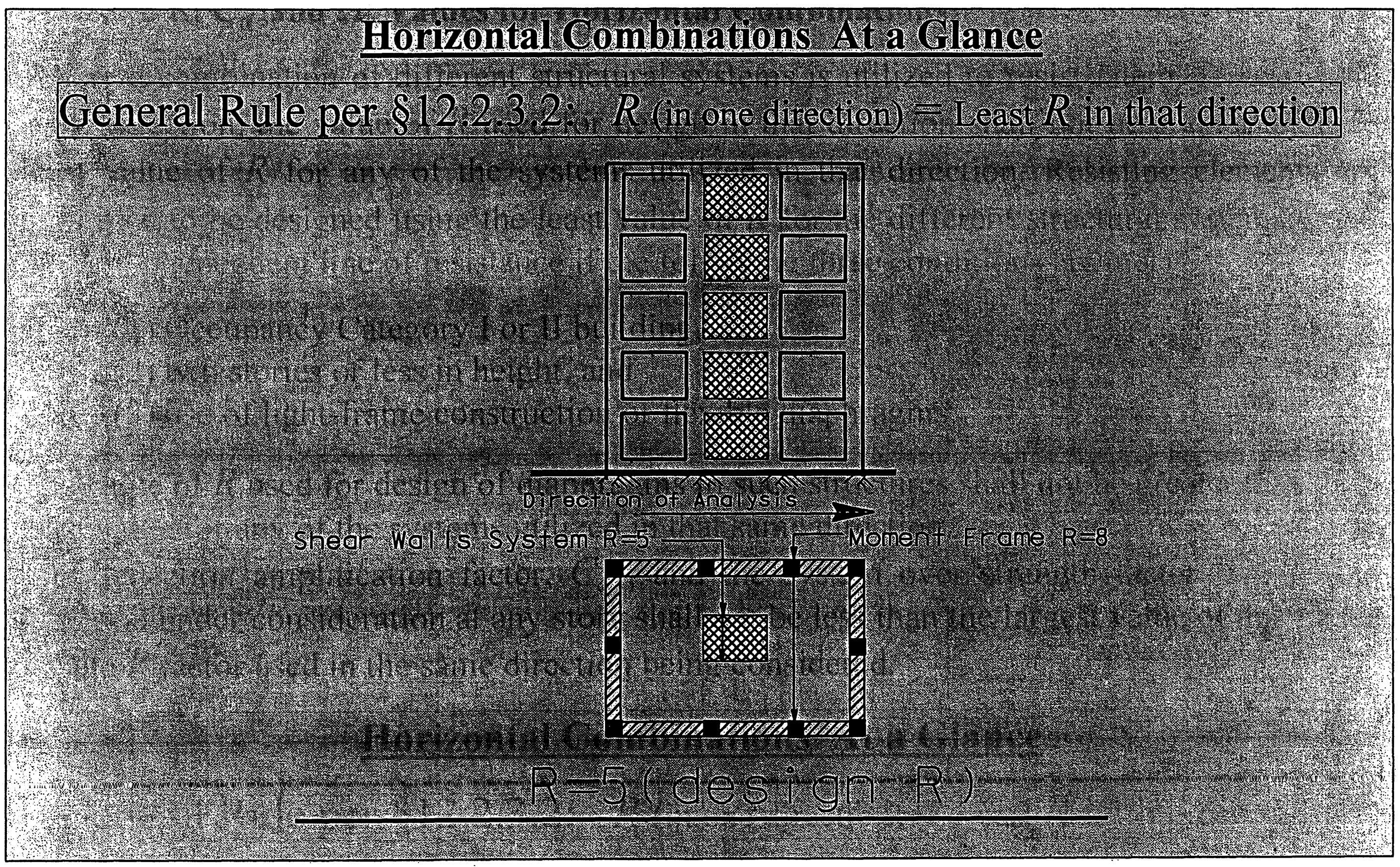


Figure 4-17 Horizontal Combinations

12.2.4 Combination Framing Detailing Requirements. Structural components common to different framing systems used to resist seismic motions in any direction shall be designed using the detailing requirements of Chapter 12 required by the highest response modification coefficient, R, of the connected framing systems.

4.7 BUILDING SEPARATIONS

§12.12.3 Building Separation. All portions of the structure shall be designed and constructed to act as an integral unit in resisting seismic forces unless separated structurally by a distance sufficient to avoid damaging contact under total deflection (δ_x) as determined in Section 12.8.6

$$\delta_{MT} = \delta_{M1} + \delta_{M2} \qquad \text{(Eq. 5-5)}$$

and δ_{M1} and δ_{M2} are the maximum inelastic displacements of the adjacent buildings. When a structure adjoins a property line not common to a public way, that structure shall also be set back from the property line by at least the displacement δ_M of that structure.

NOTE: SEAOC recommendation for the building is given by the following equation and this was the same equation in the 2001CBC & 97 UBC Codes.

$$\delta_{MT} = \sqrt{(\delta_{M1})^2 + (\delta_{M2})^2}$$

EXCEPTION: Smaller separations or property line setbacks may be permitted when justified by rational analyses based on maximum expected ground motions.

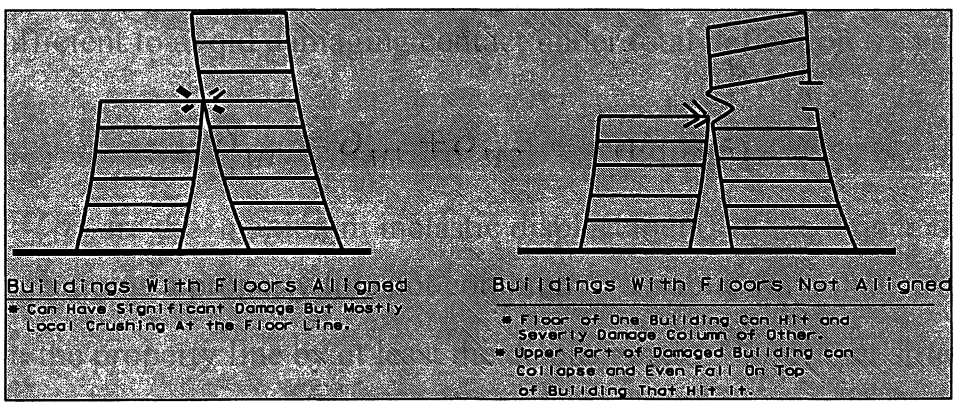


Figure 4-18 Pounding of Adjacent Buildings (Inadequate Building Separation)

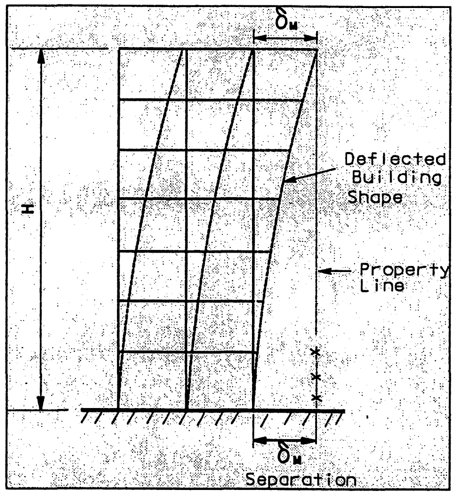


Figure 4-19 Separation Adjacent to Property Line

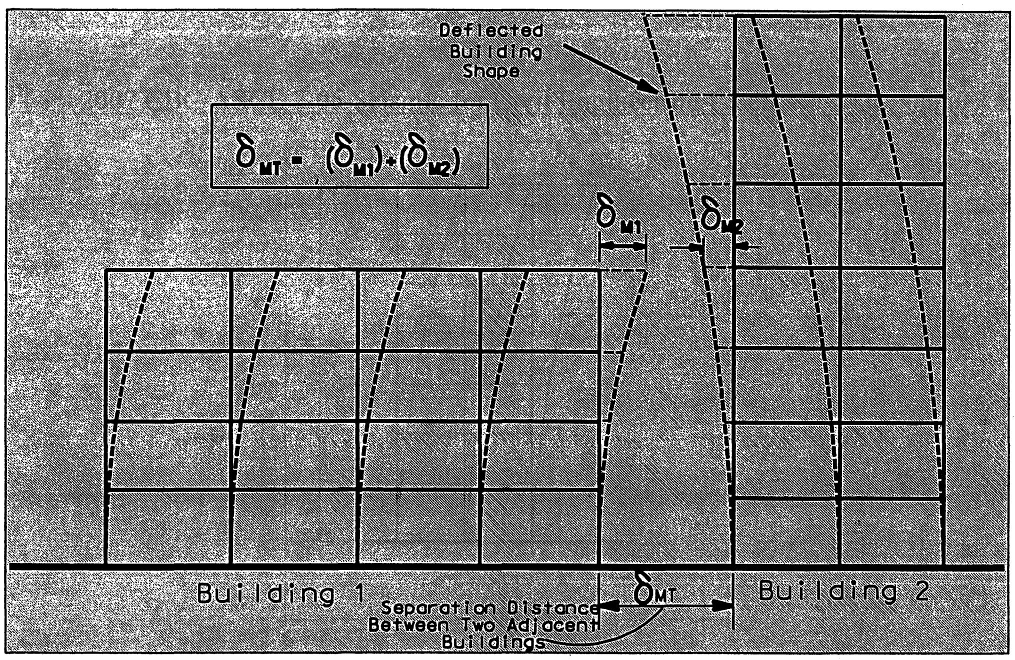


Figure 4-20 Adjacent Buildings Separation

4.8 $P-\Delta$ EFFECTS

The gravity loads are usually are concentric with respect to the compression members as columns. Due to the lateral seismic or wind forces, these concentric loads become eccentric causing an additional moment called secondary moment . The secondary moment effect may considered in the analysis based on the following criteria :

Primary bending moment (M_P): the moment resulted from the lateral force "F"

$$M_P = (F)(h)$$

Secondary bending moment (M_S): the moment resulted from the lateral displacement of the gravity loads (DL +LL) due to the lateral force

$$M_S = (DL + LL)(\Delta)$$

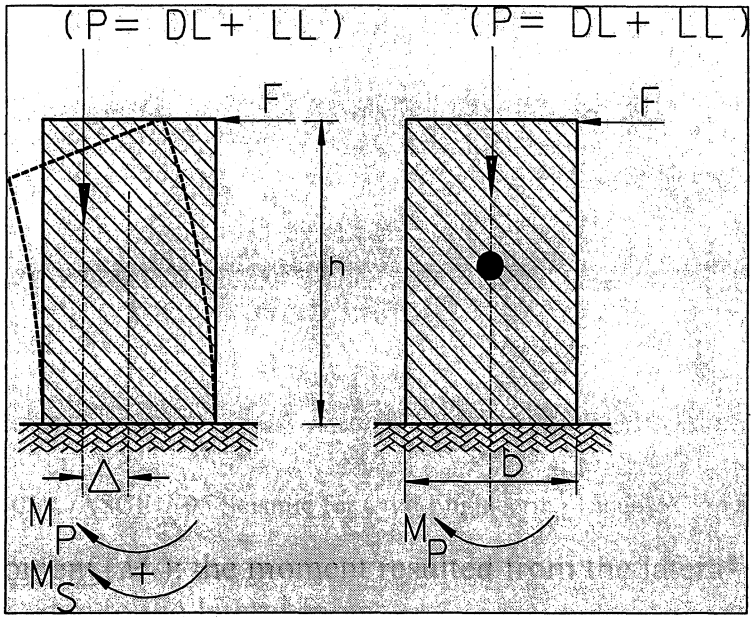


Figure 4-21 $P-\Delta$ Effects

If $\theta = \dfrac{P_x \Delta}{V_x h_{sx} C_d} \le 0.10$	$\Rightarrow P-\Delta$ need not be considered

§12.8.7 P-Delta Effects. P-delta effects on story shears and moments, the resulting member forces and moments, and the story drifts induced by these effects are not required to be considered where the stability coefficient (θ) as determined by the following equation is equal to or less than 0.10:

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d} \qquad (12.8\text{-}16)$$

where :

P_x = the total vertical design load at and above Level x (kip or kN); where computing P_x, no individual load factor need exceed 1.0

Δ = the design story drift as defined in Section 12.8.6 occurring simultaneously with V_x (in. or mm)

V_x = the seismic shear force acting between Levels x and $x - 1$ (kip or kN)

h_{sx} = the story height below Level x (in. or mm)

C_d = the deflection amplification factor in Table 12.2-1

The stability coefficient (θ) shall not exceed θ_{max} determined as follows:

$$\theta_{\max} = \frac{0.5}{\beta C_d} \leq 0.25 \qquad (12.8\text{-}17)$$

where β is the ratio of shear demand to shear capacity for the story between Levels x and $x - 1$. This ratio is permitted to be conservatively taken as 1.0.

Where the stability coefficient (θ) is greater than 0.10 but less than or equal to θ_{max}, the incremental factor related to P-delta effects on displacements and member forces shall be determined by rational analysis. Alternatively, it is permitted to multiply displacements and member forces by $1.0/(1 - \theta)$.

Where θ is greater than θ_{max}, the structure is potentially unstable and shall be redesigned.

Where the P-delta effect is included in an automated analysis, Eq. 12.8-17 shall still be satisfied, however, the value of θ computed from Eq. 12.8-16 using the results of the P-delta analysis is permitted to be divided by (1 + θ) before checking Eq. 12.8-17.

4.9 – SIMPLIFIED STATIC (ALTERNATIVE DESIGN) FOR SIMPLIFIED BEARING WALL OR BUILDING FRAME SYSTEMS

For structures that are classified as Occupancy Category I or II have bearing wall or building frame systems and **NOT** exceeding three stories in height, this alternative procedure is permitted per §12.14 of the ASCE 7-05.

Table 4-24 Simplified Design Procedure

Occupancy Category	Maximum Height	Soil Type (Site Class)	Structural Systems	Seismic Design Category (SDC)
I & II (See Table 1604.5)	Three (See Table 12.14-1)	A to D (**NO** E & F Type)	1-Bearing Wall 2- Building Frames Note: **NO** SDC "F"	Only Table 11.6-1 (based on S_{DS})

§12.14.1.1 Simplified Design Procedure. The procedures of this section are permitted to be used in lieu of other analytical procedures in Chapter12 for the analysis and design of simple buildings with bearing wall or building frame systems, subject to all of the limitations listed in this Section 12.14.1.1. Where these procedures are used, the seismic design category shall be determined from Table 11.6-1 using the value of S_{DS} from Section 12.14.8.1.

The simplified design procedure is permitted to be used if the following limitations are met:

1. The structure shall qualify for Occupancy Category I or II in accordance with Table 1-1.
2. The site class, defined in Chapter 20, shall **NOT** be class E or F.
3. The structure shall not exceed three stories in height above grade.
4. The seismic-force resisting system shall be either a bearing wall system or building frame system, as indicated in Table 12.14-1.
5. The structure shall have at least two lines of lateral resistance in each of two major axis directions.
6. At least one line of resistance shall be provided on each side of the center of mass in each direction.
7. For structures with flexible diaphragms, overhangs beyond the outside line of shear walls or braced frames shall satisfy the following:

$$a \leq d/5 \qquad (12.14\text{-}1)$$

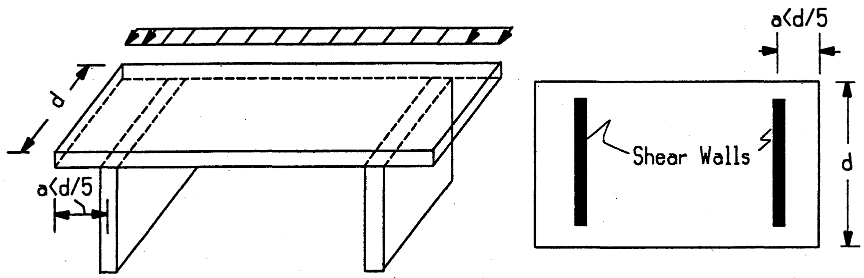


Figure 4-22 Overhang of Flexible Diaphragm

where

a = the distance perpendicular to the forces being considered from the extreme edge of the diaphragm to the line of vertical resistance closest to that edge

d = the depth of the diaphragm parallel to the forces being considered at the line of vertical resistance closest to the edge.

8. For buildings with a diaphragm that is not flexible, the distance between the center of rigidity and the center of mass parallel to each major axis shall not exceed 15 percent of the greatest width of the diaphragm parallel to that axis. In addition, the following two equations shall be satisfied:

$$\sum_{i=1}^{m} k_{1i} d_i^2 + \sum_{j=1}^{n} k_{2j} d_j^2 \ge 2.5\left(0.05 + \frac{e_1}{b_1}\right) b_1^{\,2} \sum_{i=1}^{m} k_{1i} \qquad (12.14-2A)$$

$$\sum_{i=1}^{m} k_{1i} d_i^2 + \sum_{j=1}^{n} k_{2j} d_j^2 \ge 2.5\left(0.05 + \frac{e_2}{b_2}\right) b_2^{\,2} \sum_{i=1}^{m} k_{1i} \qquad (12.14-2B)$$

TABLE 12.14-1 DESIGN COEFFICIENTS AND FACTORS FOR SEISMIC FORCE–RESISTING SYSTEMS FOR SIMPLIFIED DESIGN PROCEDURE

Seismic Force–Resisting System	Response Modification Coefficient, R^a	Limitations[b] Seismic Design Category B	C	D, E
A. BEARING WALL SYSTEMS				
1. Special reinforced concrete shear walls	5	P	P	P
2. Ordinary reinforced concrete shear walls	4	P	P	NP
3. Detailed plain concrete shear walls	2	P	NP	NP
4. Ordinary plain concrete shear walls	1 ½	P	NP	NP
5. Intermediate precast shear walls	4	P	P	40[C]
6. Ordinary precast shear walls	3	P	NP	NP
7. Special reinforced masonry shear walls	5	P	P	P
8. Intermediate reinforced masonry shear walls	3 ½	P	P	NP
9. Ordinary reinforced masonry shear walls	2	P	NP	NP
10. Detailed plain masonry shear walls	2	P	NP	NP
11. Ordinary plain masonry shear walls	1 ½	P	NP	NP
12. Prestressed masonry shear walls	1 ½	P	NP	NP
13. Light-frame walls sheathed with wood structural panels rated for shear resistance or steel sheets	6 ½	P	P	P
14. Light-framed walls with shear panels of all other materials	2	P	P	NP[d]
15. Light-framed wall systems using flat strap bracing	4	P	P	P

B. BUILDING FRAME SYSTEMS				
1. Steel eccentrically braced frames, moment-resisting connections at columns away from links	8	P	P	P
2. Steel eccentrically braced frames, non-moment-resisting connections at columns away from links	7	P	P	P
3. Special steel concentrically braced frames	6	P	P	P
4. Ordinary steel concentrically braced frames	3 ¼	P	P	P
5. Special reinforced concrete shear walls	6	P	P	P
6. Ordinary reinforced concrete shear walls	5	P	P	NP
7. Detailed plain concrete shear walls	2	P	NP	NP
8. Ordinary plain concrete shear walls	1 ½	P	NP	NP
9. Intermediate precast shear walls	5	P	P	40[c]
10. Ordinary precast shear walls	4	P	NP	NP
11. Composite steel and concrete eccentrically braced frames	8	P	P	P
12. Composite steel and concrete concentrically braced frames	5	P	P	P
13. Ordinary composite steel and concrete braced frames	3	P	P	NP
14. Composite steel plate shear walls	6 ½	P	P	P
15. Special composite reinforced concrete shear walls with steel elements	6	P	P	P
16. Ordinary composite reinforced concrete shear walls with steel elements	5	P	P	NP
17. Special reinforced masonry shear walls	5 ½	P	P	P
18. Intermediate reinforced masonry shear walls	4	P	P	NP
19. Ordinary reinforced masonry shear walls	2	P	NP	NP
20. Detailed plain masonry shear walls	2	P	NP	NP
21. Ordinary plain masonry shear walls	1 ½	P	NP	NP
22. Prestressed masonry shear walls	1 ½	P	NP	NP
23. Light-frame walls sheathed with wood structural panels rated for shear resistance or steel sheets	7	P	P	P
24. Light-framed walls with shear panels of all other materials	2 ½	P	P	NP[d]
25. Buckling-restrained braced frames, non-moment-resisting beam-column connections	7	P	P	P
26. Buckling-restrained braced frames, moment-resisting beam-column connections	8	P	P	P
27. Special steel plate shear wall	7	P	P	P

[a] Response modification coefficient, R, for use throughout the standard.
[b]P = permitted; NP = not permitted.

[c] Light-framed walls with shear panels of all other materials not permitted in Seismic Design Category E.
[d] Light-framed walls with shear panels of all other materials permitted up to 35 ft in height in Seismic Design Category D and not permitted in Seismic Design Category E.

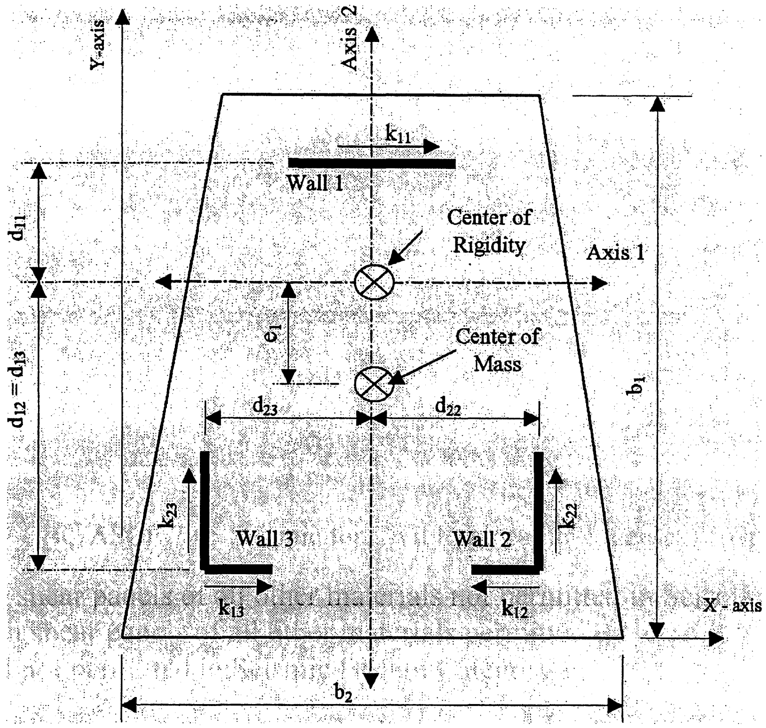


FIGURE 12.14-1 NOTATION USED IN TORSION CHECK FOR NONFLEXIBLE DIAPHRAGMS

where (see Fig. 12.14-1):

k_{1i} is the lateral load stiffness of wall "*i*" or braced frame "*i*" parallel to major axis 1
k_{2j} *is* the lateral load stiffness of wall "*j*" or braced frame "*j*" parallel to major axis 2
d_{1j} *is* the distance from the wall "*i*" or braced frame "*i*" to the center of rigidity, perpendicular to major axis 1
d_{2j} is the distance from the wall "*j*" or braced frame "*j*" to the center of rigidity, perpendicular to major axis 2
e_1 is the distance perpendicular to major axis 1 between the center of rigidity and the center of mass
b_1 is the width of the diaphragm perpendicular to major axis 1
e_2 is the distance perpendicular to major axis 2 between the center of rigidity and the center of mass
b_2 is the width of the diaphragm perpendicular to major axis 2
m is the number of walls and braced frames resisting lateral force in direction 1
n is the number of walls and braced frames resisting lateral force in direction 2

Eq. 12.14-2 A and B need not be checked where a structure fulfills all the following limitations:

1. The arrangement of walls or braced frames is symmetric about each major axis direction.
2. The distance between the two most separated lines of walls or braced frames is at least 90 percent of the dimension of the structure perpendicular to that axis direction.
3. The stiffness along each of the lines considered for item 2 above is at least 33 percent of the total stiffness in that axis direction.

9. Lines of resistance of the lateral force-resisting system shall be oriented at angles of no more than 15° from alignment with the major orthogonal horizontal axes of the building.
10. The simplified design procedure shall be used for each major orthogonal horizontal axis direction of the building.
11. System irregularities caused by in-plane or out-of-plane off-sets of lateral force-resisting elements shall not be permitted.

EXCEPTION: *Out-of-plane and in-plane offsets of shear walls are permitted in two-story buildings of light-frame construction provided that the framing supporting the upper wall is designed for seismic force effects from overturning of the wall amplified by a factor of 2.5.*

12. The lateral-load-resistance of any story shall not be less than 80 percent of the story above.

12.14.8 Simplified Lateral Force Analysis Procedure. An equivalent lateral force analysis shall consist of the application of equivalent static lateral forces to a linear mathematical model of the structure. The lateral forces applied in each direction shall sum to a total seismic base shear given by Section 12.14.8.1 and shall be distributed vertically in accordance with Section12.14.8.2.For purposes of analysis, the structure shall be considered fixed at the base.

12.14.8.1 Seismic Base Shear. The seismic base shear, V, in a given direction shall be determined in accordance with Eq. 12.14-9:

$$V = \frac{FS_{DS}}{R}W \qquad (12.14\text{-}11)$$

where

$$S_{DS} = \frac{2}{3}F_a S_a$$

where F_a is permitted to be taken as 1.0 for rock sites, 1.4 for soil sites, or determined in accordance with Section 11.4.3. For the purpose of this section, sites are permitted to be considered to be rock if there is no more than 10 ft (3 m) of soil between the rock surface and the bottom of spread footing or mat foundation. In calculating S_{DS}, S_s shall be in accordance with Section 11.4.1, **but need not be taken larger than 1.5**.

$F = 1.0$ for one-story buildings
$F = 1.1$ for two-story buildings
$F = 1.2$ for three-story buildings

R = the response modification factor from Table 12.14-1

W = effective seismic weight of structure that shall include the total dead load and other loads listed in the following text

1. In areas used for storage, a minimum of 25 percent of the floor live load (floor live load in public garages and open parking structures need not be included).
2. Where provision for partitions is required by Section 4.2.2 in the floor load design, the actual partition weight, or a minimum weight of 10psf (0.48kN/m2) of floor area, whichever is greater.
3. Total operating weight of permanent equipment.
4. Where the flat roof snow load, P_f ,exceeds 30 psf (1.44 kN/m2), 20 percent of the uniform design snow load, regardless of actual roof slope.

12.14.8.2 Vertical Distribution. The forces at each level shall be calculated using the following equation:

$$F_x = \frac{w_x}{W} V \qquad (12.14\text{-}12)$$

where w_x = the portion of the effective seismic weight of the structure, W, at level x.

12.14.8.3 Horizontal Shear Distribution. The seismic design story shear in any story, V_x (kip or kN), shall be determined from the following equation:

$$V_x = \sum_{i=x}^{n} F_i \qquad (12.14\text{-}13)$$

where F_i = the portion of the seismic base shear, V (kip or kN) induced at Level, i.

12.14.8.3.1 Flexible Diaphragm Structures. The seismic design story shear in stories of structures with flexible diaphragms, as defined in Section 12.14.5, shall be distributed to the vertical elements of the lateral force resisting system using tributary area rules. Two-dimensional analysis is permitted where diaphragms are flexible.

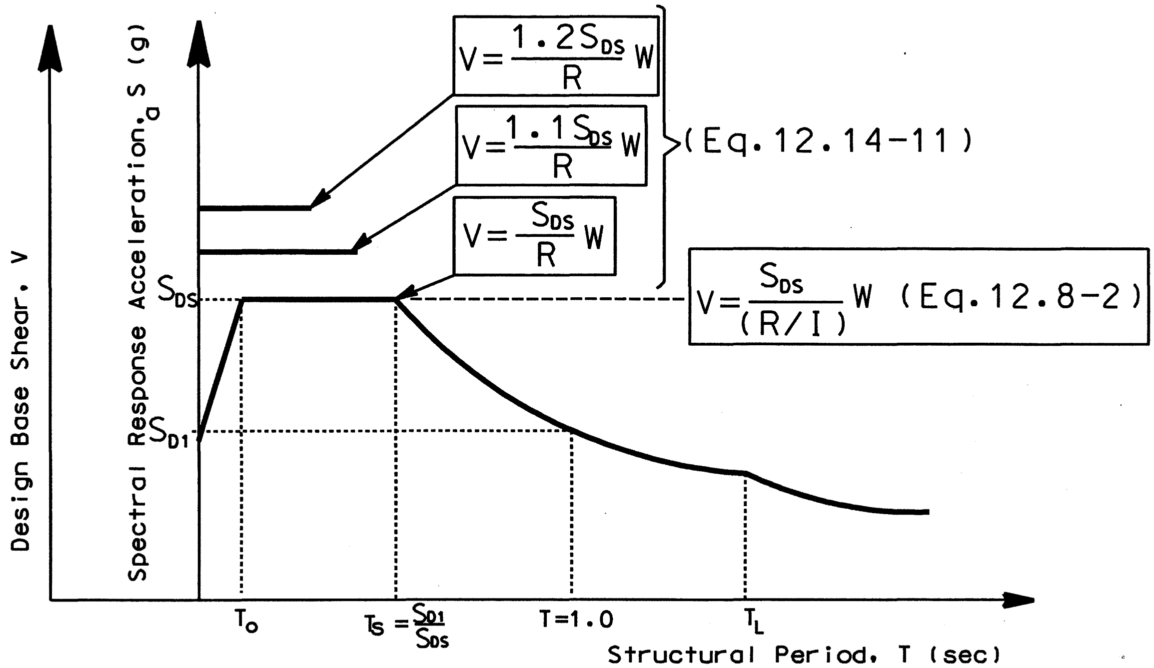
12.14.8.3.2 Structures with Diaphragms That Are Not Flexible. For structures with diaphragms that are not flexible, as defined in Section 12.14.5, the seismic design story shear, V_x , (kip or kN) shall be distributed to the various vertical elements of the seismic force–resisting system in the story under consideration based on the relative lateral stiffnesses of the vertical elements and the diaphragm.

12.14.8.3.2.1 Torsion. The design of structures with diaphragms that are not flexible shall include the torsional moment, M_t (kip-ft. or KN-m) resulting from eccentricity between the locations of center of mass and the center of rigidity.

12.14.8.4 Overturning. The structure shall be designed to resist overturning effects caused by the seismic forces determined in Section12.14.8.2.The foundations of structures shall be designed for not less than 75 percent of the foundation overturning design moment, M_f (kip-ft or kN-m) at the foundation-soil interface.

12.14.8.5 Drift Limits and Building Separation. Structural drift need not be calculated. Where a drift value is needed for use in material standards, to determine structural separations between buildings, for design of cladding, or for other design requirements, it shall be taken as 1 percent of building height unless computed to be less. All portions of the structure shall be designed to act as an integral unit in resisting seismic forces unless separated structurally by a distance sufficient to avoid damaging contact under the total deflection.

Simplified (Alternative) Lateral Force Procedure At a Glance



Occupancy Category	Maximum Height	Soil Type (Site Class)	Structural Systems	Seismic Design Category (SDC) "A" to "E"NO "F"
I & II (See Table 1604.5)	Three Stories (See Table 12.14-1)	A to D (NO E & F Type)	1- Bearing Wall 2-Building Frame (See Table 12.14-1)	Only Table 11.6-1 (based on S_{DS} and $S_S \leq 1.5$ *§12.14.8.1*)

- $S_{DS} = 2/3\ F_a S_s$ (§12.14.8.1)
- F_a is permitted to be taken as 1.0 for rock sites & 1.4 for soil sites
- $S_s \leq 1.5$ (§12.14.8.1)
- $\rho = 1.0$
- $\Omega_0 = 2.5$ (§12.14.3.2.1)
- Torsional moment due to M_t
- Drift = 1% of building height unless computed to be less

The advantageous of the simplified method could be summarized as follows:

a. NO need to calculate the period of the structure.

b. NO need to calculate the drift and it will be taken as 1% of the building height unless computed to be less per §12.14.8.5 i.e. the simplified method is used for buildings for which the drift is not controlling factor in design.

c. Redundancy factor " ρ " will be taken, i.e. NO need to calculate it.

d. Simplified distribution (rectangular) of the base shear.

4.10 – DIRECTION OF LOADING & ORTHOGONAL EFFECTS

4.11– DYNAMIC ANALYSIS

4-12 MPLE PROBLEMS

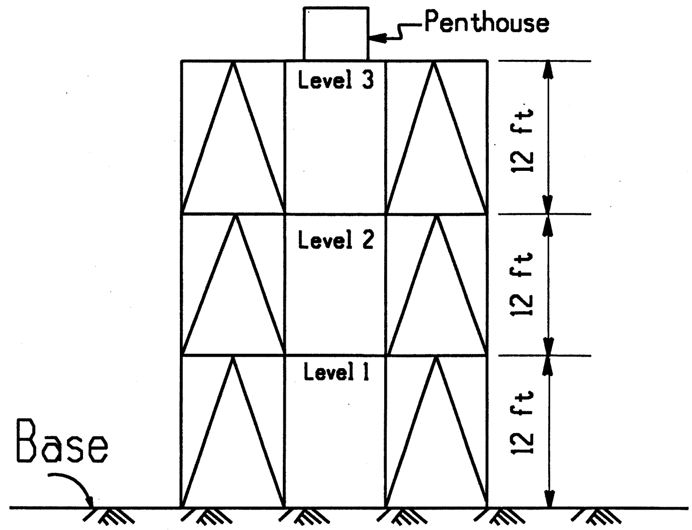
Sample Problem 4-1: Structural Period "Approximate Method"

Given: Three-story steel special concentrically braced frame (CBF) office building with a penthouse on the roof as shown is located in central California. The building with soil type C and SDC "C". The story height is 12 feet. S_{DS} & S_{D1} are 0.42g and 0.26g respectively, The effective seismic dead load "W" for levels 1& 2 is 120 kips each and for the roof including the penthouse is 90 kips.

Find: The structure period "T" is most nearly

Answer:

A) 0.29 s B) 0.30 s
C) 0.44 s D) 0.49 s



Solution:

Which method should be used to calculate T ?

i) Approximate method Eq. (12.8-7): YES, enough information are given (h_n, type of structure to get C_t & x)

ii) Approximate method Eq. (12.8-8): NO, only for concrete and steel moment resisting frames not exceeding 12 stories and the story height is at least 10 ft

$$T_a = 0.1N \qquad (12.8\text{-}8)$$

iii) Rayleigh Method Eq. (15.4-6): NO, not enough information (forces & displacements)

$$T = 2\pi\sqrt{\left(\sum_{i=1}^{n} w_i\delta_i^2\right) \div \left(g\sum_{i=1}^{n} f_i\delta_i\right)} \qquad (15.4-6)$$

$$T_a = C_t\, h_n^x \qquad (12.8\text{-}7)$$

For CBF, C_t = 0.020 & x =0.75 (for all other structures, Table 12.8-2)

$$T = 0.020(36)^{3/4} = 0.29\,Sec$$

Answer B: wrong answer, 0.1 N = 0.30 s

Answer C: wrong answer if C_t= 0.030 is chosen

Answer D: wrong answer if C_t= 0.028 is chosen

Answer: A ⇐

Note:

1- When you pick the value of C_t, the type of the structure should match the description given in the Code.

Sample Problem 4-2: Base Shear (V) Calculation

Given: Same as Sample problem 4-1

Find: Static design base shear, "V" is most nearly

Answer: A) 51.3 kips B) 49.30 kips C) 27.54 kips D) 23.10 kips

Solution:

Which method should be used to calculate V?

ii) ELF (§12.8): YES, number of stories, type of structure, soil type, occupancy category.
iii) Simplified (§12.14): MAY BE, if the requirements in section 12.14.1.1 are met.
iv) Dynamic: NO, not enough information (acceleration or response spectra).

Steps using ELF Method:

1) Calculate the structure period T
2) Calculate the period $T_S = \frac{S_{D1}}{S_{DS}}$ (T_S is going to be used for comparison only)
3) If $T < T_S$, Eq. (12.8-2) governs and no need to check the minimum values of V (Eqs. 12.8-5 & 12.8-6)

I = 1.0, 2007 CBC Table 1604.5 & Table 11.5-1 (Occupancy Category I)
R = 6.0, Table 12.2-1(Building Frame Systems- Special Steel CBF)

$T_S = \frac{0.26g}{0.42g} = 0.62\ Sec > T = 0.29\ Sec$, Eq. (12.8-2) governs

Also, $T = 0.29s > T_0 = 0.20T_s = 0.20 \times 0.63 = 0.126s$

$$C_S = \frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{0.42}{6/1} = 0.07 \qquad (12.8\text{-}2)$$

$$V = C_s W = 0.07\,(120+120+90) = 0.07\,(330) = 23.10 \text{ kips (Max.)} \qquad (12.8\text{-}1)$$

Note : Eq. (12.8-3) yields a value greater than Eq. (12.8-2) which is the maximum specified by the Code.

$$C_S = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.26}{0.29\,(6/1)} = 0.1494 \quad for \quad T \le T_L \qquad (12.8\text{-}3)$$

$$V = C_s W = 0.1494\,(120+120+90) = 0.1494(330) = 49.30 \text{ kips} \qquad (12.8\text{-}1)$$

Answer: D ⇐

NOTE: Building up to 4 story buildings (low rise structures = short period structures) most likely will be controlled by the Eq. 12.8-2

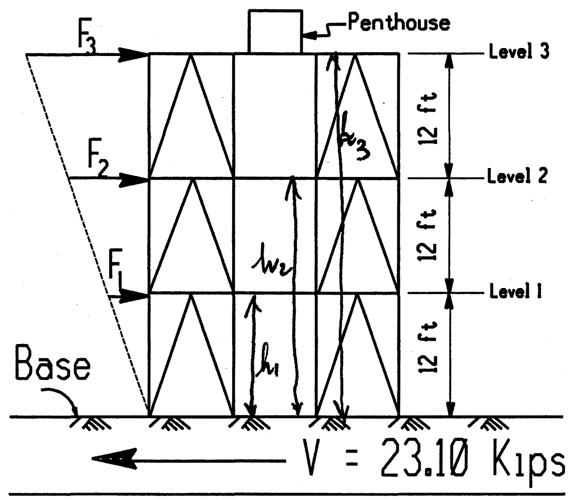
Sample Problem 4-3: Base Shear Distribution

Given: Three-story steel special concentrically braced frame (CBF) office building with a penthouse on the roof as shown is located in central California. The building with soil type C and SDC "C". The seismic base shear was found to be 23.10 kips

Find: The seismic shear force at level three is most nearly:

Answer:

A) 23.01 kips B) 9.91 kips
C) 8.81 kips D) 4.38 kips



Solution: $V = F_1 + F_2 + F_3$

Which method should be used to distribute the base shear?

i)The base shear can be distributed among levels using the following Equation which is used only if the base shear *(V)* was calculated using the : (ELF) Method" :

$$F_x = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} V \qquad (12.8-11)$$

ii) Also, the base shear can be distributed among levels using the following Equation which is used only if the base shear *(V)* was calculated using the : Simplified Static Method":
(3 story or less)

$$F_x = \frac{w_x}{W} V \qquad (12.14\text{-}12)$$

The base shear is given in the problem statement and it is not stated if the shear was calculated using the "ELF" or "Simplified". The base shear given in the problem will be assumed that it is based in the "ELF" and the distribution should be done according to Eq. 12.8-11). From Sample Problem 4-1, $T = 0.29$ s < 0.5 s, therefore the exponent $k = 1$ (§12.8-3)

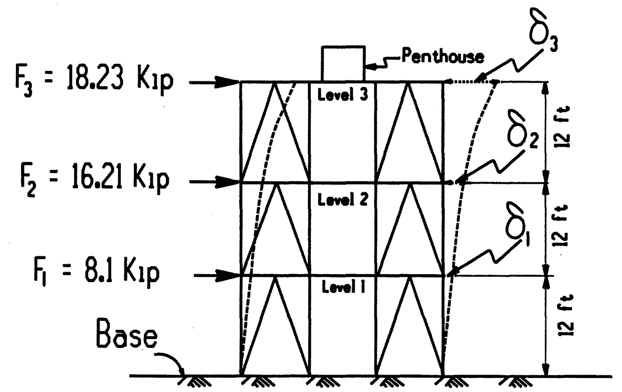
Level	h_x^k (ft)	W_x (kip)	$w_x h_x^k$	$\frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$	F_x (kips)
1	12	120	1440	0.1905	4.38
2	24	120	2880	0.3810	8.81
3	36	90	3240	0.4285	9.91
$\sum$			7560		23.10

Answers "A" is the total force at the base & Answers "C" is the force at level "2"
Answers "D" is the force at level "1"

Answer: B ⇐

Sample Problem 4-4: Structural Period "Method "B"

Given: Three-story steel special concentrically braced frame (CBF) building with a penthouse on the roof as shown. S_{DS} & S_{D1} are 0.42g and 0.26g respectively, The effective seismic dead load "W" for levels 1& 2 is 120 kips each and for the roof including the penthouse is 90 kips. The story height is 12 feet. The seismic forces at each level are shown. The lateral displacements are δ_1=0.30", δ_2=0.70" δ_3=1.15".



Find: The structural period "*T*" is most nearly

Answer: A) 0.29 *Sec.* B) 0.42 *Sec.* C) 0.52 *Sec.* D) 0.74 *Sec.*

Solution: *Which method should be used to calculate T ?*

i) Approximate Method Eq. (12.8-7): YES, enough information are given (h_n, type of structure to get C_t & x)

ii) Rayleigh Method Eq. (15.4-6): YES, enough information (forces & displacements)

$$T = 2\pi\sqrt{\left(\sum_{i=1}^{n} w_i\delta_i^2\right) \div \left(g\sum_{i=1}^{n} f_i\delta_i\right)} \qquad (15.5-6)$$

$$\sum_{i=1}^{n} w_i\delta_1^2 = (120)(0.30)^2 + (120)(0.70)^2 + (90)(1.15)^2 = 188.6\ kip.in^2$$

$$g\sum_{i=1}^{n} f_i\delta_i = (386.64\ in./\sec.^2)\{(8.1)(0.30) + (16.21)(0.70) + (18.23)(1.15)\} = 13{,}424.12\ kip\frac{in^2}{Sec^2}$$

$$T = 2\pi\sqrt{\left(\sum_{i=1}^{n} w_i\delta_i^2\right) \div \left(g\sum_{i=1}^{n} f_i\delta_i\right)} = 2\pi\sqrt{188.6\ kip.in^2 \Big/ 13{,}424.12\,kip\frac{in^2}{Sec^2}} = 0.74\ Sec$$ XX

The fundamental period, T, shall not exceed the product of the coefficient for upper limit on calculated period (C_u) from Table 12.8-1 and the approximate fundamental period, T_a, determined in accordance with section 12.8.2.1

$$T \le C_U T_a \qquad (4\text{-}1)$$

NOTE: The above limit of T is applicable **only for strength calculations**. For drift calculation, the actual value of T will be used without checking the limit of Equation (4-1)

0.74 *Sec.* ≤ 1.44(0.29) = 0.42 *Seconds*

Answer A: wrong answer because this the period based on Approximate Method.

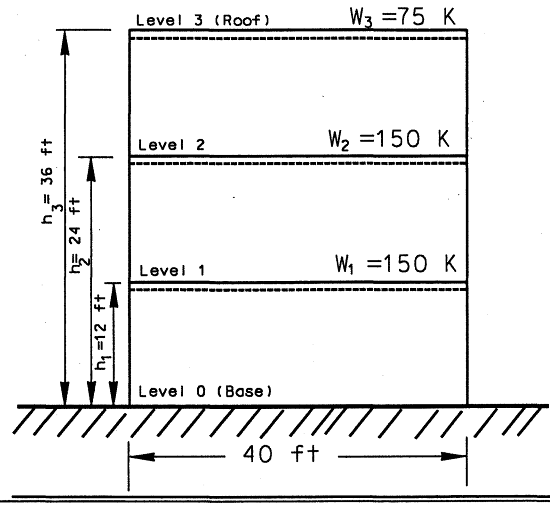
Answer C: wrong answer

Answer D: wrong answer because this is the calculated value without Code limitations

Answer: B ⇐

Sample Problem 4-5: Seismic Base V -Simplified Method

Given: Three-story wood structural panel wall building is shown. The building is classified as Occupancy Category II. The story height is 12 feet. The weight of level 1 & 2 are $W_1 = W_2 =$ 150 kips while level three is $W_3 = 75$ kips. The $S_{DS} = 1.0$ and SDC "D". The soil type is not known at the site.



Find: The seismic base shear *V* is most nearly

Answer: A) 69.23 kips B) 63.46 kips
C) 57.69 kips D) 47.69 kips

Solution: *Which method should be used to calculate the base shear?*

i) ELF §12.8 : <u>YES</u>, number of stories, type of structure, soil type, period & occupancy category are matching the requirements for the ELF method.
See Table 12.6-1

ii) Simplified (Alternate § 12.14): <u>YES</u>, number of stories (3 stories), type of construction (bearing wall system), occupancy category, soil type, SDC. However, ALL the 12 requirements of § 12.14 should be met before using the simplified method.

iii) Dynamic: <u>NO</u>, not enough information (acceleration or response spectra)

$$V = \frac{FS_{DS}}{R} W \qquad (12.14\text{-}11)$$

$F = 1.2$ (3 stories) §12.14.8.1

$R = 6.5$ (Table 12.14-1, system # 13 under bearing wall system)

$$V = \frac{FS_{DS}}{R} W = \frac{1.2 \times 1.0}{6.5} \times 375 = 69.23\, kips$$

NOTE: If the Static Lateral force procedure is used in lieu of the Simplified method, the calculated base shear will be:

$$C_S = \frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{1.0}{6.5/1} = 0.1538 \qquad (12.8\text{-}2)$$

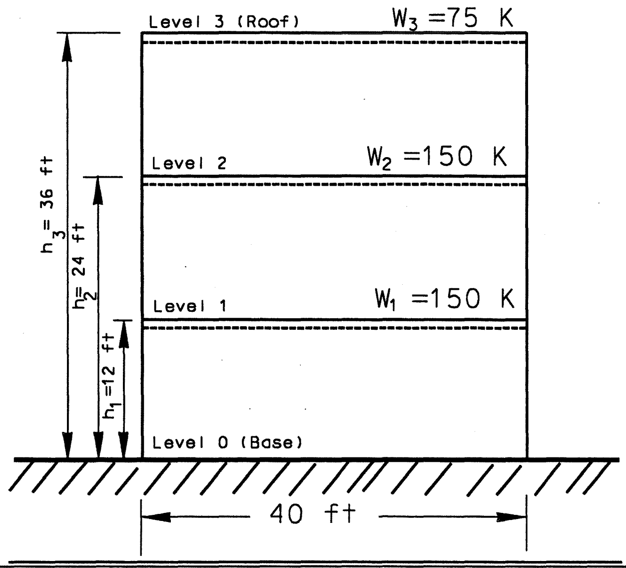
$$V = C_s W = (0.1538)(375) = 57.69 \text{ kips (answer C)} \qquad (12.8\text{-}1)$$

Which is about 20 % less than the simplified method.
The advantage of the simplified method is that there is no need to calculate drift and it will be taken as 1% of the building height unless computed to be less per §12.14.8.5 i.e. the simplified method is used for buildings for which the drift is not controlling factor in design.

Answer: A ⇐

Sample Problem 4-6: Base Shear Distribution -Simplified Method

Given: Three-story wood structural panel wall building is shown. The story height is 12 feet. The weight of level 1 & 2 are $W_1 = W_2 = 150$ kips while level three is $W_3 = 75$ kips. The seismic base shear using the simplified static was calculated and found to be 69.23 kips.



Find: The seismic force at the level three is most nearly

Answer: A) 27.69 kips B) 24.69 kips
C) 22.59 kips D) 13.85 kips

Solution: *Which method should be used to distribute the base shear?*

i) The base shear would be distributed over levels using equation (12.8-11) which is used only if the base shear *(V)* was calculated using the " ELF"

$$F_x = C_{vx} V \qquad (12.8\text{-}11)$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad (12.8-12)$$

ii) Also, the base shear would be distributed among levels using equation (12.14-12) which is used only if the base shear *(V)* was calculated using the "Simplified Method"

$$F_x = \frac{w_x}{W} V \qquad (12.14\text{-}12)$$

From the problem statement, it is clear that the type of structure (wood structural panel wall building) is eligible for the "Simplified Method" and the base shear was calculated based on that method. Therefore,

$$F_3 = \frac{w_x}{W} V = \frac{75}{375}(69.23) = 13.85\, kips$$

$$F_1 = F_2 = \frac{w_x}{W} V = \frac{150}{375}(69.23) = 27.69\, kips$$

Answers "A" is the force at level "1" or level "2"
Answers "B & C" are wrong answers

Answer: D⇐

Sample Problem 4-7: Structural Period

Given: A steel special moment–resisting frame (SMF) structure is shown.

Find: The structural period is most nearly

Answer: A) 0.55 sec. B) 0.83 sec. C) 0.70 sec. D) 0.97 sec.

Solution: The appropriate method will be used because we do not have enough information about the Rayleigh method. The height of the structure above its base is 84 feet. The 18-feet basement below the grade line is not considered in determining the "h_n"

$$T = C_t\,(h_n)^x \qquad (12.8-7)$$

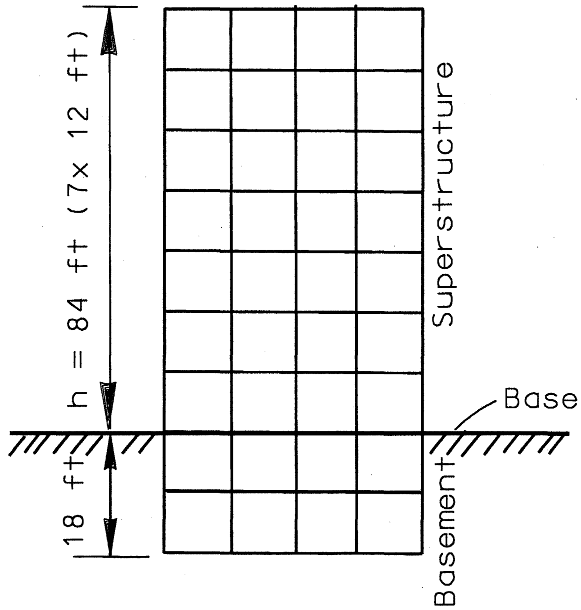
For SMRF: $C_t = 0.028$ & $x = 0.8$

$T = 0.028(84)^{0.8} = 0.97\text{sec.} \Leftarrow$

Also,

$T_a = 0.1\ N = 0.1(7) = 0.70$ s (12.8-8)

Answer: D⇐



Answer A is for C_t =0.020

Answer B is for C_t =0.030

Answer C is for 0.1N

Sample Problem 4-8: Structural Period

Given: A steel eccentric braced framed (EBF) structure is shown.

Find: The structural period is most nearly

Answer: A) 0.31 sec. B) 0.36 sec. C) 0.46 sec. D) 0.54 sec.

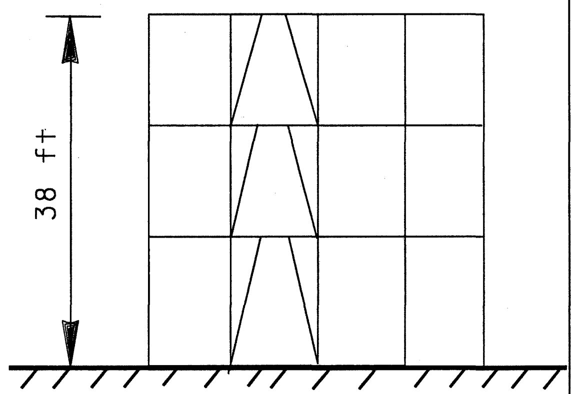
Solution:
The appropriate method because we do not have enough information about the Rayleigh Method.

Also, Equation 12.8-8 (0.1 N) is not applicable because of the type of the structure (Not a moment resisting frame)

For SMRF, $C_t = 0.030$, x =0.75 & $h_n = 38$ feet

$$T = C_t\,(h_n)^x \ (12.8-7) \rightarrow T = 0.030(38)^{3/4} = 0.46\text{sec.} \Leftarrow$$

Answer: C⇐



Answer A is for C_t =0.020
Answer B is random answer
Answer D is for C_t =0.035

Sample Problem 4-9: Structural Period

Given: A concrete special moment–resisting frame (SMF) structure is shown.

Find: The structural period is most nearly

Answer: A) 0.45 sec. B) 0.41 sec. C) 0.34 sec. D) 0.27 sec.

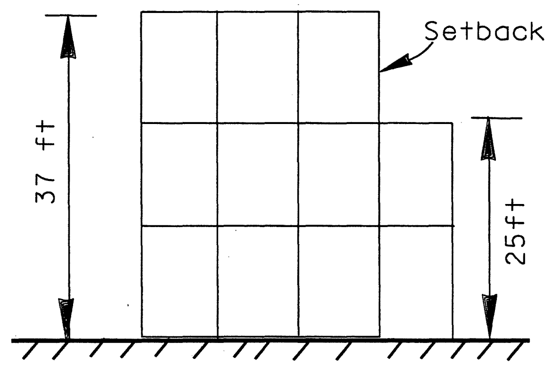
Solution: The appropriate method will be used because we do not have enough information about the other method. For SMF, C_t = 0.016 & x =0.9

- The height of tallest part of the structure is 37 feet and the height at the setback is 25 feet.
- Roof penthouses are generally not considered in determining "h_n" (see Sample Problem 4-1), but heights of setbacks are included except when the setback represents more than a 130 % change in the lateral force system dimension. This type of vertical irregularity (130%) requires a dynamic analysis if the structure has more than 5 stories or 65 feet in height.

h_n = 36 feet

$$T = C_t\,(h_n)^x \rightarrow T = 0.016(37)^{0.9} = 0.41\text{ sec.} \Leftarrow$$

Answer: B ⇐



Answer A is for C_t =0.03
Answer C is $h_n = 25$ feet
Answer D is for C_t =0.020

Sample Problem 4-10: Structural Period

Given: A concrete tilt-up building with a panelized wood roof is shown

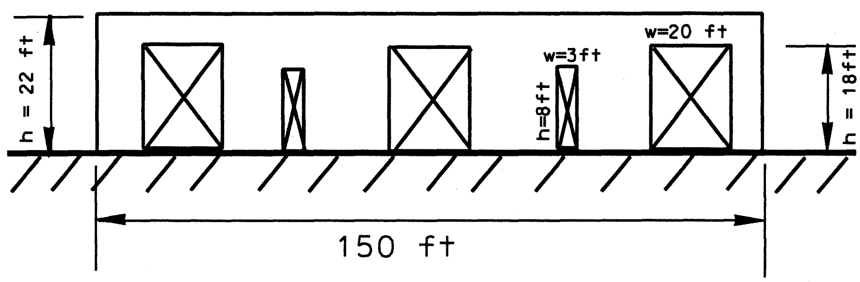
Find: The structural period is most nearly

Answer: A) 0.17 sec.
B) 0.20 sec.
C) 0.26 sec.
D) 0.30 sec.

Solution:

$$T = C_t\,(h_n)^x \qquad (12.8-7)$$

$$\rightarrow T = 0.020(22)^{3/4} = 0.20\text{ sec.} \Leftarrow$$



Answer A is for C_t =0.02 & $h_n = 18$ feet
Answer C is $h_n = 18$ feet & C_t =0.03
Answer D is for C_t =0.030 & $h_n = 22$ feet

Note: This type of structural system has a flexible roof diaphragm and rigid walls. The Code formula for period does not take into consideration the fact the period is highly dependent on the roof diaphragm.

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Chapter 5

Seismic Design Requirements for Nonstructural Components Chapter 13 ASCE7-05

Topics to be covered

- Introduction
- Exemption of Certain Nonstructural Components
- Nonstructural Components Are Considered Non-Building Stru.
- Seismic Design Category (SDC) for Nonstructural Components
- Design Seismic Force, F_P
- Amplification Factor, a_P
- Component Importance Factor, I_P
- Response Modification Factor, R_P
- The Ratio z/h
- Architectural Components
- Mechanical and Electrical Components
- Nonstructural Components Anchorage

16 Sample Problems

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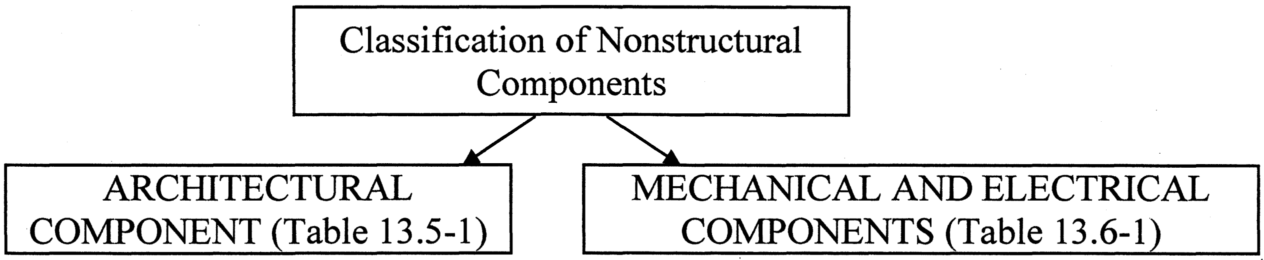
Chapter 5- Seismic Design Requirements for Nonstructural Components

5.1 INTRODUCTION

§13.1.1 Scope. This chapter establishes minimum design criteria for nonstructural components that are permanently attached to structures and for their supports and attachments.



5.2 SEISMIC DESIGN CATEGORY (SDC) FOR NONSTRUCTURAL COMPONENTS

§13.1.2 Seismic Design Category. Nonstructural components shall be assigned to the same seismic design category as the structure that they occupy or to which they are attached.

5.3 EXEMPTIONS OF CERTAIN NONSTRUCTURAL COMPONENTS

§13.1.4 Exemptions. The following nonstructural components are exempt from the requirements of this section:

1. Architectural components in Seismic Design Category B other than parapet supported by bearing walls or shear walls provided that the component importance factor, I_p, is equal to 1.0.
2. Mechanical and electrical components in Seismic Design Category B.
3. Mechanical and electrical components in Seismic Design Category C provided that the component importance factor, I_p, is equal to 1.0.
4. Mechanical and electrical components in Seismic Design Categories D, E, and F where the component importance factor, I_p, is equal to 1.0 and either:
   a. Flexible connections between the components and associated ductwork, piping, and conduit are provided.
   b. Components are mounted at 4 ft or less above a floor level and weigh 400 lb or less.
5. Mechanical and electrical components in Seismic Design Categories D, E, and F where the component importance factor, I_p, is equal to 1.0 and both:
   a. Flexible connections between the components and associated ductwork, piping, and conduit are provided, and

b. The components weigh 20 lb (89 N) or less or, for distribution systems, weighing 5 lb/ft (73 N/m) or less.

5.4 NONSTRUCTURAL COMPONENTS ARE CONSIDERED NONBUILDING STRUCTURES

§13.1.5 Applicability of Nonstructural Component Requirements. Where the weight of a nonstructural component is greater than or equal to 25 percent of the effective seismic weight, W, defined in Section 12.7.2, the component shall be classified as a nonbuilding structure and shall be designed in accordance with Section 15.3.2.

Nonbuilding structures (including storage racks and tanks) that are supported by other structures shall be designed in accordance with Chapter 15. Where Section 15.3 requires that seismic forces be determined in accordance with Chapter 13 and values for R_p are not provided in Table 13.5-1 or 13.6-1, R_p shall be taken as equal to the value of R listed in Section 15. The value of a_p shall be determined in accordance with footnote a of Table 13.5-1 or 13.6-1.

5.5 GENERAL DESIGN REQUIREMENTS FOR NOSTRUCTURAL COMPONENTS

§13.2.1 Applicable Requirements for Architectural, Mechanical, and Electrical Components, Supports, and Attachments. Architectural, mechanical, and electrical components, supports, and attachments shall comply with the sections referenced in Table 13.2-1. These requirements shall be satisfied by one of the following methods:

1. Project-specific design and documentation prepared and submitted by a registered design professional.
2. Submittal of the manufacturer's certification that the component is seismically qualified by
    a. Analysis.
    b. Testing in accordance with the alternative set forth in Section 13.2.5.
    c. Experience data in accordance with the alternative set forth in Section 13.2.6

TABLE 13.2-1 APPLICABLE REQUIREMENTS FOR ARCHITECTURAL, MECHANICAL, AND ELECTRICAL COMPONENTS: SUPPORTS AND ATTACHMENTS

Nonstructural Element (i.e., Component, Support, Attachment)	General Design Requirements Section 13.2	Force and Displacement Requirements Section 13.3	Attachment Requirements Section 13.4	Architectural Component Requirements Section 13.5	Mechanical and Electrical Component Requirements Section 13.6
Architectural Components and Supports and Attachments for Architectural Components	X	X	X	X	
Mechanical and Electrical Components with $Ip > 1$	X	X	X		X
Supports and Attachments for Mechanical and Electrical Components	X	X	X		X

5.6 PARAMETERS FOR NONSTRUCTURAL COMPONENTS SESISMIC DEMAND EQUATION

§13.3.1 Seismic Design Force. The horizontal seismic design force (F_p) shall be applied at the component's center of gravity and distributed relative to the component's mass distribution and shall be determined in accordance with Eq. 13.3-1:

$$F_p = \frac{0.4 a_p S_{DS} W_p}{\left(R_p / I_p\right)}\left(1 + 2\frac{z}{h}\right) \qquad (13.3-1)$$

F_p is not required to be taken as greater than

$$(F_P)_{\max} \leq 1.6 S_{DS} I_P W_p \quad \Leftarrow \text{max} \qquad (13.3-2)$$

NOTES:

1- $(F_p)_{\max}$ will not govern unless the nonstructural component is located near or at the roof level (i.e. $z \cong h$) and $(a_p/R_p) \geq 2.0$ [See Figure 5-1]

2- The units of (F_p), $(F_p)_{\max}$ and $(F_p)_{\min}$ will be the same units of W_p (kips, lbs, lb/ft) because the rest of the parameters in Equations 13.3-1 through 13.3-3 are dimentionless.

and F_p shall not be taken as less than

$$(F_P)_{\min} \geq 0.3 S_{DS} I_P W_p \quad \Leftarrow \text{min} \qquad (13.3-3)$$

NOTE: $(F_p)_{\min}$ needs to be checked when the nonstructural component is located near, at or below the base level (i.e. $z \cong 0$)

where

F_p = seismic design force
S_{DS} = spectral acceleration, short period, as determined from Section 11.4.4
a_p = component amplification factor that varies from 1.00 to 2.50 (select appropriate value from Table 13.5-1 or 13.6-1)
I_p = component importance factor that varies from 1.00 to 1.50 (see Section 13.1.3)
W_p = component operating weight
R_p = component response modification factor that varies from 1.00 to 12 (select appropriate value from Table 13.5-1 or 13.6-1)
z = height in structure of point of attachment of component with respect to the base. For items at or below the base, z shall be taken as "0". The value of z/h need not exceed 1.0
h = average roof height of structure with respect to the base

❖ Component Amplification Factor, a_P

Component amplification factor that varies from 1.00 to 2.50 and is determined from the following two tables:

TABLE 13.5-1 COEFFICIENTS FOR ARCHITECTURAL COMPONENT

Architectural Component or Element	a_p [a]	R_p [b]
1-Interior Nonstructural Walls and Partitions[b]		
Plain (unreinforced) masonry walls	1.0	1.5
All other walls and partitions	1.0	2.5
2-Cantilever Elements (Unbraced or braced to structural frame below its center of mass)		
Parapets and cantilever interior nonstructural walls	2.5	2.5
Chimneys and stacks where laterally braced or supported by the structural frame	2.5	2.5
3- Cantilever Elements (Braced to structural frame above its center of mass)		
Parapets	1.0	2.5
Chimneys and Stacks	1.0	2.5
Exterior Nonstructural Walls[b]	1.0[b]	2.5
4- Exterior Nonstructural Wall Elements and Connections[b]		
Wall Element	1.0	2.5
Body of wall panel connections	1.0	2.5
Fasteners of the connecting system	1.25	1.0
5-Veneer		
Limited deformability elements and attachments	1.0	2.5
Low deformability elements and attachments	1.0	1.5
6- Penthouses (except where framed by an extension of the building frame)	2.5	3.5
7- Ceilings		
All	1.0	2.5
8- Cabinets Storage cabinets and laboratory equipment	1.0	2.5
9- Access Floors		
Special access floors (designed in accordance with Section 13.5.7.2)	1.0	2.5
All other	1.0	1.5
10- Appendages and Ornamentations	2.5	2.5
11- Signs and Billboards	2.5	2.5
12- Other Rigid Components		
High deformability elements and attachments	1.0	3.5
Limited deformability elements and attachments	1.0	2.5
Low deformability materials and attachments	1.0	1.5
13- Other Flexible Components		
High deformability elements and attachments	2.5	3.5
Limited deformability elements and attachments	2.5	2.5
Low deformability materials and attachments	2.5	1.5

[a]A lower value for a_p shall not be used unless justified by detailed dynamic analysis. The value for a_p shall not be less than1.0. The value of $a_p = 1$ is for rigid components and rigidly attached components. The value of $a_p = 2.5$ is for flexible components and flexibly attached components. See Section 11.2 for definitions of rigid and flexible.

[b]Where flexible diaphragms provide lateral support for concrete or masonry walls and partitions, the design forces for anchorage to the diaphragm shall be as specified in Section 12.11.2.

From the foot note "a" above:

- The value of $a_p = 1$ is for rigid components and rigidly attached components.
  §11.2 Component, Rigid: Component, including its attachments, having a fundamental period less than or equal to 0.06 s.
- The value of $a_p = 2.5$ is for flexible components and flexibly attached components.
  §11.2 Component, Flexible: Component, including its attachments, having a fundamental period greater than 0.06 s.

TABLE 13.6-1 SEISMIC COEFFICIENTS FOR MECHANICAL AND ELECTRICAL COMPONENTS

A- MECHANICAL AND ELECTRICAL COMPONENTS	a_p[a]	R_p[b]
1- Air-side HVAC, fans, air handlers, air conditioning units, cabinet heaters, air distribution boxes, and other mechanical components constructed of sheet metal framing.	2.5	6.0
2- Wet-side HVAC, boilers, furnaces, atmospheric tanks and bins, chillers, water heaters, heat exchangers, evaporators, air separators, manufacturing or process equipment, and other mechanical components constructed of high-deformability materials.	1.0	2.5
3- Engines, turbines, pumps, compressors, and pressure vessels not supported on skirts and not within the scope of Chapter 15.	1.0	2.5
4- Skirt-supported pressure vessels not within the scope of Chapter 15.	2.5	2.5
5- Elevator and escalator components.	1.0	2.5
6- Generators, batteries, inverters, motors, transformers, and other electrical components constructed of high deformability materials.	1.0	2.5
7- Motor control centers, panel boards, switch gear, instrumentation cabinets, and other components constructed of sheet metal framing.	2.5	6.0
8- Communication equipment, computers, instrumentation, and controls.	1.0	2.5
9- Roof-mounted chimneys, stacks, cooling and electrical towers laterally braced below their center of mass.	2.5	3.0
10- Roof-mounted chimneys, stacks, cooling and electrical towers laterally braced above their center of mass.	1.0	2.5
11- Lighting fixtures.	1.0	1.5
12- Other mechanical or electrical components.	1.0	1.5

B- VIBRATION ISOLATED COMPONENTS AND SYSTEMS[b]	a_p[a]	R_p[b]
1- Components and systems isolated using neoprene elements and neoprene isolated floors with built-in or separate elastomeric snubbing devices or resilient perimeter stops.	2.5	2.5

2- Spring isolated components and systems and vibration isolated floors closely restrained using built-in or separate elastomeric snubbing devices or resilient perimeter stops.	2.5	2.0
3- Internally isolated components and systems.	2.5	2.0
4- Suspended vibration isolated equipment including in-line duct devices and suspended internally isolated components.	2.5	2.5

C- DISTRIBUTION SYSTEMS	$a_p{}^a$	$R_p{}^b$
1- Piping in accordance with ASME B31, including in-line components with joints made by welding or brazing.	2.5	12.0
2- Piping in accordance with ASME B31, including in-line components, constructed of high or limited deformability materials, with joints made by threading, bonding, compression couplings, or grooved couplings.	2.5	6.0
3- Piping and tubing not in accordance with ASME B31, including in-line components, constructed of high-deformability materials, with joints made by welding or brazing.	2.5	9.0
4- Piping and tubing not in accordance with ASME B31, including in-line components, constructed of high-or limited-deformability materials, with joints made by threading, bonding, compression couplings, or grooved couplings.	2.5	4.5
5- Piping and tubing constructed of low-deformability materials, such as cast iron, glass, and nonductile plastics.	2.5	3.0
6- Ductwork, including in-line components, constructed of high-deformability materials, with joints made by welding or brazing.	2.5	9.0
7- Ductwork, including in-line components, constructed of high-or limited-deformability materials with joints made by means other than welding or brazing.	2.5	6.0
8- Ductwork, including in-line components, constructed of low-deformability materials, such as cast iron, glass, and nonductile plastics.	2.5	3.0
9- Electrical conduit, bus ducts, rigidly mounted cable trays, and plumbing.	1.0	2.5
10- Manufacturing or process conveyors (nonpersonnel).	2.5	3.0
11- Suspended cable trays.	2.5	6.0

[a] A lower value for a_p is permitted where justified by detailed dynamic analyses. The value for a_p shall not be less than 1.0. The value of a_p equal to 1.0 is for rigid components and rigidly attached components. The value of a_p equal to 2.5 is for flexible components and flexibly attached components.

[b] Components mounted on vibration isolators shall have a bumper restraint or snubber in each horizontal direction. The design force shall be taken as 2 F_p if the nominal clearance (air gap) between the equipment support frame and restraint is greater than 0.25 in. If the nominal clearance specified on the construction documents is not greater than 0.25 in., the design force is permitted to be taken as F_p .

❖ Component Importance Factor, I_P

§13.1.3 Component Importance Factor. All components shall be assigned a component importance factor as indicated in this section. The component importance factor, I_p, shall be taken as 1.5 if any of the following conditions apply:

1. The component is required to function for life-safety purposes after an earthquake , including fire protection sprinkler systems.
2. The component contains hazardous materials.
3. The component is in or attached to an Occupancy Category IV structure and it is needed for continued operation of the facility or its failure could impair the continued operation of the facility.

All other components shall be assigned a component importance factor, I_p, equal to 1.0.

❖ Component Response Modification Factor, R_P

The component response modification factor varies from 1.0 to 12.0. Select appropriate value from Table 13.5-1 for ARCHITECTURAL COMPONENT or from Table 13.6-1 for MECHANICAL AND ELECTRICAL COMPONENTS.

❖ Component Operating Weight, W_P

The component operating weight is the maximum weight of the component and its contents assuming its full.

❖ The Ratio z/h

The two parameters z and h are defined as follows:

z = height in structure of point of attachment of component with respect to the base. For items at or below the base, z shall be taken as "0". The value of z/h need not exceed 1.0

h = average roof height of structure with respect to the base.

The following table and figure show the ratio z/h for different positions of the components:

Table 5-1 Values of z/h ratio

Position	-2	-1	0 (Base)	1	2	3	4	5 (Roof)
z/h	0	0	0	z_1/h	z_2/h	z_3/h	z_4/h	1

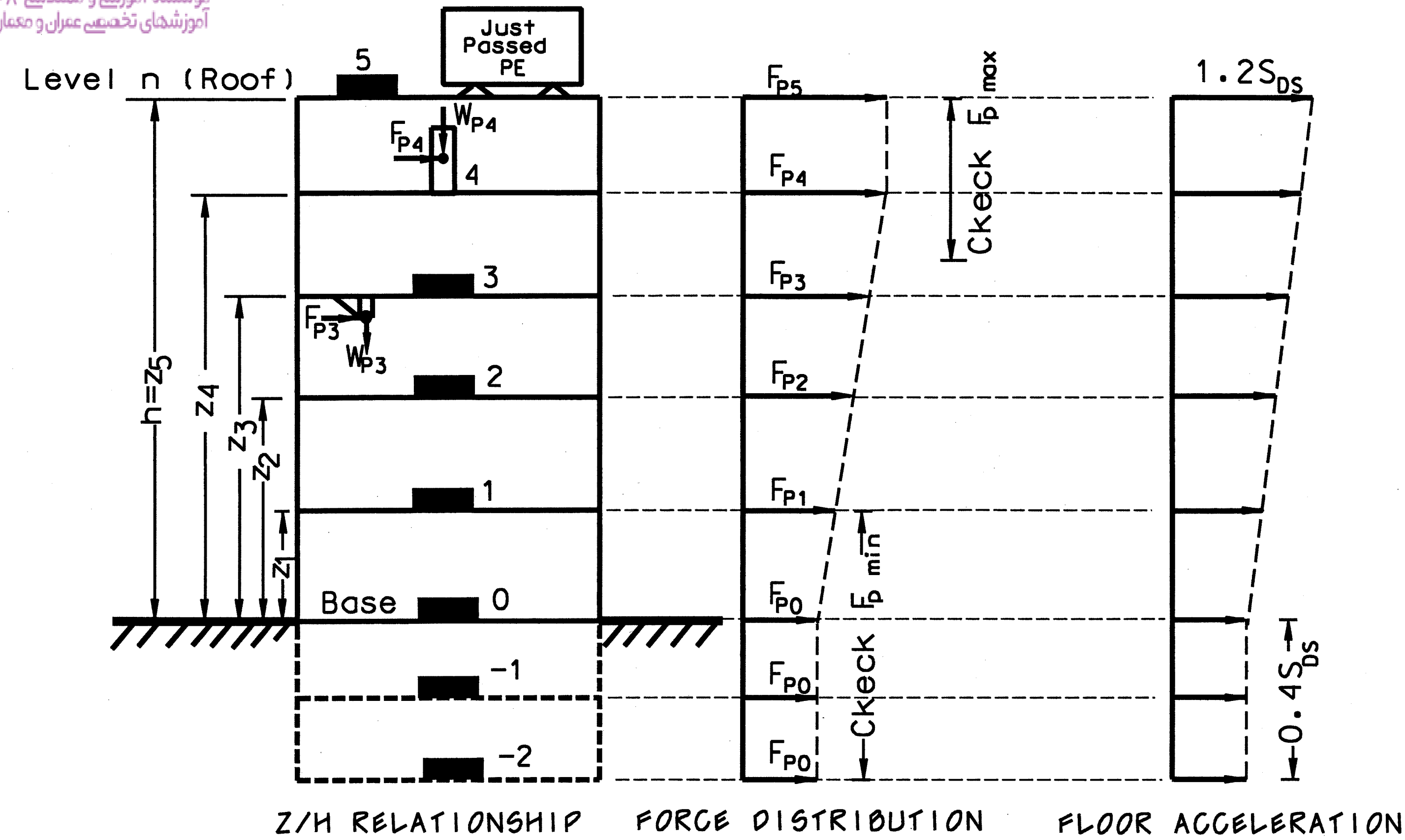


Figure 5-1 Relationship of z/h Ratio

Important Facts About The Components Horizontal Seismic Force, F_P

- The force (F_p) shall be applied independently in at least two orthogonal horizontal directions in combination with service loads associated with the component, as appropriate.
- For vertically cantilevered systems, the force F_p shall be assumed to act in any horizontal direction. In addition, the component shall be designed for a concurrent vertical force $\pm 0.2\ S_{DS}W_p$.
- The redundancy factor, ρ, is permitted to be taken equal to 1.0 and the overstrength factor, Ω_0, does not apply.
- Because all the parameters except W_p of the previous equations are dimentionless, the units of F_p will be the same units used for W_p (kips, k/ft, ksf, … etc)
- The force (F_p) shall be applied at the component's center of gravity and distributed relative to the component's mass distribution.

In lieu of the forces determined in accordance with Eq. 13.3-1, accelerations at any level are permitted to be determined by the modal analysis procedures of Section 12.9 with R = 1.0. Seismic forces shall be in accordance with Eq. 13.3-4:

$$F_p = \frac{a_i a_p W_p}{\left(\frac{R_p}{I_p}\right)} A_x \qquad (13.3-4)$$

Where a_i is the acceleration at level i obtained from the modal analysis and where A_x is the torsional amplification factor determined by Eq.12.8-14. Upper and lower limits of F_p, determined by Eqs. 13.3-2 and 13.3-3 shall apply.

5.7 ARCHITECTURAL COMPONENTS

§13.5.1 General. Architectural components, and their supports and attachments, shall satisfy the requirements of this section. Appropriate coefficients shall be selected from Table 13.5-1.

EXCEPTIONS: *Components supported by chains or otherwise suspended from the structure are not required to satisfy the seismic force and relative displacement requirements provided they meet all of the following criteria:*

1. *The design load for such items shall be equal to 1.4 times the operating weight acting down with a simultaneous horizontal load equal to 1.4 times the operating weight. The horizontal load shall be applied in the direction that results in the most critical loading for design.*
2. *Seismic interaction effects shall be considered in accordance with Section 13.2.3.*
3. *The connection to the structure shall allow a 360^o range of motion in the horizontal plane.*

§13.5.2 Forces and Displacements. All architectural components, and their supports and attachments, shall be designed for the seismic forces defined in Section 13.3.1.

Architectural components that could pose a life-safety hazard shall be designed to accommodate the seismic relative displacement requirements of Section 13.3.2. Architectural components shall be designed considering vertical deflection due to joint rotation of cantilever structural members.

❖ Amplification Factor for Architectural Components, a_P

The amplification factor that varies from 1.00 to 2.50 and is determined from table 13.5-1

❖ Response Modification Factor for Architectural Components, R_P

The component response modification factor that varies from 1.5 to 3.5 and it will selected from Table 13.5-1 for ARCHITECTURAL COMPONENTS

❖ Horizontal Seismic Force for Architectural Components, F_P

The horizontal seismic design force (F_p) shall be applied at the component's center of gravity and distributed relative to the component's mass distribution and shall be determined in accordance with Eq. 13.3-1:

$$F_p = \frac{0.4 a_p S_{DS} W_p}{\left(R_p / I_p\right)} \left(1 + 2\frac{z}{h}\right) \qquad (13.3-1)$$

F_p is not required to be taken as greater than

$$(F_P)_{\max} \leq 1.6 S_{DS} I_P W_p \quad \Leftarrow \text{max} \qquad (13.3-2)$$

and F_p shall not be taken as less than

$$(F_P)_{\min} \geq 0.3 S_{DS} I_P W_p \quad \Leftarrow \text{min} \qquad (13.3-3)$$

The following sections of ASCE7-05 list the detailed requirements for each of the Nonstructural Components listed below:

§ 13.5.3 Exterior Nonstructural Wall Elements and Connections.
§ 13.5.4 Glass.
§ 13.5.5 Out-of-Plane Bending.
§ 13.5.6 Suspended Ceilings.
§ 13.5.7 Access Floors.
§ 13.5.8 Partitions.
§13.5.9 Glass in Glazed Curtain Walls, Glazed Storefronts, and Glazed Partitions.

5.8 MECHANICAL AND ELECTRICAL COMPONENTS

§13.6.1 General. Mechanical and electrical components and their supports shall satisfy the requirements of this section. The attachment of mechanical and electrical components and their supports to the structure shall meet the requirements of Section 13.4. Appropriate coefficients shall be selected from Table 13.6-1.

EXCEPTIONS: Light fixtures, lighted signs, and ceiling fans not connected to ducts or piping, which are supported by chains or otherwise suspended from the structure, are not required to satisfy the seismic force and relative displacement requirements provided they meet all of the following criteria:

1. *The design load for such items shall be equal to 1.4 times the operating weight acting down with a simultaneous horizontal load equal to 1.4 times the operating weight. The horizontal load shall be applied in the direction that results in the most critical loading for design.*
2. *Seismic interaction effects shall be considered in accordance with Section 13.2.3.*
3. *The connection to the structure shall allow a 360° range of motion in the horizontal plane.*

Where design of mechanical and electrical components for seismic effects is required, consideration shall be given to the dynamic effects of the components, their contents, and where appropriate, their supports. In such cases, the interaction between the components and the supporting structures, including other mechanical and electrical components, shall also be considered.

§13.6.2 Component Period. The fundamental period of the mechanical and electrical component (and its attachment to the building), T_p, shall be determined by the following equation provided that the component and attachment can be reasonably represented analytically by a simple spring and mass single degree-of-freedom system:

$$T_p = 2\pi\sqrt{\frac{W_p}{K_p g}} \qquad (13.6\text{-}1)$$

Where:

T_p = component fundamental period
W_p = component operating weight
g = gravitational acceleration
K_p = stiffness of resilient support system of the component and attachment ,determined in terms of load per unit deflection at the center of gravity of the component

Alternatively, the fundamental period of the component in s (T_p) is permitted to be determined from experimental test data or by a properly substantiated analysis.

❖ **Amplification Factor for Mechanical & Electrical Components,** a_P

The amplification factor that varies from 1.00 to 2.50 and is determined from table 13.6-1

❖ **Response Modification Factor for Mechanical & Electrical Components,** R_P

The component response modification factor that varies from 1.5 to 12.0 and it will selected from Table 13.6-1 for MECANICAL and ELECTRICAL COMPONENTS

❖ **Horizontal Seismic Force for Mechanical & Electrical Components,** F_P

The horizontal seismic design force (F_p) shall be applied at the component's center of gravity and distributed relative to the component's mass distribution and shall be determined in accordance with Eq. 13.3-1:

$$F_p = \frac{0.4 a_p S_{DS} W_p}{\left(R_p / I_p\right)}\left(1 + 2\frac{z}{h}\right) \qquad (13.3-1)$$

F_p is not required to be taken as greater than

$$(F_P)_{\max} \le 1.6 S_{DS} I_P W_p \quad \Leftarrow \max \qquad (13.3-2)$$

and F_p shall not be taken as less than

$$(F_P)_{\min} \ge 0.3 S_{DS} I_P W_p \quad \Leftarrow \min \qquad (13.3-3)$$

The following sections of ASCE7-05 list the detailed requirements for the Mechanical and Electrical Nonstructural Components. ONLY PART OF THE DEATILED REQUIREMENTS WAS SELECTED AND PRESENTED BELOW:

§13.6.3 Mechanical Components. HVAC ductwork shall meet the requirements of Section 13.6.7. Piping systems shall meet the requirements of Section 13.6.8. Boilers and vessels shall meet the requirements of Section 13.6.9. Elevators shall meet the requirements of Section 13.6.10. All other mechanical components shall meet the requirements of Section 13.6.11. Mechanical components with I_p greater than 1.0 shall be designed for the seismic forces and relative displacements defined in Sections 13.3.1 and 13.3.2 and shall satisfy the following additional requirements:

1. Provision shall be made to eliminate seismic impact for components vulnerable to impact, for components constructed of non ductile materials, and in cases where material ductility will be reduced due to service conditions (e.g., low temperature applications).
2. The possibility of loads imposed on components by attached utility or service lines, due to differential movement of support points on separate structures, shall be evaluated.
3. Where piping or HVAC ductwork components are attached to structures that could displace relative to one another and for isolated structures where such components cross the isolation interface, the components shall be designed to accommodate the seismic relative displacements defined in Section 13.3.2.

§13.6.4 Electrical Components. Electrical components with I_p greater than 1.0 shall be designed for the seismic forces and relative displacements defined in Sections 13.3.1 and 13.3.2 and shall satisfy the following additional requirements:

1. Provision shall be made to eliminate seismic impact between components.
2. Loads imposed on the components by attached utility or service lines that are attached to separate structures shall be evaluated.
3. Batteries on racks shall have wrap-around restraints to en-sure that the batteries will not fall from the rack. Spacers shall be used between restraints and cells to prevent damage to cases. Racks shall be evaluated for sufficient lateral load capacity.
4. Internal coils of dry type transformers shall be positively attached to their supporting substructure within the trans-former enclosure.
5. Electrical control panels, computer equipment, and other items with slide-out components shall have a latching mechanism to hold the components in place.
6. Electrical cabinet design shall comply with the applicable National Electrical Manufacturers Association (NEMA) standards. Cutouts in the lower shear panel that have not been made by the manufacturer and reduce significantly the strength of the cabinet shall be specifically evaluated.
7. The attachments for additional external items weighing more than 100 lb (445 N) shall be specifically evaluated if not provided by the manufacturer.
8. Where conduit, cable trays, or similar electrical distribution components are attached to structures that could displace relative to one another and for isolated structures where such components cross the isolation interface, the components shall be designed to accommodate the seismic relative displacements defined in Section 13.3.2.

§13.6.5 Component Supports. Mechanical and electrical component supports (including those with I_p = 1.0) and the means by which they are attached to the component shall be designed for the forces and displacements determined in Sections 13.3.1 and 13.3.2. Such supports include structural members, braces, frames, skirts, legs, saddles, pedestals, cables, guys, stays, snubbers, and tethers, as well as elements forged or cast as a part of the mechanical or electrical component.

§13.6.6 Utility and Service Lines. At the interface of adjacent structures or portions of the same structure that may move independently, utility lines shall be provided with adequate flexibility to accommodate the anticipated differential movement between the portions that move independently. Differential displacement calculations shall be determined in accordance with Section 13.3.2.

The possible interruption of utility service shall be considered in relation to designated seismic systems in Occupancy Category IV as defined in Table 1-1. Specified attention shall be given to the vulnerability of underground utilities and utility interfaces between the structure and the ground where Site Class E or F soil is present, and where the seismic coefficient S_{DS} at the under-ground utility or at the base of the structure is equal to or greater than 0.33.

§13.6.7 HVAC Ductwork. Seismic supports are not required for HVAC ductwork with I_p = 1.0 if either of the following conditions is met for the full length of each duct run:

a. HVAC ducts are suspended from hangers 12 in. (305 mm) or less in length. The hangers shall be detailed to avoid significant bending of the hangers and their attachments.
b. HVAC ducts have a cross-sectional area of less than 6 ft^2 (0.557 m^2).

HVAC duct systems fabricated and installed in accordance with standards approved by the authority having jurisdiction shall be deemed to meet the lateral bracing requirements of this section..

§13.6.8 Piping Systems. Piping systems shall satisfy the requirements of this section except that elevator system piping shall satisfy the requirements of Section 13.6.10.

Except for piping designed and constructed in accordance with NFPA 13, seismic supports shall not be required for other piping systems where one of the following conditions is met:

1. Piping is supported by rod hangers; hangers in the pipe run are 12 in. (305 mm) or less in length from the top of the pipe to the supporting structure; hangers are detailed to avoid bending of the hangers and their attachments; and provisions are made for piping to accommodate expected deflections.

2. High-deformability piping is used; provisions are made to avoid impact with larger piping or mechanical components or to protect the piping in the event of such impact; and the following size requirements are satisfied :

a. For Seismic Design Categories D, E, or F where I_p is greater than 1.0, the nominal pipe size shall be 1 in. (25 mm) or less.
b. For Seismic Design Category C, where Ip is greater than 1.0, the nominal pipe size shall be 2 in. (51 mm) or less.
c. For Seismic Design Categories D, E, or F where I_p is equal to 1.0, the nominal pipe size shall be 3 in. (76 mm) or less.

§13.6.9 Boilers and Pressure Vessels. Boilers or pressure vessels designed in accordance with ASME BPVC shall be deemed to meet the force, displacement, and other requirements of this section. In lieu of the specified force and displacement requirements provided in the ASME BPVC, the force and displacement requirements of Sections 13.3.1 and 13.3.2 shall be used. Other boilers and pressure vessels designated as having an I_p = 1.5, but not constructed in accordance with the requirements of ASME BPVC shall comply with the requirements of Section 13.6.11.

§13.6.10 Elevator and Escalator Design Requirements. Elevators and escalators designed in accordance with the seismic requirements of ASME A17.1 shall be deemed to meet the seismic force requirements of this section, except as modified in the following text.

§13.6.11 Other Mechanical and Electrical Components.

Mechanical and electrical components, including distribution systems, not designed and constructed in accordance with the reference documents in Chapter 23 shall meet the following:

1. Components, their supports and attachments shall comply with the requirements of Sections 13.4, 13.6.3, 13.6.4, and 13.6.5.
2. Where mechanical components contain a sufficient quantity of hazardous material to pose a danger if released, and for boilers and pressure vessels not designed in accordance with ASME BPVC, the design strength for seismic loads in combination with other service loads and appropriate environmental effects shall be based on the following material properties.

   a. For mechanical components constructed with ductile materials (e.g., steel, aluminum, or copper), <u>90 percent of the minimum specified yield strength.</u>

   b. For threaded connections in components constructed with ductile materials, <u>70 percent of the minimum specified yield strength.</u>

   c. For mechanical components constructed with nonductile materials (e.g., plastic, cast iron, or ceramics), <u>10 percent of the material minimum specified tensile strength</u>.

   d. For threaded connections in piping constructed with non-ductile materials, <u>8 percent of the material minimum specified tensile strength.</u>

5.9 NONSTRUCTURAL COMPONENT ANCHORAGE

Components and their supports shall be attached (or anchored) to the structure in accordance with the requirements of this section and the attachment shall satisfy the requirements for the parent material as set forth elsewhere in this standard.

Component attachments shall be bolted, welded, or otherwise positively fastened without consideration of frictional resistance produced by the effects of gravity. A continuous load path of sufficient strength and stiffness between the component and the supporting structure shall be provided. Local elements of the structure including connections shall be designed and constructed for the component forces where they control the design of the elements or their connections. The component forces shall be those determined in Section 13.3.1, except that modifications to F_p and R_p due to anchorage conditions need not be considered. The design documents shall include sufficient information relating to the attachments to verify compliance with the requirements of this section.

§13.4.2 Anchors in Concrete or Masonry. Anchors embedded in concrete or masonry shall be proportioned to carry the least of the following:

a. 1.3 times the force in the component and its supports due to the prescribed forces.
b. The maximum force that can be transferred to the anchor by the component and its supports.

The value of R_p used in Section 13.3.1 to determine the forces in the connected part shall not exceed 1.5 unless

a. The component anchorage is designed to be governed by the strength of a ductile steel element.
b. The design of post-installed anchors in concrete used for the component anchorage is prequalified for seismic applications in accordance with ACI 355.2.
c. The anchor is designed in accordance with Section 14.2.2.14.

§13.4.3 Installation Conditions. Determination of forces in attachments shall take into account the expected conditions of installation including eccentricities and prying effects.

§13.4.4 Multiple Attachments. Determination of force distribution of multiple attachments at one location shall take into account the stiffness and ductility of the component, component supports, attachments, and structure and the ability to redistribute loads to other attachments in the group. Designs of anchorage in concrete in accordance with Appendix D of ACI 318 shall be considered to satisfy this requirement.

5-10 SAMPLE PROBLEMS

Sample Problem 5-1: Exemption of Some of Nonstructural Components

Given: Which of the following nonstructural components are exempt from the requirements of Chapter 13 of ASCE 7- 05:

I. Mechanical and electrical components in Seismic Design Category(SDC) "C" with $I_P = 1.0$
II. Architectural, mechanical and electrical components in Seismic Design Category "A"
III. Mechanical and electrical components in Seismic Design Category(SDC) "B"
IV. None of the above

Answer: A) I , II B) II & III C) I, II & III D) IV

Solution:

There are five exemptions are listed below. Compare the given choices with these exemptions carefully and match one or more of the m with the possible answers. Note either and both in exemptions #4 & #5.

§13.1.4 Exemptions. The following nonstructural components are exempt from the requirements of this section:

1. Architectural components in Seismic Design Category B other than parapet supported by bearing walls or shear walls provided that the component importance factor, I_p, is equal to 1.0.
2. Mechanical and electrical components in Seismic Design Category B.
3. Mechanical and electrical components in Seismic Design Category C provided that the component importance factor, I_p, is equal to 1.0.
4. Mechanical and electrical components in Seismic Design Categories D, E, and F where the component importance factor, I_p, is equal to 1.0 and either:
   a. Flexible connections between the components and associated ductwork, piping, and conduit are provided.
   b. Components are mounted at 4 ft or less above a floor level and weigh 400 lb or less.
5. Mechanical and electrical components in Seismic Design Categories D, E, and F where the component importance factor, I_p, is equal to 1.0 and both:
   a. Flexible connections between the components and associated ductwork, piping, and conduit are provided, and
   b. The components weigh 20 lb (89 N) or less or, for distribution systems, weighing 5 lb/ft (73 N/m) or less.

NOTE: SDC "A" includes Occupancy Categories I, II, III & IV structures located in least vulnerable seismic area where $S_{DS} < 0.167$ & $S_{D1} < 0.067$ where a minor ground movement is expected. Therefore, if a nonstructural component in SDC "B" is exempt, SDC "A" will be included too.

Answer: C ⇐

Sample Problem 5-2: The Function of an Anchor Strap

Given:
Anchor straps at the boundaries of floor and roof diaphragms are required for concrete or masonry wall buildings in order to:

I) Transfer diaphragm shear to the shear walls
II) Ensure positive anchorage of the diaphragm to the walls
III) Transfer dead and live loads to the wall
IV) Reduce shrinkage of wood joists

Answer: A) I B) II C) II & III D) III & IV

Solution:
Per **§ 13.4 of ASCE 7- 05** "Components and their supports shall be attached (or anchored) to the structure in accordance with the requirements of this section and the attachment shall satisfy the requirements for the parent material as set forth elsewhere in this standard."

Component attachments shall be bolted, welded, or otherwise positively fastened without consideration of frictional resistance produced by the effects of gravity. A continuous load path of sufficient strength and stiffness between the component and the supporting structure shall be provided."

The positively fastened of the anchor strap will prevent the roof not to be pulled out in case of an earthquake event.

Answer: B ⇐

Sample Problem 5-3: Weight of Suspended Ceilings

Given: A suspended ceiling in a commercial building. Per ASCE 7-05, the weight of the ceiling, W_p, shall include the ceiling grid and panels; light fixtures if attached to, clipped to, or laterally supported by the ceiling grid; and other components that are laterally supported by the ceiling. However, W_p shall be taken as not less than :

A) 4 psf
B) 8 psf
C) 12 psf
D) 16 psf

Solution:
§13.5.6 Suspended Ceilings.
§13.5.6.1 Seismic Forces. The weight of the ceiling, W_p, shall include the ceiling grid and panels; light fixtures if attached to, clipped to, or laterally supported by the ceiling grid; and other components that are laterally supported by the ceiling. W_p shall be taken as not less than 4 psf (19 N/m^2).

Answer: A ⇐

Sample Problem 5-4: Design Force in the Anchors Embedded in Concrete or Masonry

Given:
Anchors embedded in concrete or masonry walls shall be proportioned to carry the least of:

I) 1.3 times the force in the component and its supports due to the prescribed forces.
II) 1.5 times the force in the component and its supports due to the prescribed forces.
III) The maximum force that can be transferred to the anchor by the component and its supports.
IV) All of the above

Answer: A) I B) I &II C) I & III D) IV

Solution:

§13.4.2 Anchors in Concrete or Masonry. Anchors embedded in concrete or masonry shall be proportioned to carry the least of the following:

a. 1.3 times the force in the component and its supports due to the prescribed forces.
b. The maximum force that can be transferred to the anchor by the component and its supports.

Answer: C⇐

Sample Problem 5-5: Weight of Equipment Fastened to An Access Floor

Given: The access floor of a building shall include the weight of the floor and what percentage of the equipment fastened to it:

A) 25%
B) 50%
C) 75%
D) 100%

Solution:

§13.5.7 Access Floors.
§13.5.7.1 General. The weight of the access floor, W_p, shall include the weight of the floor system, 100 percent of the weight of all equipment fastened to the floor, and 25 percent of the weight of all equipment supported by, but not fastened to the floor. The seismic force, F_p, shall be transmitted from the top surface of the access floor to the supporting structure.

Answer: D⇐

Sample Problem 5-6: Weight of Equipment NOT Fastened to An Access Floor

Given: The access floor of a building shall include the weight of the floor and what percentage of the equipment supported by the floor and NOT fastened to it:

A) 25%
B) 50%
C) 75%
D) 100%

Solution:

§13.5.7 Access Floors.

§13.5.7.1 General. The weight of the access floor, W_p, shall include the weight of the floor system, 100 percent of the weight of all equipment fastened to the floor, and 25 percent of the weight of all equipment supported by, but not fastened to the floor. The seismic force, F_p, shall be transmitted from the top surface of the access floor to the supporting structure.

Answer: A ⇐

Sample Problem 5-7: Response Modification Factor for Elements and Components (R_p)

Given: A penthouse NOT framed by an extension of the building frame.

Find: The response modification factor (R_p) used should be

Answers: A) 1.0 B) 1.5 C) 2.5 D) 3.5

Solution: ASCE § 13.5, Table 13.5-1, **Penthouses (except where framed by an extension of the building frame) -** $R_p = 3.5$

Answer: D ⇐

Sample Problem 5-8: Anchor Spacing

Given: A concrete/masonry wall anchored to the roof of a building

Find: The wall is required to be designed to resist bending moment between the anchors where the anchor spacing exceeds:

A) 2 feet
B) 4 feet
C) 6 feet
D) 4 times the thickness of the wall

Solution:

§12.11.2 Anchorage of Concrete or Masonry Structural Walls.
Structural walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 ft (1,219 mm).

Answer: D ⇐

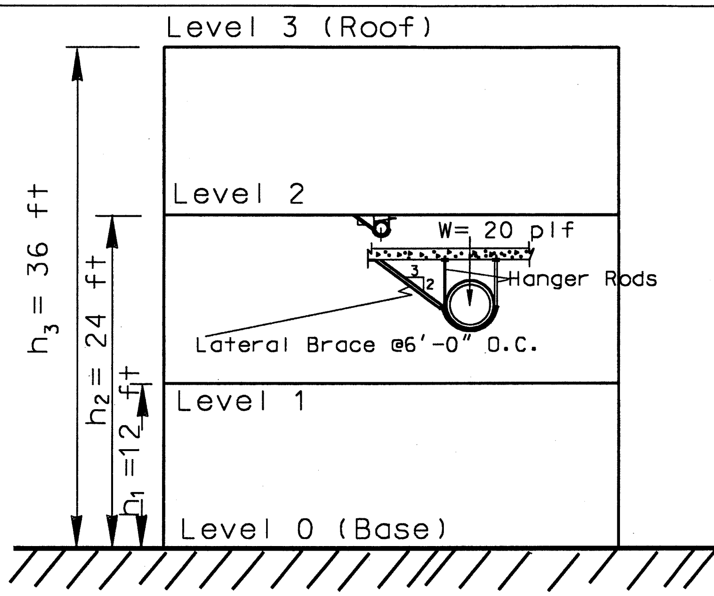
Sample Problem 5-9: Nonstructural Component At the 2nd Level (not on the Roof or at the Base)

Given: A utility pipe is attached to the ceiling of a hospital as shown below. An enlarged detail shows the pipe and the lateral brace which is located at 8'-0" o.c. to prevent the pipe from swinging horizontally during an earthquake. Consider $S_{DS} = 1.03g$, $I_p = 1.5$, $a_p = 2.5$ & $R_p = 6.0$

Find: The seismic design axial force in the brace is most nearly.

Answer: A) 89 lb B) 231 lb C) 278 lb D) 475 lb

Solution:



$z/h = (24/36) = 0.67$ (pipe at level 2).

$$F_p = \frac{0.4 a_p S_{DS} W_p}{\left(R_p / I_p\right)}\left(1 + 2\frac{z}{h}\right) \qquad (13.3-1)$$

$$= \frac{0.4 \times 2.5 \times 1.03 \times (20 \times 8)}{(2.5/1.5)}\{1 + 2(0.67)\}$$

$$= 231.38\, lb/brace \Leftarrow Governs$$

The force in the brace

$$= 231.38\frac{\sqrt{13}}{3} = 278.08\ lb/brace \quad \Leftarrow$$

Check the minimum and maximum values of F_p

$$(F_P)_{\max} \le 1.6 S_{DS} I_P W_p = 1.6(1.03)(1.5)(20 \times 8) = 395.52\ lb/brace \qquad (13.3-2)$$

$$(F_P)_{\min} \ge 0.3 S_{DS} I_P W_p = 0.3(1.03)(1.5)(20 \times 8) = 74.16\, lb/brace \qquad (13.3-3)$$

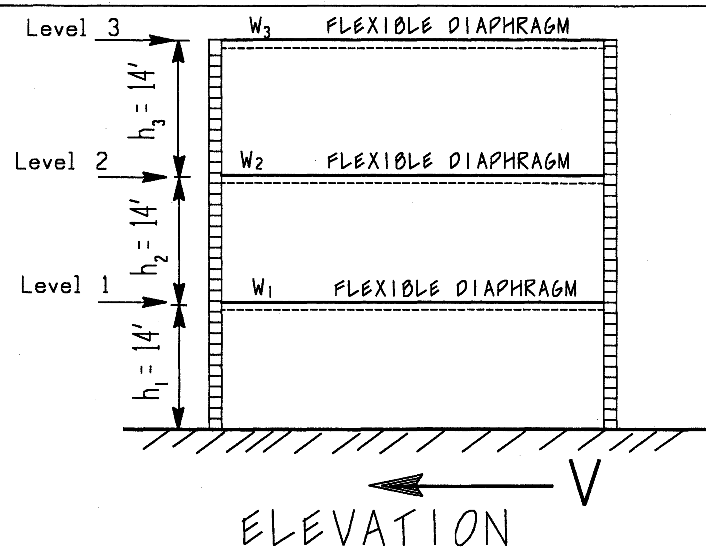
Notes:

- *Maximum value typically governs when the nonstructural component is located at or near the roof.*
- *Minimum value needs to be verified when the nonstructural component is located at or near the base.*
- *Because the pipe is attached to the 2nd level, neither the minimum nor the maximum governs.*

Answer: C⇐

Sample Problem 5-10: Anchorage Force for Walls with Flexible Diaphragms

Given: A three story commercial building consists of masonry exterior walls and flexible diaphragm as shown below. The building assigned a Seismic Design Category (SDC) C and the masonry walls has a unit weight of 65 psf. The seismic design response acceleration at short periods $S_{DS} = 1.20g$



Find: The anchorage force at level 2 is most nearly:

Answer: A) 91 plf B) 280 plf C) 480 plf D) 874 plf

Solution:

Per section 12.11.2 shown below, the anchorage force is as follows:
$I = 1.0$ (Table 11.5-1)
W_p = the weight of the wall tributary to the anchor
$= W_{\text{wall}}(h_2/2 + h_3/2) = 65(14'/2 + 14'/2) = 910$ plf

§12.11.2 Anchorage of Concrete or Masonry Structural Walls.
The anchorage of concrete or masonry structural walls to supporting construction shall provide a direct connection capable of resisting the greater of the following:

a. The force set forth in Section 12.11.1.
b. A force of $400 S_{DS} I$ lb/ linear ft ($5.84 S_{DS} I$ kN/m) of wall
c. 280 lb/linear ft (4.09 kN/m) of wall

Structural walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 ft (1,219 mm).

§12.11.2.1 Anchorage of Concrete or Masonry Structural Walls to Flexible Diaphragms.
In addition to the requirements set forth in Section 12.11.2, anchorage of concrete or masonry structural walls to flexible diaphragms in structures assigned to Seismic Design Category C, D, E, or F shall have the strength to develop the out-of-plane force given by Eq. 12.11-1:

$$F_p = 0.8 S_{DS} I W_p \qquad (12.11\text{-}1)$$

where
F_p = the design force in the individual anchors
S_{DS} = the design spectral response acceleration parameter at short periods per Section 11.4.4

I = the occupancy importance factor per Section 11.5.1

W_p = the weight of the wall tributary to the anchor

$$F_p \geq 0.10\, W_P = 0.10 \times 910 = 91\, plf$$

$$F_p \geq 400\, S_{DS}\, I = 400 \times 1.2 \times 1.0 = 480\, plf$$

$$F_p \geq 0.8\, S_{DS}\, I\, W_P = 0.8 \times 1.2 \times 1.0 \times 910 = 874\, plf \quad \Leftarrow governs$$

$$F_p \geq 280\, plf \Leftarrow \min.\, per\ item\ c\ above$$

Answer: D⇐

Sample Problem 5-11 : Nonstructural Components on the Roof

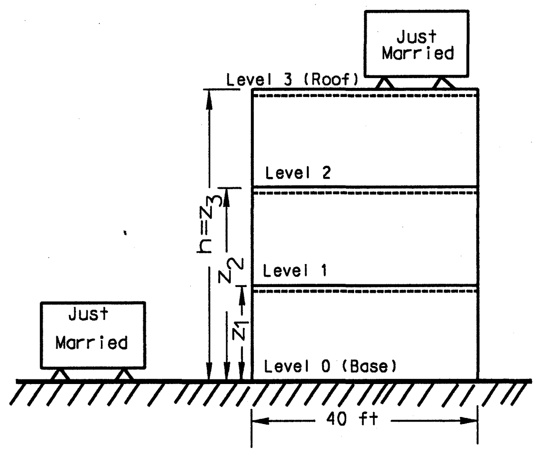
Given: A sign weighs 2000 lbs on the roof of a residential building as shown. The design spectral acceleration $S_{DS} = 1.20$g.

Find: The seismic design force, F_p, is most nearly

Answer: A) 3.84 kips B) 2.88 kips C) 0.96 kips D) 0.72 kips

Solution:

Chapter 13 of the ASCE 7-05 deals with the "seismic design requirements for nonstructural components". A billboard on the roof of a commercial building is considered " an architectural component" per Table 13.5-1. If the billboard is located outside the building and is self supported (as shown next to the building), then it is considered a nonbuilding structure and it should be designed per Chapter 15 of the ASCE7- 05.



Calculating the minimum and maximum values as follows:

$$(F_P)_{\max} \leq 1.6 S_{DS} I_P W_p = 1.6(1.20)(1.0)(2) = 3.84\ kips \qquad (13.3-2)$$

Maximum value typically governs when the component is located at or near the roof.

$$(F_P)_{\min} \geq 0.3 S_{DS} I_P W_p = 0.3(1.20)(1.0)(2) = 0.72\ kpis \qquad (13.3-3)$$

Minimum value needs to be verified when the nonstructural component is located at or near the base.

I =1.0 (Table 11.5-1), a_p= 2.5, R_p= 2.5 - Table 13.5-1(signs & billboards) & z/h = 1.0 – the sign is on the roof.

$$F_p = \frac{0.4 a_p S_{DS} W_p}{\left(R_p / I_p\right)}\left(1 + 2\frac{z}{h}\right) = \frac{0.4 \times 2.5 \times 1.20 \times 2}{(2.5/1.0)}\{1 + 2(1.0)\} = 2.88\ kips \Leftarrow \qquad (13.3-1)$$

Answer: C⇐

Sample Problem 5-12: Anchorage of Masonry/Concrete Shear Walls to Wood Diaphragms

Given: To anchor a concrete or masonry shear wall to a wood diaphragm assigned to Seismic Design Category "C" through "F", which of the following techniques is permitted per ASCE 7-05.

I) Cross-grain bending
II) Toe nails
III) Cross-grain tension
IV) None of the above

Answer: A) I B) I &II C) I , II & III D) IV

Solution:

§12.11.2.2.3 Wood Diaphragms. In wood diaphragms, the continuous ties shall be in addition to the diaphragm sheathing. Anchorage shall not be accomplished by use of toe nails or nails subject to withdrawal nor shall wood ledgers or framing be used in cross-grain bending or cross-grain tension.

Answer: D⇐

Sample Problem 5-13: Nonstructural Components Classification

Given: A billboard mounted on top of a building is considered:

A) Building structure
B) Nonbuilding structure
C) An architectural component
D) None of the above

Solution:
Table 13.5-1, (ASCE 7-05) indicates that a billboard on supported by a building structure is considered an architectural component

Note: A billboard supported on the ground (self supporting) is considered a nonbuilding structure and § 15.4 0f ASCE 7-05 will apply. See table 15.4-2 of ASCE 7-05.

Answer: C⇐

Sample Problem 5-14: Types of Nonstructural Components

Given: A heat exchanger attached to a building structure is considered

A) An architectural component
B) A mechanical component
C) Nonstructural component
D) Both "B" & "C"

Solution:
Table 13.6-1 of the ASCE7-07 indicates that the heat exchanger is considered a mechanical nonstructural component.

Answer: D ⇐

Sample Problem 5-15: Rigid versus Flexible Components

Given: A rigid component is defined as a component, including its attachments, having a period of :

A) ≤ 0.06 second
B) > 0.06 second
C) ≤ 0.70 second
D) < 0.70 second

Solution:
§11.2 (page 110) ASCE 7-05
COMPONENT: A part or element of an architectural, electrical, mechanical, or structural system.
Component, Flexible: Component, including its attachments, having a fundamental period greater than 0.06 s.
Component, Rigid: Component, including its attachments, having a fundamental period less than or equal to 0.06 s.

Answer: A ⇐

Sample Problem 5-16: Types of Nonstructural Components

Given: A chimney of a single-family wood framed house is considered:

A) Equipment
B) A structure
C) Nonstructural component
D) Nonbuilding structure

Solution:

Table 13.5-1, (ASCE7-05) indicates that a chimney in a building is considered a nonstructural component (an architectural component)

Note: A chimney supported on the ground (self supporting) is considered a nonbuilding structure and §15.4 of the ASCE 7-05 will apply.

Answer: C ⇐

Chapter 6

Seismic Design Requirements for Nonbuilding Structures
Chapter 15 ASCE 7-05

Topics to be covered

- Definition of Nonbuilding Structures
- Exemption for Some of Nonbuilding Structures
- Classifications (Types) of Nonbuilding Structures
- Structural Analysis Procedures for Nonbuilding Structures
- T, I, W, R, Ω_0 and C_d For Nonbuilding Structures
- Rigid vs. Flexible Nonbuilding Structures
- Tanks and Vessels
- Nonbuilding Structures **Similar** to Buildings
- Nonbuilding Structures **Not Similar** to Buildings

17 Sample Problems

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Chapter 6- Seismic Design Requirements For Nonbuilding Structures

6.1 INRODUCTION

Nonbuilding structures (self- supporting and supported by other structures) as defined in §15.1.1 shall be designed and detailed to provide sufficient stiffness, strength and ductility resist the effects of minimum seismic lateral forces given in Chapter 15.

§ 15.1.1 ***Nonbuilding Structures.*** *Nonbuilding structures include all self-supporting structures that carry gravity loads and that may be required to resist the effects of earthquake, with the exception of building structures specifically excluded in Section 11.1.2, and other nonbuilding structures where specific seismic provisions have yet to be developed, and therefore, are not set forth in Chapter 15. Nonbuilding structures supported by the earth or supported by other structures shall be designed and detailed to resist the minimum lateral forces specified in this section. Design shall conform to the applicable requirements of other sections as modified by this section. Foundation design shall comply with the requirements of Sections 12.1.5, 12.13, and Chapter 14.*

6.2 EXCEMTION FOR SOME OF NONBUILDING STRUCTURES

The following nonbuilding structures are except from the requirements of the ASCE7-05 as stated in §11.1.2 (item 4)

"§ 4. Structures that require special consideration of their response characteristics and environment that are not addressed in Chapter 15 and for which other regulations provide seismic criteria, such as vehicular bridges, electrical transmission towers, hydraulic structures, buried utility lines and their appurtenances, and nuclear reactors."

Based on the above statement of ASCE7-05, the following are the exempt nonbuilding structures :

a. vehicular bridges
b. electrical transmission towers
c. hydraulic structures (dams, spillways …etc)
d. buried utility lines and their appurtenances
e. nuclear reactors
f. railroad bridges
g. tunnels

6.3 CLASSIFICATIONS (TYPES) OF NONBUILDING STRUCTURES

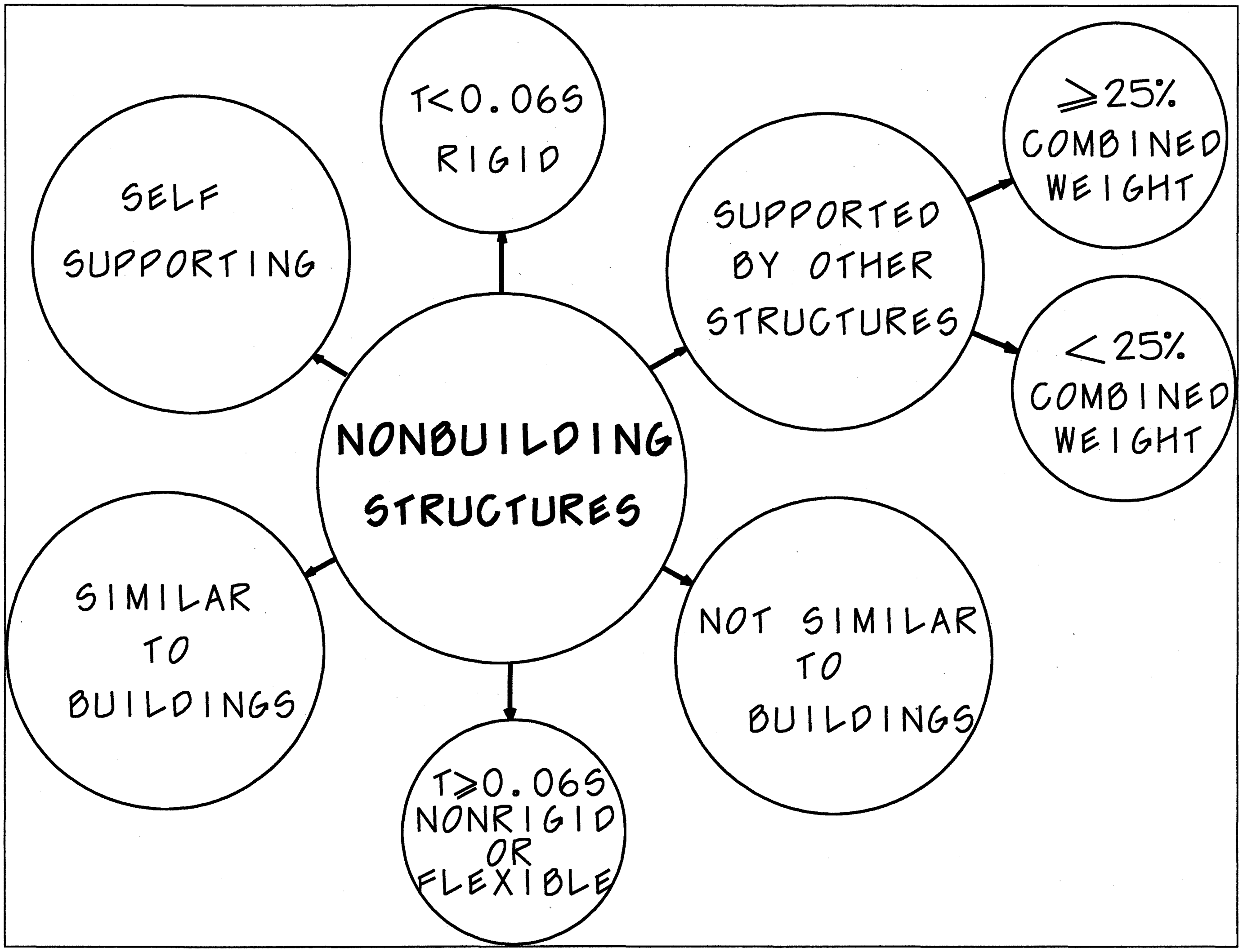


Figure 6-1 Classifications of Nonbuilding Structures

Rigid Nonbuilding Structure: Nonbuilding structures that have a fundamental period, T, less than 0.06 s, including their anchorages (e.g. short storage bin with high stiffness)

Flexible Nonbuilding Structure: Nonbuilding structures that have a fundamental period, T, $\geq$0.06 s, including their anchorages (e.g. tall storage bin with low stiffness)

Nonbuilding Structure Similar to Buildings: Those are designed, constructed, have lateral and vertical force resting system and finally respond to seismic ground motion as buildings.

Nonbuilding Structure Not Similar to Buildings: They do not have lateral and vertical force resting system and do not respond to ground motion as buildings.

Self Supporting Nonbuilding Structures: Those are supported directly by earth.

Nonbuilding Structures Supported by Other Structures: Those are supported by other structures and they may weight more or less than 25% of the combined weight.

6.4 STRUCTURAL ANALYSIS PROCEDURES FOR NONBUILDING STR.

The selection of the structural analysis depends on the type of the nonbuilding structure. Table 6.1 includes a summary of the procedures per ASCE 7-05.

§15.1.3 Structural Analysis Procedure Selection. *Structural analysis procedures for nonbuilding structures that are similar to buildings shall be selected in accordance with Section 12.6. Nonbuilding structures that are not similar to buildings shall be designed using either the equivalent lateral force procedure in accordance with Section 12.8, the modal analysis procedure in accordance with Section 12.9, the linear response history analysis procedure in accordance with Section 16.1, the nonlinear response history analysis procedure in accordance with Section 16.2, or the procedure prescribed in the specific reference document.*

§ 15.3.3 Architectural, Mechanical, and Electrical Components.
Architectural, mechanical, and electrical components supported by nonbuilding structures shall be designed in accordance with Chapter 13 of this standard.

6.5 DESIGN PARAMTERS FOR NONBUILDING SRT.(*T, I, W, R*, Ω_0 & C_d)

A) Fundamental Period, *T*:

§15.4.4 Fundamental Period. The fundamental period of the nonbuilding structure shall be determined using the structural properties and deformation characteristics of the resisting elements in a properly substantiated analysis as indicated in Section 12.8.2. Alternatively, the fundamental period T is permitted to be computed from the following equation:

$$T = 2\pi \sqrt{\frac{\sum_{i=1}^{n} w_i \delta_i^2}{g \sum_{i=1}^{n} f_i \delta_i}} \qquad (15.4-6)$$

Where:

f_i = lateral force (in accordance with the principles of structural mechanics) at level i.
w_i = effective seismic weight of level i
δ_i = the elastic deflections, shall be calculated using the applied lateral forces, f_i.
g = acceleration due to gravity (32.2 ft/sec^2 or 386.4 in/sec^2)

Equation (15.4-6) will be reduced for a single degree of freedom (SDOF) system to:

$$T = 2\pi \sqrt{\frac{m}{k}} = 2\pi \sqrt{\frac{W}{k\,g}}$$

Note : This equation (for SDOF) is similar to Equation (13.6-1) for the component period.

Table 6-1 Structural Analysis Procedures For Nonbuilding Structures

Self – Supporting (Supported by Earth)		Supported by Other Structures	
Similar to Buildings	NOT Similar to Buildings	< 25% Combined Weight	≥ 25% Combined Weight
Structural analysis procedures for nonbuilding structures that are similar to buildings shall be selected in accordance with Section 12.6	Nonbuilding structures that are not similar to buildings shall be designed using either: 1. The equivalent lateral force procedure in accordance with Section 12.8, 2. the modal analysis procedure in accordance with Section 12.9, 3. the linear response history analysis procedure in accordance with Section 16.1, 4. the nonlinear response history analysis procedure in accordance with Section 16.2, 5. or the procedure prescribed in the specific reference document.	➢ The design seismic forces of the nonbuilding structure shall be determined in accordance with Chapter 13 where the values of R_p and a_p shall be determined in accordance to Section 13.1.5. ➢ The supporting structure shall be designed in accordance with the requirements of Chapter 12 or Section 15.5 as appropriate with the weight of the nonbuilding structure considered in the determination of the effective seismic weight, W. *(W_p included in W)*	For the condition where the weight of the nonbuilding structure is equal to or greater than 25 percent of the combined weight of the nonbuilding structure and supporting structure, an analysis combining the structural characteristics of both the non-building structure and the supporting structures shall be performed to determine the seismic design forces as described in items 1 & 2 of §15.3.2 of ASCE 7-05. (see below)

1. *Where the nonbuilding structure has rigid component dynamic characteristics (as defined in Section15.4.2),the nonbuilding structure shall be considered a rigid element with appropriate distribution of its effective seismic weight. The supporting structure shall be designed in accordance with the requirements of Chapter 12 or Section 15.5 as appropriate, and the R value of the combined system is permitted to be taken as the R value of the supporting structural system. The nonbuilding structure and attachments shall be designed for the forces using the procedures of Chapter 13 where the value of R_p shall be taken as equal to the R value of the nonbuilding structure as set forth in Table 15.4-2 and a_p shall be taken as 1.0.*

2. *Where the nonbuilding structure has nonrigid characteristics (as defined in Section 15.4.2), the nonbuilding structure and supporting structure shall be modeled together in a combined model with appropriate stiffness and effective seismic weight distributions. The combined structure shall be designed in accordance with Section 15.5 with the R value of the combined system taken as the lesser R value of the nonbuilding structure or the supporting structure. The nonbuilding structure and attachments shall be designed for the forces determined for the nonbuilding structure in the combined analysis.*

The following equations(the approximate fundamental period equations for building structures of §12.8 of ASCE 7-05) shall not be used for determining the period of a nonbuilding structure.

$$T_a = C_t h_n^x \tag{12.8-7}$$

$$T_a = 0.1\,N \tag{12.8-8}$$

$$T_a = \frac{0.0019}{\sqrt{C_w}} h_n \tag{12.8-9}$$

$$C_w = \frac{100}{A_B} \sum_{i=1}^{x} \left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{\left[1 + 0.83\left(\frac{h_i}{D_i}\right)^2\right]} \tag{12.8-10}$$

B) Importance Factor, *I*:

§15.4.1.1 Importance Factor. The importance factor, I, and occupancy category (O.C.) for nonbuilding structures are based on the relative hazard of the contents and the function. The value of I shall be the largest value determined by the following:

a. Applicable reference document listed in Chapter 23 (Seismic Design Reference Documents)
b. The largest value as selected from Table 11.5-1.

TABLE 11.5-1 IMPORTANCE FACTORS

Occupancy Category	I
I or II	1.0
III	1.25
IV	1.5

c. As specified elsewhere in Chapter 15.

C) Effective Seismic Weight, *W*

§15.4.3 Loads. The seismic effective weight W for nonbuilding structures shall include

i. All dead load as defined for structures in Section 12.7.2.

§12.7.2 Effective Seismic Weight. The effective seismic weight, W, of a structure shall include the total dead load and other loads listed below:

1. In areas used for storage, a minimum of 25 percent of the floor live load (floor live load in public garages and open parking structures need not be included).
2. Where provision for partitions is required by Section 4.2.2 in the floor load design, the actual partition weight or a minimum weight of 10 psf (0.48 kN/m2) of floor area, whichever is greater.

3. *Total operating weight of permanent equipment.*
4. *Where the flat roof snow load, P_f, exceeds 30 psf (1.44 kN/m2), 20 percent of the uniform design snow load, regardless of actual roof slope*

ii. For purposes of calculating design seismic forces in nonbuilding structures, W also shall include all normal operating contents for items such as tanks, vessels, bins, hoppers, and the contents of piping.

iii. W shall include snow and ice loads where these loads constitute 25 percent or more of W (i.e. $\geq 25\% W$) or where required by the building official based on local environmental characteristics.

D) Response Modification Factor, *R*, Overstrength Factor, Ω_0 Deflection Amplification Factor, C_d :

a. For nonbuilding structures *similar to buildings*, a system shall be selected from among the types indicated in Table 12.2-1 or Table 15.4-1 subject to the system limitations and height limits, based on the seismic design category indicated in the table. The appropriate values of R, Ω_0, and C_d indicated in Table 15.4-1 shall be used in determining the base shear, element design forces, and design story drift as indicated in this standard.

b. For nonbuilding structures *not similar to buildings*, a system shall be selected from among the types indicated in Table 15.4-2 subject to the system limitations and height limits, based on seismic design category indicated in the table. The appropriate values of R, Ω_0, and C_d indicated in Table 15.4-2 shall be used in determining the base shear, element design forces, and design story drift as indicated in this standard.

c. Where neither *Table 15.4-1 nor Table 15.4-2* contains an appropriate entry, applicable strength and other design criteria shall be obtained from a reference document that is applicable to the specific type of nonbuilding structure. Design and detailing requirements shall comply with the reference document.

6.6 RIGID vs. FLEXIBLE NONBUILDING STRUCTURES

According to Figure 6.1 nonbuilding structures are classified to *rigid or flexible* depending on the period. Nonbuliding structures having a fundamental period, T, *less than 0.06 s, including their anchorages, shall be classified and design as a rigid otherwise it will be flexible.*

§15.4.2 Rigid Nonbuilding Structures. Nonbuilding structures that have a fundamental period, T, less than 0.06 s, including their anchorages, shall be designed for the lateral force obtained from the following:

$$V = 0.30 S_{DS} W I \qquad (15.4\text{-}5)$$

Where:

V = the total design lateral seismic base shear force applied to a nonbuilding structure
S_{DS} = the site design response acceleration as determined from Section 11.4.4
W = nonbuilding structure operating weight
I = the importance factor determined in accordance with Section 15.4.1.1

The force shall be distributed with height in accordance with Section 12.8.3.

> ***§12.8.3 Vertical Distribution of Seismic Forces.*** *The lateral seismic force (F_x) (kip or kN) induced at any level shall be determined from the following equations:*
>
> $$F_x = C_{vx} V \qquad (12.8\text{-}11)$$
>
> *and*
>
> $$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad (12.8-12)$$
>
> *where*
>
> C_{vx} = *vertical distribution factor,*
>
> V = *total design lateral force or shear at the base of the structure (kip or kN)*
>
> w_i *and* w_x = *the portion of the total effective seismic weight of the structure (W) located or assigned to Level i or x*
>
> h_i *and* h_x = *the height (ft or m) from the base to Level i or x*
>
> k = *an exponent related to the structure period as follows:*
> *for structures having a period of 0.5 s or less, k = 1 for structures having a period of 2.5s or more, k = 2 for structures having a period between 0.5 and 2.5 s, k shall be 2 or shall be determined by linear interpolation between 1 and 2*

Table 6-2 Rigid and Flexible Nonbuilding Structures

Rigid Nonbuilding Structures	Flexible Nonbuilding Structures	
Fundamental period, $T < 0.06$ *second*	Fundamental period, $T \geq 0.06$ *second*	
$V = 0.30 S_{DS} WI$ *(Eq.15.4-5 ASCE7-05)*	**Similar to Buildings**	**Not Similar to Buildings**
	ELF for Buildings per §12.8 (6 Equations) with same minimum values.	ELF for Buildings per §12.8 (6 Equations) with **NEW** minimum values.

6.7 TANKS and VESSELS NONBUILDING STRUCTURES

Tanks and vessels that are self supported (not supported by other structures) shall be design to resist two forces:

1- The impulsive component due to effect of the fluid mass including tank weight.
2- The convective (sloshing) force component which is resulted from motion of the fluids above the freeboard. The freeboard as shown in Figure 6-2 is defined as the distance between the maximum permitted fluid level and to of the tanks. Providing adequate freeboard will prevent damage due to the convective (sloshing) force.

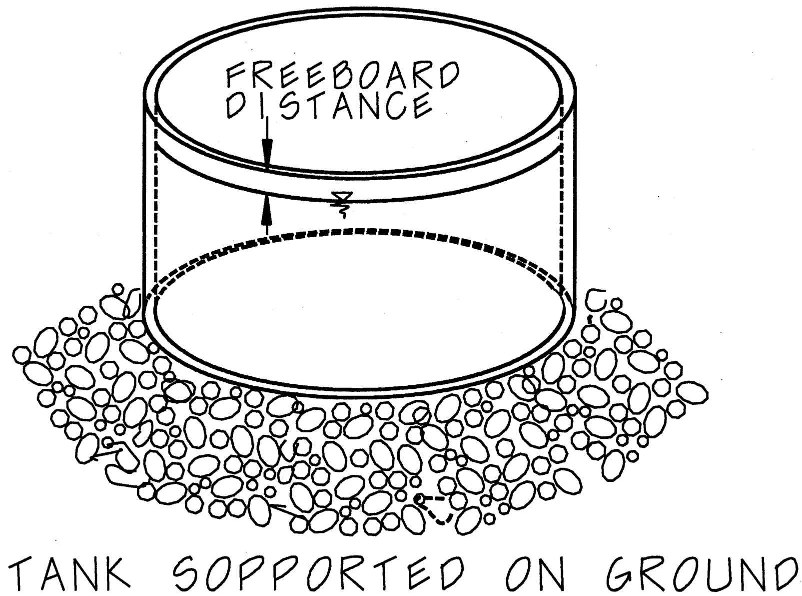


Figure 6-2 Tank and Freeboard Distance

The impulsive and convective components shall be combined by the direct sum or the square root of the sum of the squares (SRSS) method where the modal periods are separated.

§15.7.1 General. This section applies to all *tanks, vessels, bins, and silos, and similar containers storing liquids, gases, and granular solids supported at the base*.

Tanks and vessels covered herein include reinforced *concrete, prestressed concrete, steel, aluminum, and fiber-reinforced plastic materials*.

Tanks supported on elevated levels in buildings shall be designed in accordance with Section 15.3.(§ 5.3- Nonbuilding Structures Supported by Other Structures)

§15.7.2 Design Basis. Tanks and vessels storing liquids, gases, and granular solids shall be designed in accordance with this standard and shall be designed to meet the requirements of the applicable reference documents listed in Section 23. Resistance to seismic forces shall be determined from a substantiated analysis based on the applicable reference documents listed in Chapter23.

a. Damping for *the convective (sloshing) force component shall be taken as 0.5 percent*.

b. Impulsive and convective components shall be combined by the direct sum or the square root of the sum of the squares (SRSS) method where the modal periods are separated. If significant modal coupling may occur, the complete quadratic combination (CQC) method shall be used.

c. Vertical earthquake forces shall be considered in accordance with the applicable reference document. If the reference document permits the user the option of including or excluding the vertical earthquake force, to comply with this standard, it shall be included. For tanks and vessels not covered by a reference document, the forces due to the vertical acceleration shall be defined as follows:

(1) Hydrodynamic vertical and lateral forces in tank walls: The increase in hydrostatic pressures due to the vertical excitation of the contained liquid shall correspond to an effective increase in unit weight, γ_L, of the stored liquid equal to $0.2S_{DS}I\,\gamma_L$.

(2) Hydrodynamic hoop forces in cylindrical tank walls: In a cylindrical tank wall, the hoop force per unit height, N_h, at height y from the base, associated with the vertical excitation of the contained liquid, shall be computed in accordance with Eq. 15.7-1.

$$N_h = 0.2\,S_{DS}\gamma_L(H_L - y)\left(\frac{D_i}{2}\right) \qquad (15.7\text{-}1)$$

where

D_i = inside tank diameter

H_L = liquid height inside the tank

y = distance from base of the tank to height being investigated

γ_L = unit weight of stored liquid

(3) Vertical inertia forces in cylindrical and rectangular tank walls: Vertical inertia forces associated with the vertical acceleration of the structure itself shall be taken equal to $0.2S_{DS}IW$.

6.8 NONBUILDING STRUCTURES *SIMILAR* TO BUILDINGS

Definition:

NONBUILDING STRUCTURE SIMILAR TO A BUILDING: A nonbuilding structure that is *designed and constructed in a manner similar to buildings*, *will respond to strong ground motion in a fashion similar to buildings*, and have basic lateral and vertical seismic-force-resisting-system conforming to one of the types indicated in Tables 12.2-1 or 15.4-1.

Three criteria for nonbuilding structures similar to buildings:

1- *designed and constructed in a manner similar to buildings*
2- *will respond to strong ground motion in a fashion similar to buildings, and*
3- *have basic lateral and vertical seismic-force-resisting-system conforming to one of the types indicated in Tables 12.2-1 or 15.4-1.*

§ 15.5 of ASCE7-05 has a list subsection for nonbuilding structures where the specific requirements of analysis and design are explained. The list includes:

- **§ 15.5.2 Pipe Racks**
- **§ 15.5.3 Steel Storage Racks**
- **§ 15.5.4 Electrical Power Generating Facilities**
- **§ 15.5.5 Structural Towers for Tanks and Vessels**
- **§ 15.5.6 Piers and Wharves**

§ 15.5.1 General. Nonbuilding structures similar to buildings as defined in Section 11.2 shall be designed in accordance with this standard as modified by this section and the specific reference documents. This general category of nonbuilding structures shall be designed in accordance with the seismic requirements of this standard and the applicable portions of Section 15.4. The combination of load effects, E, shall be determined in accordance with Section 12.4.

Seismic Base Shear Determination , V:

Table 6-3 Seismic Base Shear (V) Flexible Nonbuilding Structures

Flexible Nonbuilding Structures	
Fundamental period, $T \geq 0.06$ *second*	
Similar to Buildings	**Not Similar to Buildings**
ELF for Buildings per §12.8 (6 Equations) with same minimum values.	ELF for Buildings per §12.8 (6 Equations) with **NEW** minimum values.

12.8.1 Seismic Base Shear. The seismic bases hear, V, in a given direction shall be determined in accordance with the following equation:

$$V = C_s W \qquad (12.8\text{-}1)$$

where

C_s = the seismic response coefficient determined in accordance with Section 12.8.1.1
W = the effective seismic weight per Section 12.7.2.

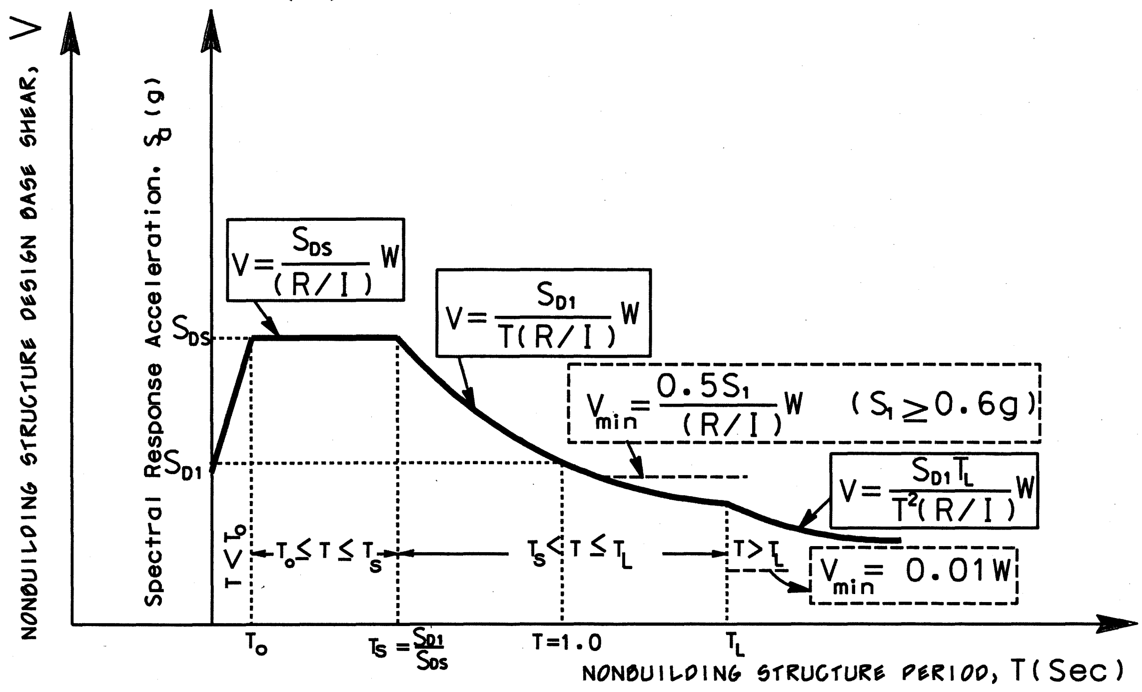
§12.8.1.1 Calculation of Seismic Response Coefficient. The seismic response coefficient, C_s, shall be determined in accordance with Eq. 12.8-2.

$$C_S = \frac{S_{DS}}{(R/I)} \qquad (12.8\text{-}2)$$

The value of C_s computed in accordance with Eq. 12.8-2 need not exceed the following:

$$C_S = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} \qquad for \quad T \le T_L \qquad (12.8\text{-}3)$$

$$C_S = \frac{S_{D1}\,T_L}{T^2\left(\frac{R}{I}\right)} \qquad for \quad T > T_L \qquad (12.8\text{-}4)$$



DESIGN RESPONSE SPECTRUM FOR NONBUILDING STRUCTURES SIMILAR TO BUILDINGS

Figure 6-3 Design Response Spectrum for Nonbuilding **Similar** to Buildings

C_s shall not be less than

$$(C_s)_{min} \ge 0.01 \qquad (12.8\text{-}5)$$

In addition, for structures located where S_1 is equal to or greater than 0.6g, C_s shall not be less than

$$(C_s)_{min} \ge \frac{0.5 S_1}{(R/I)} \quad (if\ S_1 \ge 0.6g) \qquad (12.8.6)$$

TABLE 15.4-1 SEISMIC COEFFICIENTS FOR NONBUILDING STRUCTURES SIMILAR TO BUILDINGS

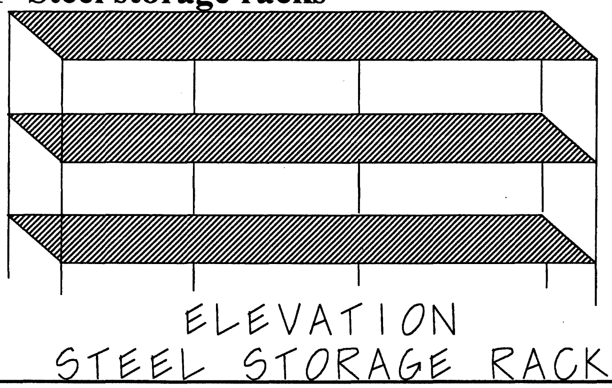
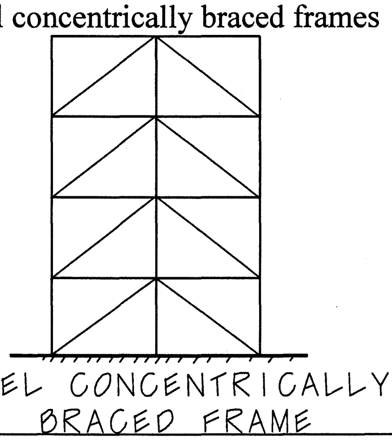
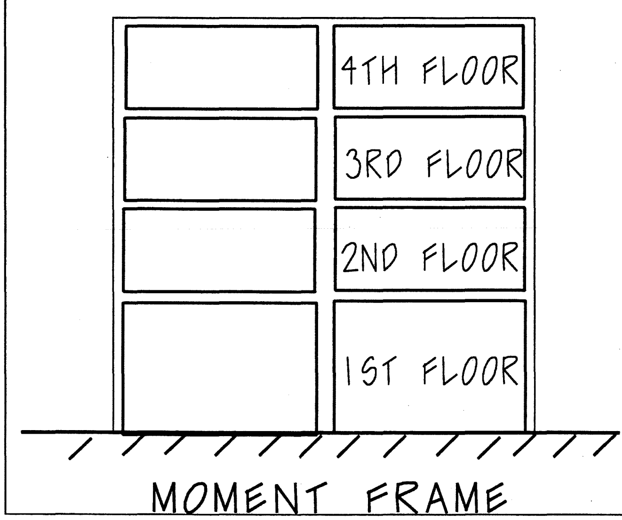
Nonbuilding Structure Type	R	Ω_0	C_d	STRUCTURAL SYSTEM AND HEIGHT LIMITS (ft)[a,e] A&B	C	D	E	F
1- Steel storage racks	4	2	3.5	NL	NL	NL	NL	NL

Nonbuilding Structure Type	R	Ω_0	C_d	STRUCTURAL SYSTEM AND HEIGHT LIMITS (ft)[a,e] A&B	C	D	E	F
2-Building frame systems:								
Special steel concentrically braced frames	6	2	5	NL	NL	160	160	100
Ordinary steel concentrically braced frame	3 ¼	2	3 ¼	NL	NL	35[b]	35[b]	NP[b]
With permitted height increase	2 ½	2	2 ½	NL	NL	160	160	100
With unlimited height	1.5	1	1.5	NL	NL	NL	NL	NL

Nonbuilding Structure Type	R	Ω_0	C_d	STRUCTURAL SYSTEM AND HEIGHT LIMITS (ft)[a,e] A&B	C	D	E	F
3-Moment-resisting frame systems:								
Special steel moment frames	8	3	5.5	NL	NL	NL	NL	NL
Special reinforced concrete moment frames	8	3	5.5	NL	NL	NL	NL	NL
Intermediate steel moment frames	4.5	3	4	NL	NL	35[c,d]	NP[c,d]	NP[c,d]
With permitted height increase	2.5	2	2.5	NL	NL	160	160	100
With unlimited height	1.5	1	1.5	NL	NL	NL	NL	NL
Intermediate reinforced concrete moment frames	5	3	4.5	NL	NL	NP	NP	NP
With permitted height increase	3	2	2.5	NL	NL	50	50	50
With unlimited height	0.8	1	1	NL	NL	NL	NL	NL
Ordinary moment frames of steel	3.5	3	3	NL	NL	NP[c,d]	NP[c,d]	NP[c,d]
With permitted height increase	2.5	2	2.5	NL	NL	100	100	NP[c,d]
With unlimited height	1	1	1	NL	NL	NL	NL	NL
Ordinary reinforced concrete moment frames	3	3	2.5	NL	NP	NP	NP	NP
With permitted height increase	0.8	1	1	NL	NL	50	50	50

[a]NL = no limit and NP = not permitted. Height shall be measured from the base.
[b]Steel ordinary braced frames are permitted in pipe racks up to 65 ft (20 m).
[c]Steel ordinary moment frames and intermediate moment frames are permitted in pipe racks up to a height of 65 ft (20 m) where the moment joints of field connections are constructed of bolted end plates.
[d]Steel ordinary moment frames and intermediate moment frames are permitted in pipe racks up to a height of 35 ft (11 m).
[e]For the purpose of height limit determination, the height of the structure shall be taken as the height to the top of the structural frame making up the primary seismic-force resisting system.

Table 6-4 Examples of Nonbuilding Structures **Similar to** Buildings

Nonbuilding Structure Type	R	Ω_0	C_d	STRUCTURAL SYSTEM AND HEIGHT LIMITS (ft)[a,e]				
				A&B	C	D	E	F
1- Steel storage racks 	4	2	3.5	NL	NL	NL	NL	NL
2-Building frame systems:								
Special steel concentrically braced frames 	6	2	5	NL	NL	160	160	100
3-Moment-resisting frame systems:								
Special steel moment frames **AND** Special reinforced concrete moment frames 	8	3	5.5	NL	NL	NL	NL	NL

6.9 NONBUILDING STRUCTURES *NOT SIMILAR* TO BUILDINGS

Definition:

Nonbuilding structures that **do not have *lateral*** and ***vertical seismic force-resisting systems*** that are similar to buildings shall be designed in accordance with this standard as modified by this section and the speific reference documents. Loads and load distributions shall not be less demanding than those determined in this standard. The combination of load effects, E, shall be determined in accordance with Section 12.4.2.

EXCEPTION: *The redundancy factor, ρ, per Section 12.3.4 shall be taken as 1.*

§ 15.6 of ASCE7-05 has a list subsection for nonbuilding structures NOT similar to structures where the specific requirements of analysis and design are explained. The list includes:

- § **15.6.1 Earth-Retaining Structures**
- § **15.6.2 Stacks and Chimneys**
- § **15.6.3 Amusement Structures**
- § **15.6.4 Special Hydraulic Structures**
- § **15.6.6 Telecommunication Towers**

TABLE 15.4-2 SEISMIC COEFFICIENTS FOR NONBUILDING STRUCTURES NOT SIMILAR TO BUILDINGS

Nonbuilding Structure Type	R	Ω_0	C_d	STRUCTURAL SYSTEM AND HEIGHT LIMITS (ft)[a,e]				
				A&B	C	D	E	F
1- Elevated tanks, vessels, bins, or hoppers:								
On symmetrically braced legs (not similar to buildings)	3	2[b]	2.5	NL	NL	160	100	100
On unbraced legs or asymmetrically braced legs (not similar to buildings)	3	2[b]	2.5	NL	NL	160	100	100
Single pedestal or skirt supported								
- welded steel	2	2[b]	2	NL	NL	NL	NL	NL
- welded steel with special detailing	3	2[b]	2	NL	NL	NL	NL	NL
- prestressed or reinforced concrete	2	2[b]	2	NL	NL	NL	NL	NL
- prestressed or reinforced concrete with special detailing	3	2[b]	2	NL	NL	NL	NL	NL
2- Horizontal , saddle supported welded steel vessels	3	2[b]	2.5	NL	NL	NL	NL	NL
3- Tanks or vessels supported on structural towers similar to buildings	Use values for the appropriate structure type in the categories for building frame systems and moment resisting frame system listed in Table 15.4-1							

Nonbuilding Structure Type	R	Ω_0	C_d	STRUCTURAL SYSTEM AND HEIGHT LIMITS (ft)[a,e] A&B	C	D	E	F
4- Flat-bottom ground-supported tanks:								
Steel or fiber-reinforced plastic:								
Mechanically anchored	3	2[b]	2.5	NL	NL	NL	NL	NL
Self-anchored	2.5	2[b]	2	NL	NL	NL	NL	NL
Reinforced or prestressed concrete								
reinforced nonsliding base	2	2[b]	2	NL	NL	NL	NL	NL
anchored flexible base	3.25	2[b]	2	NL	NL	NL	NL	NL
unanchored and unconstrained flexible base	1.5	1.5[b]	2	NL	NL	NL	NL	NL
All other	1.5	1.5[b]	2	NL	NL	NL	NL	NL
5-Cast-in-place concrete silos, stacks, and chimneys having walls continuous to the foundation	3	1.75	3	NL	NL	NL	NL	NL
6-All other reinforced masonry structures not similar to buildings	3	2	2.5	NL	NL	NL	50	50
7-All other nonreinforced masonry structures not similar to buildings	1.25	2	1.5	NL	NL	50	50	50
8-All other steel and reinforced distributed mass cantilever structures not covered herein including stacks, chimneys, silos, and skirt-supported vertical vessels that are not similar to buildings	3	2	2.5	NL	NL	NL	NL	NL
9-Trussed towered (freestanding or guyed), guyed stacks and chimneys	3	2	2.5	NL	NL	NL	NL	NL
Nonbuilding Structure Type	R	Ω_0	C_d	STRUCTURAL SYSTEM AND HEIGHT LIMITS (ft)[a,e] A&B	C	D	E	F
10-Cooling towers								
Concrete or steel	3.5	1.75	3	NL	NL	NL	NL	NL
Wood frames	3.5	3	3	NL	NL	NL	50	50
11-Telecommunication towers:								
Truss: steel	3	1.5	3	NL	NL	NL	NL	NL
Pole:								
Steel	1.5	1.5	1.5	NL	NL	NL	NL	NL
Wood	1.5	1.5	1.5	NL	NL	NL	NL	NL
Concrete	1.5	1.5	1.5	NL	NL	NL	NL	NL
Frame:								
Steel	3	1.5	1.5	NL	NL	NL	NL	NL
Wood	1.5	1.5	1.5	NL	NL	NL	NL	NL
Concrete	2	1.5	1.5	NL	NL	NL	NL	NL
12-Amusement structures and monuments	2	2	2	NL	NL	NL	NL	NL

Nonbuilding Structure Type	R	Ω_0	C_d	STRUCTURAL SYSTEM AND HEIGHT LIMITS (ft)[a,e] A&B	C	D	E	F
13-Inverted pendulum type structures (except elevated tanks, vessels, bins, and hoppers	2	2	2	NL	NL	NL	NL	NL
14 -Signs and billboards	3.5	1.75	3	NL	NL	NL	NL	NL
15-All other self-supporting structures, tanks, or vessels not covered above or by reference standards that are similar to buildings	1.25	2	2.5	NL	NL	50	50	50

[a]NL = no limit and NP = not permitted. Height shall be measured from the base.
[b]See Section 15.7.3a for the application of the overstrength factors, Ω_0, for tanks and vessels
[c] If a section is not indicated in the Detailing Requirements column, no specific detailing requirements apply.
[d]For the purpose of height limit determination, the height of the structure shall be taken as the height to the top of the structural frame making up the primary seismic-force resisting system.

Seismic Base Shear Determination , *V*:

Table 6-3 (repeat) Seismic Base Shear (*V*) Flexible Nonbuilding Structures

Flexible Nonbuilding Structures	
Fundamental period, $T \geq 0.06$ *second*	
Similar to Buildings	**Not Similar to Buildings**
ELF for Buildings per §12.8 (6 Equations) with same minimum values.	ELF for Buildings per §12.8 (6 Equations) with **NEW** minimum values.

12.8.1 Seismic Base Shear. The seismic bases hear, V, in a given direction shall be determined in accordance with the following equation:

$$V = C_s W \tag{12.8-1}$$

where

C_s = the seismic response coefficient determined in accordance with Section 12.8.1.1

W = the effective seismic weight per Section 12.7.2.

§12.8.1.1 Calculation of Seismic Response Coefficient. The seismic response coefficient, C_s, shall be determined in accordance with Eq. 12.8-2.

$$C_S = \frac{S_{DS}}{\left(\frac{R}{I}\right)} \tag{12.8-2}$$

The value of C_s computed in accordance with Eq. 12.8-2 need not exceed the following:

$$C_S = \frac{S_{D1}}{T\left(\dfrac{R}{I}\right)} \qquad for \quad T \le T_L \tag{12.8-3}$$

$$C_S = \frac{S_{D1}\,T_L}{T^2\left(\dfrac{R}{I}\right)} \qquad for \quad T > T_L \tag{12.8-4}$$

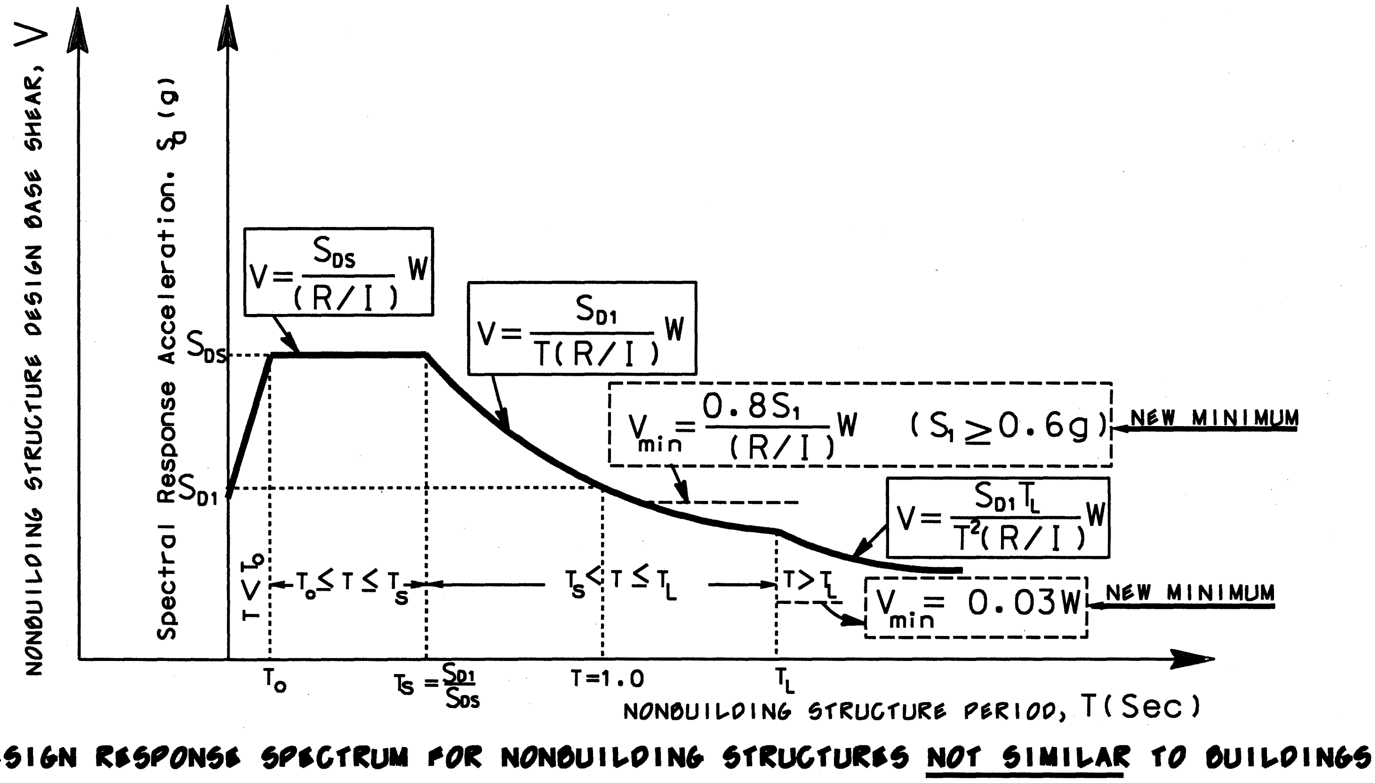


Figure 6-4 Design Response Spectrum for Nonbuilding **NOT Similar** to Buildings

C_s shall not be less than

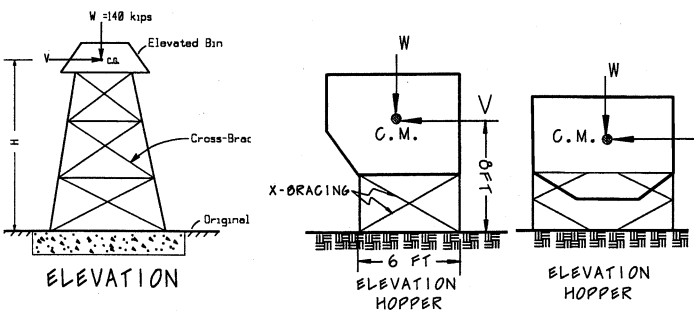
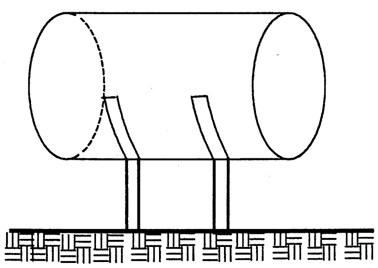
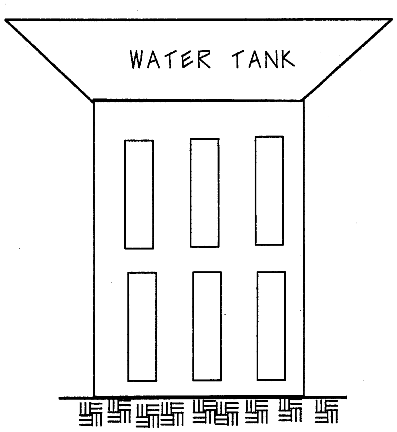
$$(C_s)_{\min} \ge 0.03 \qquad \Leftarrow \textit{New Minimum} \tag{15.4-1}$$

In addition, for structures located where S_1 is equal to or greater than $0.6g$, C_s shall not be less than

$$(C_s)_{\min} \ge \frac{0.8 S_1}{(R/I)} \quad (if\ S_1 \ge 0.6g) \quad \Leftarrow \textit{New Minimum} \tag{15.4-2}$$

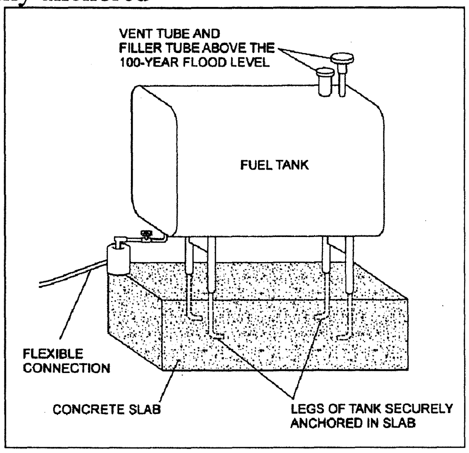
The following Table shows some examples of nonbuilding structure **NOT similar** to buildings with the listed values of R, Ω_0, C_d, and height limits for the 6 Seismic Design Categories (SDC "A" to "F")

Table 6-5 Examples of Nonbuilding Structures **NOT Similar** to Buildings

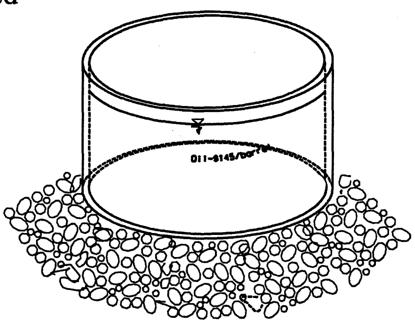
<table>
<tr><th rowspan="2">Nonbuilding Structure Type</th><th rowspan="2">R</th><th rowspan="2">Ω_0</th><th rowspan="2">C_d</th><th colspan="5">STRUCTURAL SYSTEM AND HEIGHT LIMITS (ft)[a,e]</th></tr>
<tr><th>A&B</th><th>C</th><th>D</th><th>E</th><th>F</th></tr>
<tr><td>1- Elevated tanks, bins, or hopper</td><td colspan="8"></td></tr>
<tr><td>On symmetrically braced legs (not similar to buildings)

</td><td>3</td><td>2^b</td><td>2.5</td><td>NL</td><td>NL</td><td>160</td><td>100</td><td>100</td></tr>
<tr><td>2-Horizontal, saddle supported welded steel vessels

ELEVATION
SADDLE SUPPORTED STEEL VESSEL</td><td>3</td><td>2^b</td><td>2.5</td><td>NL</td><td>NL</td><td>NL</td><td>NL</td><td>NL</td></tr>
<tr><td>3- Tanks or vessels supported on structural towers similar to buildings


ELEVATION
CONCRETE MOMENT FRAME</td><td colspan="8">Use values for the appropriate structure type in the categories for building frame systems and moment resisting frame system listed in Table 15.4-1</td></tr>
</table>

Nonbuilding Structure Type	R	Ω_0	C_d	STRUCTURAL SYSTEM AND HEIGHT LIMITS (ft)[a,e]				
				A&B	C	D	E	F
4- Flat-bottom ground-supported tanks:								
Steel or fiber-reinforced plastic:								
Mechanically anchored	3	2[b]	2.5	NL	NL	NL	NL	NL

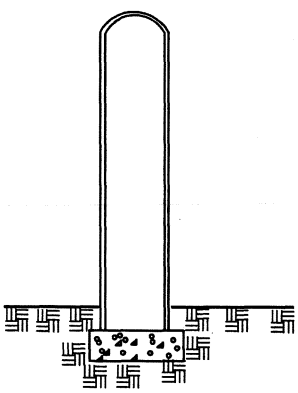


Nonbuilding Structure Type	R	Ω_0	C_d	A&B	C	D	E	F
Self-anchored	2.5	2[b]	2	NL	NL	NL	NL	NL

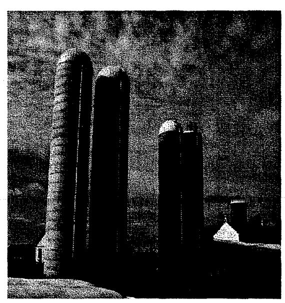


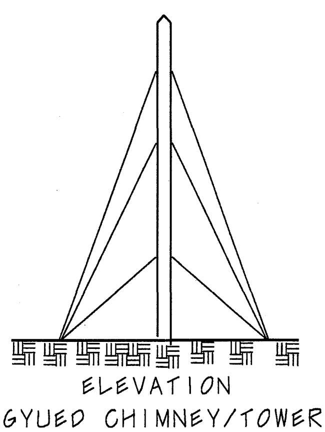
SELF-ANCHORED TANK

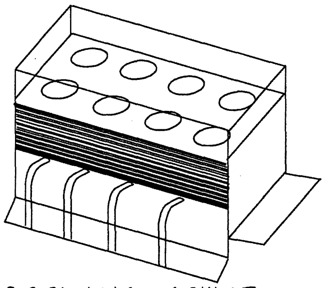
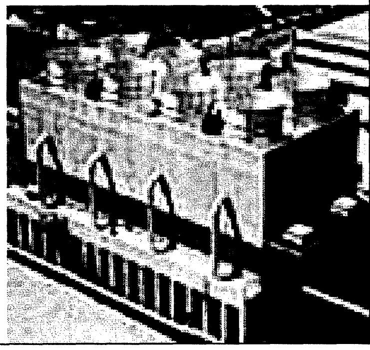
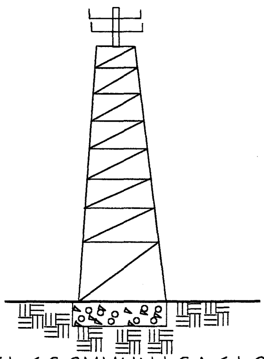
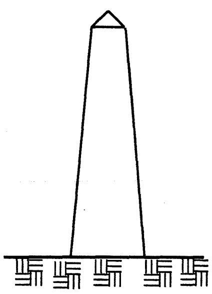
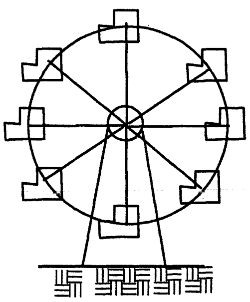
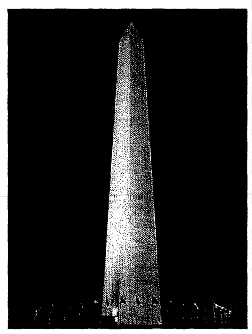
Nonbuilding Structure Type	R	Ω_0	C_d	A&B	C	D	E	F
5- Cast-in-place concrete silos, stacks, and chimneys having walls continuous to the foundation	3	1.75	3	NL	NL	NL	NL	NL

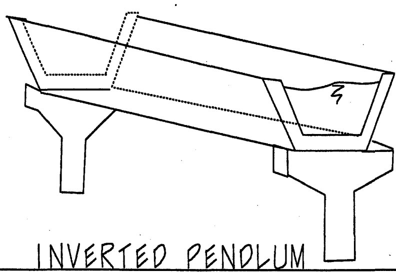
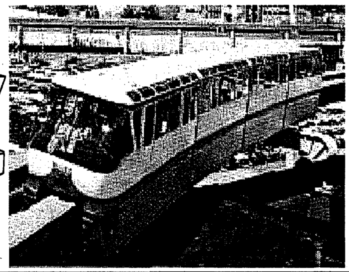
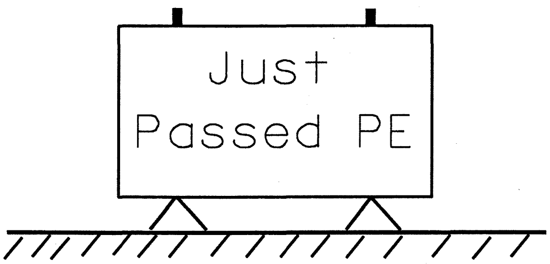


ELEVATION
SILO/CHIMNEY WITH WALLS
CONTINOUS TO FOUNDATION



Nonbuilding Structure Type	R	Ω_0	C_d	STRUCTURAL SYSTEM AND HEIGHT LIMITS (ft)[a,e]				
				A&B	C	D	E	F
6- All other reinforced masonry structures not similar to buildings	3	2	2.5	NL	NL	NL	50	50
7-All other nonreinforced masonry structures not similar to buildings	1.25	2	1.5	NL	NL	50	50	50
8- All other steel and reinforced distributed mass cantilever structures not covered herein including stacks, chimneys, silos, and skirt-supported vertical vessels that are not similar to buildings	3	2	2.5	NL	NL	NL	NL	NL
9-Trussed towered (freestanding or guyed), guyed stacks and chimneys  	3	2	2.5	NL	NL	NL	NL	NL

Nonbuilding Structure Type	R	Ω_0	C_d	STRUCTURAL SYSTEM AND HEIGHT LIMITS (ft)[a,e] A&B	C	D	E	F
10-Cooling towers								
Concrete or steel  COOLING TOWER 	3.5	1.75	3	NL	NL	NL	NL	NL
11- Telecommunication towers:								
Truss: steel  TELECOMMUNICATION STEEL TOWER	3	1.5	3	NL	NL	NL	NL	NL
12- Amusement structures and monuments   	2	2	2	NL	NL	NL	NL	NL

Nonbuilding Structure Type	R	Ω_0	C_d	STRUCTURAL SYSTEM AND HEIGHT LIMITS (ft)[a,e] A&B	C	D	E	F
13-Inverted pendulum type structures (except elevated tanks, vessels, bins, and hoppers  	2	2	2	NL	NL	NL	NL	NL
14- Signs and billboards 	3.5	1.75	3	NL	NL	NL	NL	NL
15- All other self-supporting structures, tanks, or vessels not covered above or by reference standards that are similar to buildings	1.25	2	2.5	NL	NL	50	50	50

6.10 SAMPLE PROBLEMS

Sample Problem 6-1 Types on Nonbuilding Structures

Given: Which of the following would be considered a nonbuilding structure?

I. A storage cabinet supported by a building structure
II. A transformer constructed of high deformability materials
III. A cast-in-place concrete silo having walls continuous to foundation
IV. A telecommunication tower made of steel truss on top of a mountain

Answer: A) I , II B) II & III C) III & IV D) All of the above

Solution:

From the basic definition of nonbuilding structures below, it is clear that I & II would not be classified as a nonbuilding structure. It would be considered as components and would be designed per Chapter 13 of ASCE7-05. (Check Table 13.5-1 for a list of ARCHITECURAL COMPONENT))

§ 15.1.1 Nonbuilding Structures. *Nonbuilding structures include all self-supporting structures that carry gravity loads and that may be required to resist the effects of earthquake, with the exception of building structures specifically excluded in Section 11.1.2, and other nonbuilding structures where specific seismic provisions have yet to be developed, and therefore, are not set forth in Chapter 15.*

Tables 15.4-1 & 2 have a list of nonbuilding structures and it could be used as a guide to determine the type.

Answer: C ⇐

Sample Problem 6-2 Exemption from Nonbuilding Structures Requirements

Given: Which of the following are exempt from the seismic requirements of the nonbuilding structure per ASCE 7-05?

I. A gravity dam
II. A monument
III. A cast-in-place concrete silo having walls continuous to foundation
IV. A telecommunication tower made of steel truss on top of a mountain

Answer: A) I B) I & II C) III & IV D) IV

Solution:

The following nonbuilding structures are except from the requirements of the ASCE7-05 as stated in §11.1.2 (item 4)

"§ 4. Structures that require special consideration of their response characteristics and environment that are not addressed in Chapter 15 and for which other regulations provide seismic criteria, such as vehicular bridges, electrical transmission towers, hydraulic structures, buried utility lines and their appurtenances, and nuclear reactors."

Therefore, the gravity dam is the only one that is except from ASCE7-05

Answer: A ⇐

Sample Problem 6-3 Fundamental Period Determination for Nonbuilding Structures

Given: Which of the following equation shall not be used to calculate the period (T) for a nonbuilding structure?

$$I.\quad T_a = C_t\, h_n^x$$

$$II.\quad T_a = 0.1\, N$$

$$III.\quad T_a = \frac{0.0019}{\sqrt{C_w}}\, h_n$$

$$IV.\quad T = 2\pi \sqrt{\frac{\sum_{i=1}^{n} w_i\, \delta_i^2}{g \sum_{i=1}^{n} f_i \delta_i}}$$

Answer: A) I B) II & III C) III & IV D) IV

Solution:

§15.4.4 Fundamental Period. The fundamental period of the nonbuilding structure shall be determined using the structural properties and deformation characteristics of the resisting elements. Equations12.8-7, 12.8-8, 12.8-9, and 12.8-10 shall not be used for determining the period of a nonbuilding structure.

The only equation permitted to be used is Equation IV (Equation 15.4-6 ASCE7-05).

Answer: D ⇐

Sample Problem 6-4 Response Modification Factor for Nonbuilding Structures

Given: A fiber-reinforced plastic tank that is mechanically anchored has a response modification factor of:

A) 2.0
B) 2.5
C) 3.0
D) 3.25

Solution:

A fiber-reinforced plastic tank that is mechanically anchored is a nonbuilding structure **NOT similar** to buildings. Tables 15.4- 2 (p. 163 of ASCE 7-05) has a list of this type of nonbuilding structures. The "*R*" value that should be used in design is 3.0

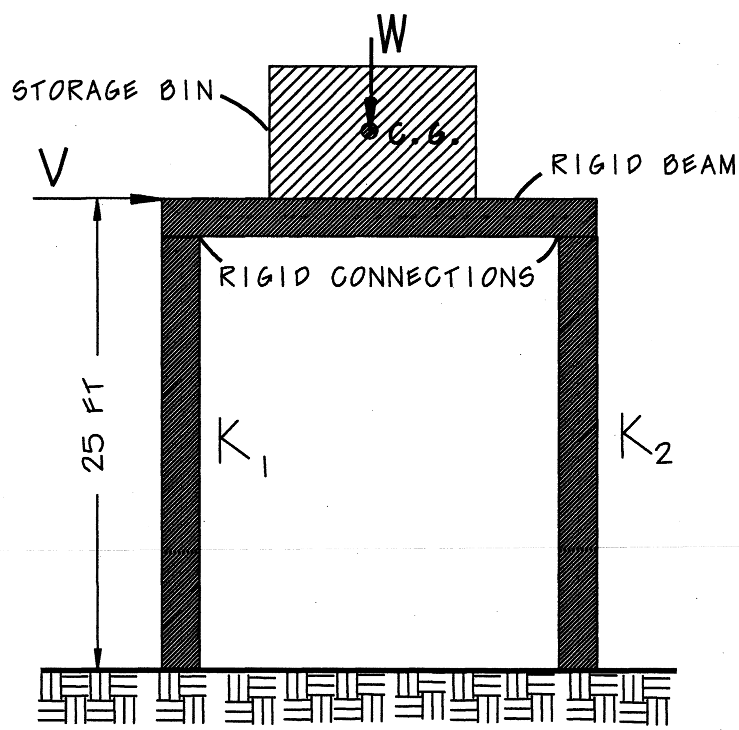
Note: A steel or fiber-reinforced plastic tank that is *self-anchored* has an "*R*" value of 2.5 which is less than 3. Remember that the large the *R*, the smaller the design base shear *V* as can be observed from the following equation (i.e. *V* is inversely proportion to *R*).

$$C_S = \frac{S_{DS}}{(R/I)}$$

Answer: C ⇐

The following figure and given information are to be used for prob. # 6.5 through 6.13

An elevated storage bin supported on special reinforced concrete moment frame is shown below. The frame is located at a site with unknown soil type. The short and 1s spectral response acceleration are S_S & S_1 are 1.84g and 0.85g respectively. The effective seismic dead load "W" including the maximum operating contents is 120 kips. The stiffness of the supporting columns ($k_1 = k_2$) was calculated and found to be 5.30 kip/inch each. Consider the storage elevated bin is classified as Occupancy Category I. Ignore the weight of the moment frame.



ELEVATION
REINFORCED MOMENT FRAME

Sample Problem 6-5 Facility Type

Find: The given facility would be classified as:

Answer: A) A diaphragm B) A Building structure
C) A Nonbuilding structure D) A Component of a structure

Solution:

§15.1.1 Nonbuilding Structures. Nonbuilding structures include all self-supporting structures that carry gravity loads and that may be required to resist the effects of earthquake, with the exception of building structures specifically excluded in Section 11.1.2, and other nonbuilding structures where specific seismic provisions have yet to be developed, and therefore, are not set forth in Chapter 15.

Answer: C ⇐

Sample Problem 6-6 Facility Type

Find: The given facility would be classified as:

Answer:

A) Nonbuilding structure similar to buildings
B) A building Structure
C) Nonbuilding structure not similar to buildings
D) Nonbuilding structure supported by other structures

Solution:

Comparing the given facility with those listed in Tables 15.4-1(nonbuilding structures similar to buildings) and Table 15.4-2 (nonbuilding structures not similar to buildings), it can be see that the given facility is a nonbuilding building structure similar to buildings

§ 11.2 of ASCE7-05: NONBUILDING STRUCTURE SIMILAR TO A BUILDING: A nonbuilding structure that is *designed and constructed in a manner similar to buildings*, *will respond to strong ground motion in a fashion similar to buildings*, and have basic lateral and vertical seismic-force-resisting-system conforming to one of the types indicated in Tables 12.2-1 or 15.4-1.

Three criteria for nonbuilding structures similar to buildings:

1- *designed and constructed in a manner similar to buildings*
2- *will respond to strong ground motion in a fashion similar to buildings, and*
3- *have basic lateral and vertical seismic-force-resisting-system conforming to one of the types indicated in Tables 12.2-1 or 15.4-1.*

Answer: A ⇐

Sample Problem 6-7 Site Class

Find: The site class would be used for the design of this nonbuilding structure is:

Answer: A) Class "B" B) Class "C" C) Class "D" D) Class "E"

Solution:

§1613.5.2 Site class definitions. Based on the site soil properties, the site shall be classified as either Site Class A, B, C, D, E or F in accordance with Table 1613.5.2. When the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used unless the building official or geotechnical data determines that Site Class E or F soil is likely to be present at the site.

Answer: C ⇐

Sample Problem 6-8 Flexible Nonbuilding Structures (similar to buildings)

Find: The structure period "T" is most nearly :

Answer: A) 0.85 sec B) 1.08 sec C) 1.12 sec D) 1.52 sec

Solution:

The storage bin is considered "a Nonbuilding Structure" based on the following definition:

§15.1.1 Nonbuilding Structures. Nonbuilding structures include all self-supporting structures that carry gravity loads and that may be required to resist the effects of earthquake, with the exception of building structures specifically excluded in Section 11.1.2, and other nonbuilding structures where specific seismic provisions have yet to be developed, and therefore, are not set forth in Chapter 15

§15.4.4 Fundamental Period. The fundamental period of the nonbuilding structure shall be determined using the structural properties and deformation characteristics of the resisting elements in a properly substantiated analysis as indicated in Section 12.8.2. Alternatively, the fundamental period T is permitted to be computed from the following equation:

$$T = 2\pi \sqrt{\frac{\sum_{i=1}^{n} w_i \delta_i^2}{g \sum_{i=1}^{n} f_i \delta_i}} \qquad (15.4-6)$$

The values of f_i represent any lateral force distribution in accordance with the principles of structural mechanics. The elastic deflections, δ_i, shall be calculated using the applied lateral forces, f_i . ***Equations12.8-7, 12.8-8, 12.8-9, and 12.8-10 shall not be used for determining the period of a nonbuilding structure.***

Equation (15.4-6) will be reduced for a single degree of freedom (SDOF) system to:

$$T = 2\pi\sqrt{\frac{m}{k}} = 2\pi\sqrt{\frac{W}{k\,g}} = 2\pi\sqrt{\frac{120\,kips}{(2\times 5.30\,k/in)\times(32.2\,ft/s^2\times 12in/ft)}} = 1.08 \text{ sec}$$

Note : This equation (for SDOF) is similar to Equation (13.6-1) for the component period.

Answer: B ⇐

Sample Problem 6-9 Determination of S_{DS} & S_{D1}

Find: The S_{DS} & S_{D1} for the given nonbuilding structure are most nearly

Answer: A) 1.84g & 0.85g B) 1.23g & 0.65g
C) 1.10g & 0.85g D) 1.23g & 0.85g

Solution:

Step1: Calculate S_{MS} & S_{M1} using Eq. 11.4-1 & 11.4-2:

§ 11.4.3 Site Coefficients and Adjusted Maximum Considered Earthquake (MCE) Spectral Response Acceleration Parameters.
The MCE spectral response acceleration for short periods (S_{MS}) and at 1s (S_{M1}), adjusted for Site Class effects, shall be determined by Eqs. 11.4-1 and 11.4-2, respectively.

For the given values of S_S & S_1 of 1.84g and 0.85g, F_a & F_v are determined from Tables 11.4-1 & 11.4-2 as follows:

$$S_S = 1.84g \text{ \& Site class D} \Rightarrow F_a = 1.0$$
$$S_1 = 0.85g \text{ \& Site class D} \Rightarrow F_v = 1.5$$

$$S_{MS} = F_a S_S = 1.0\,(1.84g) = 1.84g \qquad (11.4\text{-}1)$$
$$S_{M1} = F_v S_1 = 1.5\,(0.85g) = 1.28g \qquad (11.4\text{-}2)$$

Where

S_S = the mapped MCE spectral response acceleration at short periods as determined in accordance with Section 11.4.1,and

S_1 = the mapped MCE spectral response acceleration at a period of 1 s as determined in accordance with Section 11.4.1

where site coefficients F_a and F_v are defined in Tables 11.4-1 and 11.4-2, respectively.

Step2: Calculate S_{DS} & S_{D1} using Eq. 11.4-3 & 11.4-4:

§ 11.4.4 Design Spectral Acceleration Parameters. Design earthquake spectral response acceleration parameter at short period, S_{DS}, and at 1s period, S_{D1}, shall be determined from Eqs. 11.4-3 and 11.4-4, respectively. Where the alternate simplified design procedure of Section 12.14 is used, the value of S_{DS} shall be determined in accordance with Section 12.14.8.1, and the value for S_{D1}, need not be determined.

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(1.84g) = 1.23g \qquad (11.4\text{-}3)$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(1.28g) = 0.85g \qquad (11.4\text{-}4)$$

Answer: A $\Leftarrow$

Sample Problem 6-10 Seismic Design Category (SDC) for Nonbuilding Structure

Find: What is the appropriate Seismic Design Category (SDC) for this non-building structure?

Answer: A) SDC " A" B) SDC "C" C) SDC " D" D) SDC " E"

Solution:

Tables 1613.5.6 (1) &(2) will be used to determine SDC as follows:
$S_1 = 0.85g > 0.75g$ Table 11.6-1 can **NOT** be used alone.

The following Table shows the SDC selection based on the given information.

Facility	O.C.	Table 16.6-1		Table 16.6-2	
		S_{DS}	SDC	S_{D1}	SDC
Storage	I	1.23g	D	0.85g	D

NOTE: SDC "E" & "F" are determined based on the following table (summary of § 11.6 ASCE7-05):

Value of S_1	Occupancy Category		
	I or II	III	IV
$S_1 \geq 0.75$	E	E	F

Answer: C⇐

Sample Problem 6-11 Maximum Height for Nonbuilding Structures

Find: The maximum height permitted by the ASCE 7-05 for this nonbuilding structure is:

Answer: A) 50 ft B) 100 ft C) 160 ft D) No limit

Solution:
Per Table 15.4-1, the maximum height for special reinforced moment frame for a SDC "D" is NO LIMIT "NL".

Answer: C⇐

Sample Problem 6-12 Rigid Non-building vs. Flexible Non-building Structures

Find: The given nonbuilding structure is considered:

Answer: A) Rigid B) Flexible
C) Neither D) Not enough information

Solution:
§15.4.2 Rigid Nonbuilding Structures. Nonbuilding structures that have a fundamental period, T, less than 0.06 s , ……

Since the calculated period is 1.08s > 0.06 s, therefore it is a flexible.

Answer: B⇐

Sample Problem 6-13 Design Base shear for nonbuilding structures

Find: The design base shear (V) for the given nonbuilding structure is most nearly

Answer: A) 11.81 kips B) 13.4 kips C) 15.65 kips D) 16.67kips

Solution:

Since the calculated period is 1.08s > 0.06 s, therefore it is a flexible. The design base shear (V) will be calculated according to the equations of nonbuilding structures similar to building structures.

§12.8.1 Seismic Base Shear. The seismic bases hear, V, in a given direction shall be determined in accordance with the following equation:

$$V = C_s W \qquad (12.8\text{-}1)$$

where

C_s = the seismic response coefficient determined in accordance with Section 12.8.1.1

W = the effective seismic weight per Section 12.7.2.

§12.8.1.1 Calculation of Seismic Response Coefficient. The seismic response coefficient, C_s, shall be determined in accordance with Eq. 12.8-2.

$$C_S = \frac{S_{DS}}{\left(\frac{R}{I}\right)} \qquad (12.8\text{-}2)$$

The value of C_s computed in accordance with Eq. 12.8-2 need not exceed the following:

$$C_S = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} \quad for \quad T \le T_L \qquad (12.8\text{-}3)$$

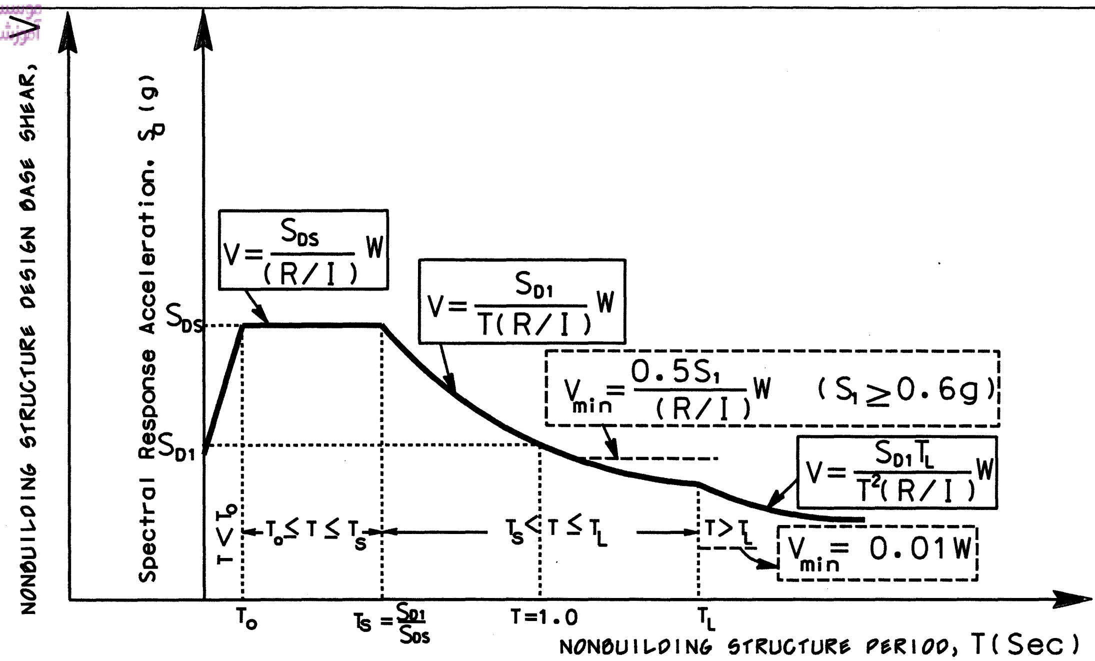
$$C_S = \frac{S_{D1}\,T_L}{T^2\left(\frac{R}{I}\right)} \quad for \quad T > T_L \qquad (12.8\text{-}4)$$

C_s shall not be less than

$$(C_s)_{\min} \ge 0.01 \qquad (12.8\text{-}5)$$

In addition, for structures located where S_1 is equal to or greater than 0.6g, C_s shall not be less than

$$(C_s)_{\min} \ge \frac{0.5\,S_1}{(R/I)} \quad (if\ S_1 \ge 0.6g) \qquad (12.8.6)$$



DESIGN RESPONSE SPECTRUM FOR NONBUILDING STRUCTURES SIMILAR TO BUILDINGS

Figure 6-3 Design Response Spectrum for Nonbuilding **Similar** to Buildings

Step 1: Calculate the period "T_S" $T_S = \dfrac{S_{D1}}{D_{DS}}$ (T_S is not part of any base shear equations, its used for comparison only)

$$T_S = \frac{S_{D1}}{D_{DS}} = \frac{0.85g}{1.23g} = 0.69 < T = 1.08s \quad \textbf{(go to step \#5)}$$

Since $T > T_s$, skip Steps 2 , 3 & 4 (not govern). Check the above figure.

Step 2: Calculate the period "T_0" $T_0 = 0.2\dfrac{S_{D1}}{D_{DS}}$ (T_0 is not part of any base shear equations, its used for comparison only)

Step 3: If $T < T_0$, the acceleration is given as $S_a = S_{DS}\left(0.4 + 0.6\dfrac{T}{T_0}\right)$ and the base shear will be as follows: $V = {S_a W}/{R/I}$

Step 4: If $T_0 \le T < T_S$, Eq. (12.8-2) governs and no need to check the minimum values of (Eq. 12.8-5) & (Eq. 12.8-6).

Step 5: If $T_S < T \le T_L$, Eq. (12.8-3) governs and the minimum values of (Eq. 12.8-5) & (Eq. 12.8-6) should be checked.

From Figure 22-15 of ASCE 7-05, T_L (long Period) values are between 4 to 16 seconds.
Therefore, skip Step 6 (not govern) because T< T_L. Check the above figure.

Step 6: If $T > T_L$, Eq.(12.8-4) governs

∴ the base shear will be calculated based on step 5 where Eq. 12.8-3 will be used and check the minimum.

R = 8 (Table 15.4-1) & I= 1 (Table 11.5-1)

$$C_S = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.85}{1.08(8/1)} = 0.0984 \qquad for \quad T \le T_L$$

C_s shall not be less than

$$0.0984 > (C_s)_{\min} \geq 0.01 \qquad \therefore \text{OK} \qquad (12.8\text{-}5)$$

In addition, for structures located where S_1 is equal to or greater than $0.6g$, C_s shall not be less than

$$0.0984 > (C_s)_{\min} \geq \frac{0.5\,S_1}{(R/I)} = \frac{0.5\,(0.85)}{(8/1)} = 0.05 \quad (if\ S_1 \geq 0.6g) \ \therefore OK \qquad (12.8.6)$$

Finally,

$$V = C_s W = 0.0984\ (120\ \text{kips}) = 11.81\ \text{kips} \qquad (12.8\text{-}1)$$

Answer: A ⇐

Sample Problem 6-14 Exemption from Nonbuilding Structures Requirements

Given: Which of the following are exempt from the seismic requirements of the a nonbuilding structure per ASCE 7-05?

I. A welded steel vessels supported on a horizontal saddle
II. A 8-inch high pressure gas line
III. A nuclear reactors
IV. A 60 kilo-volt (KV) transmission tower

Answer: A) I, II B) III &IV C) III & IV D) II, III & IV

Solution:

The following nonbuilding structures are except from the requirements of the ASCE7-05 as stated in §11.1.2 (item 4)

"§ 4. Structures that require special consideration of their response characteristics and environment that are not addressed in Chapter 15 and for which other regulations provide seismic criteria, such as vehicular bridges, electrical transmission towers, hydraulic structures, buried utility lines and their appurtenances, and nuclear reactors."

Answer: C ⇐

Sample Problem 6-15 Response Modification Factor for Nonbuilding Structures

Given: A flat-bottom ground-supported tank made of prestressed concrete with a reinforced nonsliding base**.** The response modification factor would be used for design is:

A) 2.0
B) 2.5
C) 3.0
D) 3.25

Solution:

A flat-bottom ground-supported tank made of prestressed concrete with a reinforced nonsliding base is a nonbuilding structure **NOT** similar to buildings. Tables 15.4- 2 (p. 163 of ASCE 7-05) has a list of this type of nonbuilding structures. The "*R*" value that should be used in design is 2.0

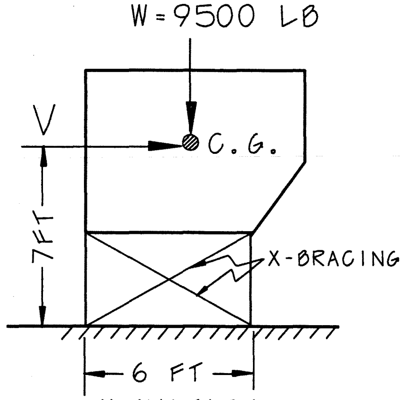
Note: A flat-bottom ground supported tank made of prestressed concrete with *an anchored flexible base* has an "*R*" value of 3.25 which is grater than 2. This is because the anchored with flexible base is capable of absorbing more energy that the reinforced nonsliding base type. Remember that the large the *R*, the smaller the design base shear *V* as can be observed from the following equation (i.e. *V* is inversely proportion to *R*).

$$C_S = \frac{S_{DS}}{(R/I)}$$

Answer: A ⇐

The following figure and given information are to be used for problems 6-16

A heavy duty hopper has symmetrically x-braced legs (not similar to buildings) supported on the ground as shown. The hopper operating weight (W) is 9500 lb acting the center of mass (CM/CG). The S_{DS} = 0.90g, S_{D1} = 0.43g & S_1 = 0.55g. The cross-bracing is provided on all four sides of the hopper and the bracing is assumed to resist axial tension loads only (no compression).



ELEVATION
HOPPER

Sample Problem 6-16 Seismic Force Calculations for Rigid Nonbuilding Structures (V)

Find: What would be the seismic base shear (V) of this non-building structure if the natural period was determined to be 0.05 seconds.

Answer: A)1.66 kips B) 2.57 kips C) 3.66 kips D) 3.60 kips

Solution:

§ 15.4.2 Rigid Nonbuilding Structures. Nonbuilding structures that have a fundamental period, T, less than 0.06 s, including their anchorages, shall be designed for the lateral force obtained from the following:

$T = 0.05$ seconds < 0.06 seconds, therefore Rigid Nonbuilding Structure

$$V = 0.30 S_{DS} WI = 0.30(0.90\text{ g})(9.5\text{ kips/g})(1.0) = 2.57\text{ kips} \qquad (15.4\text{-}5)$$

Where:

V = the total design lateral seismic base shear force applied to a nonbuilding structure

S_{DS} = the site design response acceleration as determined from Section 11.4.4

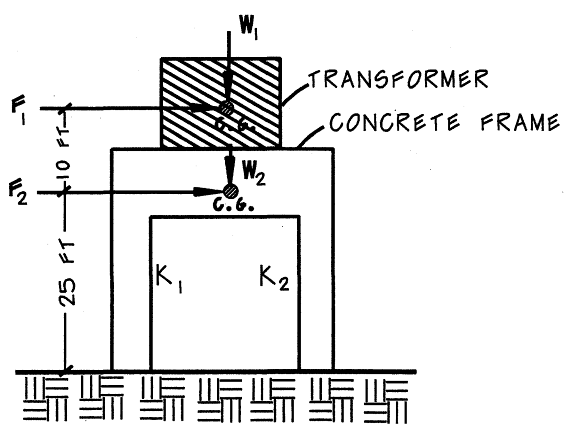
W = nonbuilding structure operating weight

I = the importance factor determined in accordance with Section 15.4.1.1

Answer: B ⇐

Sample Problem 6-17 Limitations Certain Types of Nonbuilding Structures

Given: A transformer is supported by a concrete frame as shown below. The frame is located at a site assigned a Seismic Design Category (SDC) "E" . Which type of frame would be acceptable per ASCE 7-05?



ELEVATION
TRANSFORMER SUPPORTED BY RIGID FRAME

I. Intermediate reinforced concrete moment frame
II. Ordinary reinforced concrete moment frame
III. Special reinforced concrete moment frame
IV. Intermediate reinforced concrete moment frame with unlimited height

Answer: A) I &II B) II & III C) III & IV D) I, II, III & IV

Solution:

Per Table 15.4-1, I & II are not permitted in SDC "E" . III & IV are permitted in SDC "E". It should be noted that Intermediate reinforced concrete moment frame with unlimited height has a maximum height of 50 ft. Based on the given dimensions and the note "e" (below) of Table 15.4-1, 25 ft < 50 ft.

[e] For the purpose of height limit determination, the height of the structure shall be taken as the height to the top of the structural frame making up the primary seismic-force resisting system.

Answer: C ⇐

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Chapter 7

Horizontal Diaphragms and Shear Walls

Topics to be covered

- Types of Diaphragms
- Forces in Diaphragms (Chord Force & Drag Force)
- Diaphragm Design Force, F_{px}
- Rigid Diaphragms (CM, CR, e_{actual}, $e_{accidental}$, $M_{Torsional}$ &Negative Torsional Shear)
- Shear Walls (Types, Rigidity, Walls with Openings)
- Anchorage Requirements
- Anchorage of Concrete or Masonry Walls

15 Practice Problems

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Chapter 7- Horizontal Diaphragms and Shear Walls

7.1 INTRODUCTION AND DEFINITIONS

A flexible diaphragm acts as a beam in the plane of a roof or floor that spans between shear walls (vertical diaphragms).

Diaphragms shall be designed for both the shear and bending stresses resulting from design forces. At diaphragm discontinuities, such as openings and reentrant corners, the design shall assure that the dissipation or transfer of edge (chord) forces combined with other forces in the diaphragm is within shear and tension capacity of the diaphragm.

Although the unit shear (τ_{diaph}) is a major factor in a diaphragm design, there are a number of additional items that must be addressed. The basic design considerations for a diaphragm are:

i) Sheathing thickness
ii) Diaphragm nailing
iii) Chord design
iv) Collector (strut) design
v) Diaphragm deflection
vi) Tie and anchorage requirements

The following are the definitions associated with diaphragms and shear walls:

BOUNDARY ELEMENTS: Diaphragm and shear wall boundary members to which the diaphragm transfers forces. Boundary members include chords and drag struts at diaphragm and shear wall perimeters, interior openings, discontinuities, and reentrant corners.

BOUNDARY MEMBERS: Portions along wall and diaphragm edges strengthened by longitudinal and transverse reinforcement. Boundary members include chords and drag struts at diaphragm and shear wall perimeters, interior openings, discontinuities, and reentrant corners.

DESIGN EARTHQUAKE: The earthquake effects that are two-thirds of the corresponding Maximum Considered Earthquake (MCE) effects.

DESIGN EARTHQUAKE GROUND MOTION: The earthquake ground motions that are two-thirds of the corresponding MCE ground motions.

DIAPHRAGM: Roof, floor, or other membrane or bracing system acting to transfer the lateral forces to the vertical resisting elements.

DIAPHRAGM BOUNDARY: A location where shear is transferred into or out of the diaphragm element. Transfer is either to a boundary element or to another force-resisting element.

DIAPHRAGM CHORD : A diaphragm boundary element perpendicular to the applied load that is assumed to take axial stresses due to the diaphragm moment.

DRAG STRUT (COLLECTOR, TIE, DIAPHRAGM STRUT): A diaphragm or shear wall boundary element parallel to the applied load that collects and transfers diaphragm shear forces to the vertical force-resisting elements or distributes forces within the diaphragm or shear wall.

SHEAR PANEL: A floor, roof, or wall component sheathed to act as a shear wall or diaphragm.

SUBDIAPHRAGM: A portion of a diaphragm used to transfer wall anchorage forces to diaphragm cross ties.

WALL: A component that has a slope of 60° or greater with the horizontal plane used to enclose or divide space.

Bearing Wall: Any wall meeting either of the following classifications:

1. Any metal or wood stud wall that supports more than 100 lb/linear ft (1,459 N/m) of vertical load in addition to its own weight.
2. Any concrete or masonry wall that supports more than 200 lb/linear ft (2,919 N/m) of vertical load in addition to its own weight.

Light-Framed Wall: A wall with wood or steel studs.

Light-Framed Wood Shear Wall: A wall constructed with wood studs and sheathed with material rated for shear resistance.

Nonbearing Wall: Any wall that is not a bearing wall.

Nonstructural Wall: All walls other than bearing walls or shear walls.

Shear Wall (Vertical Diaphragm): A wall, bearing or nonbearing, designed to resist lateral forces acting in the plane of the wall (sometimes referred to as a "vertical diaphragm").

Structural Wall: Walls that meet the definition for bearing walls or shear walls.

WALL SYSTEM, BEARING: A structural system with bearing walls providing support for all or major portions of the vertical loads. Shear walls or braced frames provide seismic force resistance.

WOOD STRUCTURAL PANEL: A wood-based panel product that meets the requirements of DOC PS1 or DOC PS2 and is bonded with a waterproof adhesive. Included under this designation are plywood, oriented strand board, and composite panels.

7.2 TYPES OF DIAPHRAGM

Generally speaking, there are two types of diaphragms as follows:

Table 7- 1 COMPARISON BETWEEN FLEXIBLE and RIGID DIAPHRAGMS

A- Flexible Diaphragm	B- Rigid Diaphragm
➢ Not capable of resisting torsional moment.	➢ Capable of resisting torsional moment.
➢ Load is transferred to lateral resisting elements based on <u>tributary width</u>.	➢ Load is transferred to lateral resisting elements based on their <u>relative or absolute rigidities.</u>
➢ Maximum Diaphragm Deflection > 2 (Average Drift of Vertical Element) $MMD > 2(ADVE)$ $\Delta_{max} > \frac{\delta_{max} + \delta_{min}}{2}$	➢ Maximum Diaphragm Deflection ≤ 2 (Average Drift of Vertical Element) $MMD \leq 2(ADVE)$ $\Delta_{max} \leq \frac{\delta_{max} + \delta_{min}}{2}$
➢ Examples: Wood structural panels, Corrugated steel metal (CSM) unfilled with concrete	➢ Examples: Concrete slabs, Corrugated steel metal (CSM) filled with concrete

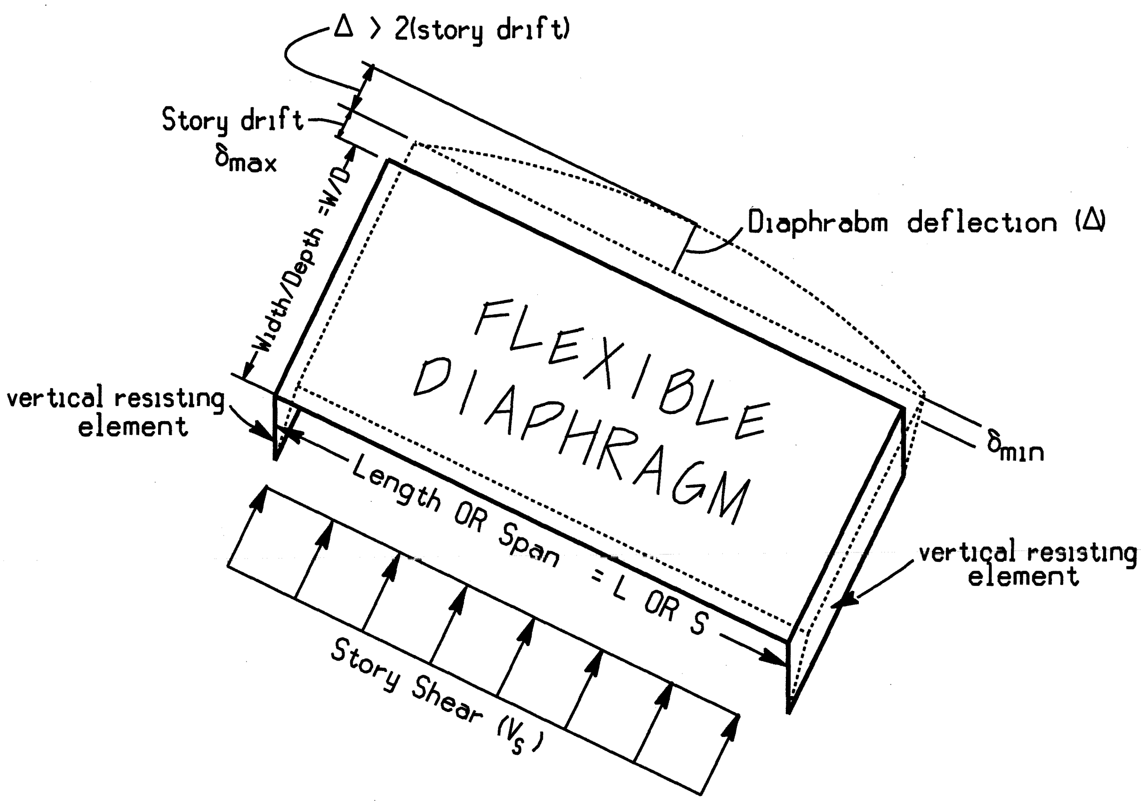


Figure 7-1 Flexible Diaphragm Definition

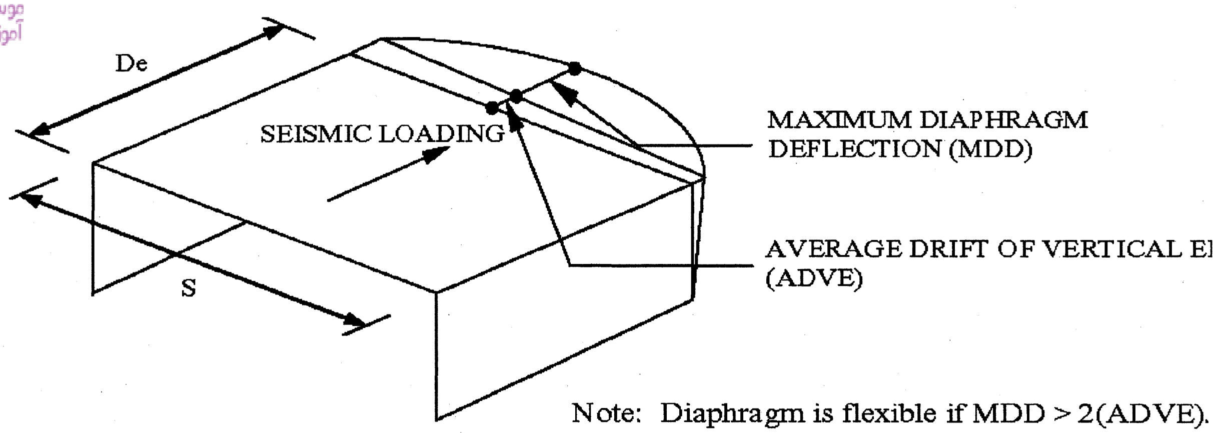


Figure 7-2 Flexible Diaphragm Condition
(FIGURE 12.3-1 FLEXIBLE DIAPHRAGM-ASCE7-05)

§12.3.1.1 **Flexible Diaphragm Condition.** Diaphragms constructed of untopped steel decking or wood structural panels are permitted to be idealized as flexible in structures in which the vertical elements are steel or composite steel and concrete braced frames, or concrete, masonry, steel, or composite shear walls. Diaphragms of wood structural panels or untooped steel decks in one-and two-family residential buildings of light-frame construction shall also be permitted to be idealized as flexible.

A-Flexible Diaphragms

Load is transferred to lateral resisting elements based on tributary width

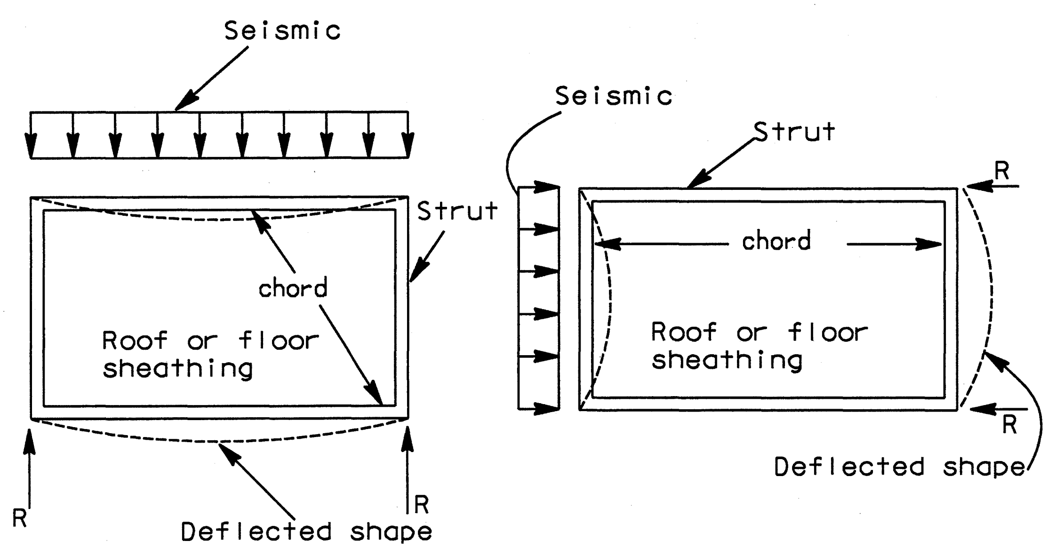


Figure 7-3 Diaphragm Forces and Boundary Members (Chords & Struts)

NOTE: Length or span (L or S) of the diaphragm is the dimension perpendicular to the load, while the width or depth (W or D) is the one parallel to the load. Therefore, the chords and struts are interchangeable depending on the direction of analysis (load).

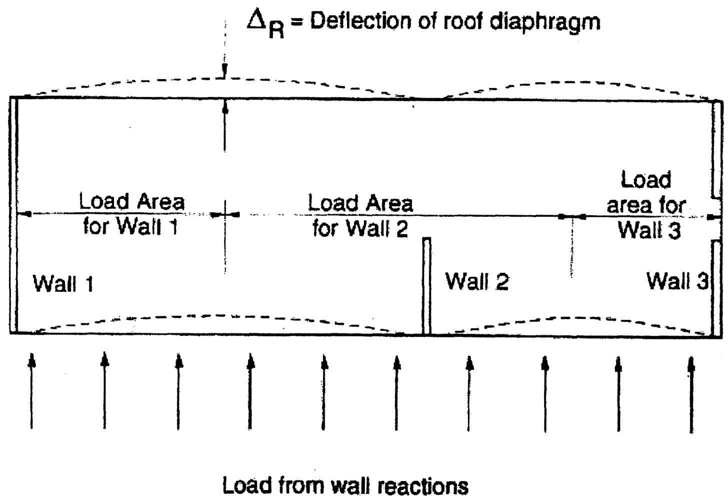


Figure 7-4 Tributary Width of Active Shear Walls

§12.3.1.3 **Calculated Flexible Diaphragm Condition**. Diaphragms not satisfying the conditions of Sections 12.3.1.1 or 12.3.1.2 are permitted to be idealized as flexible where the computed maximum in–plane deflection of the diaphragm under lateral load is more than **two times** the average story drift of adjoining vertical elements of the seismic force–resisting system of the associated story under equivalent tributary lateral load as shown in Fig. 12.3-1. The loadings used for this calculation shall be those prescribed by Section 12.8.

B-Rigid Diaphragms

Load is transferred to lateral resisting elements
based on their relative/absolute rigidities or stiffnesses

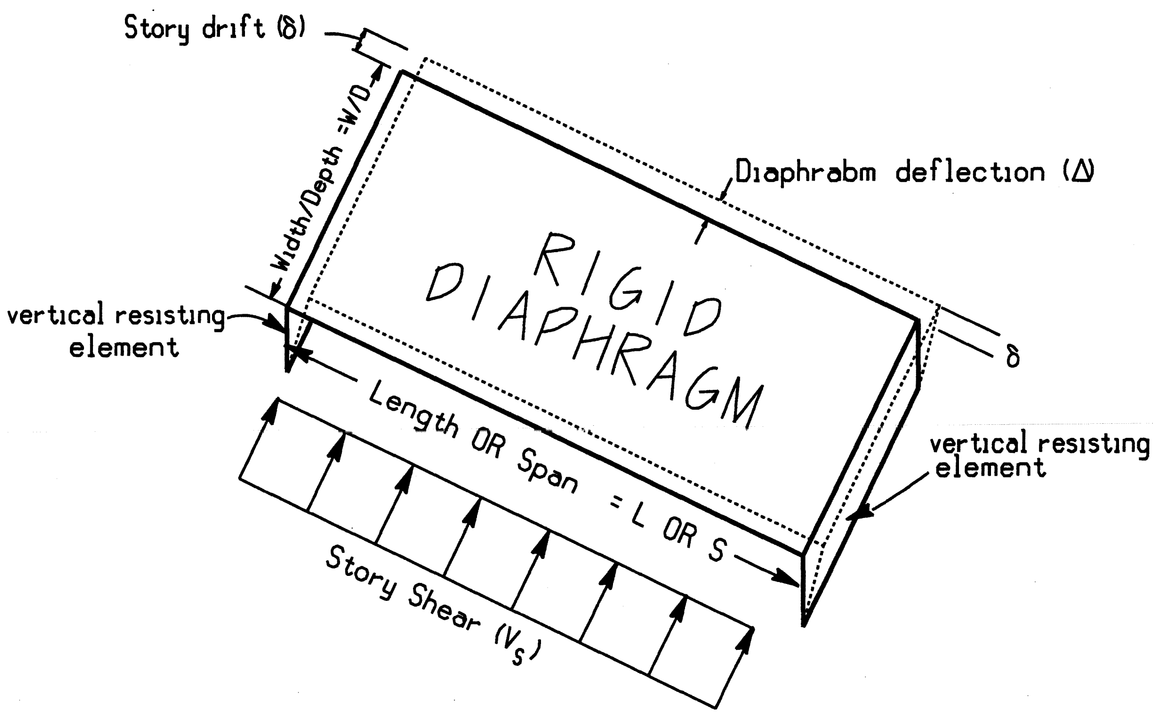


Figure 7-5 Rigid Diaphragm Condition

Rigid diaphragms are:

- Capable of resisting torsion and distribute the seismic lateral forces in proportion to the rigidities (or stiffness) of the lateral force resisting systems (shear walls, frames,… etc).
- Rigid diaphragm has a uniform deflection over it entire area, i.e. lateral displacement of any point will be the same for other points in the diaphragm.

§12.3.1.2 **Rigid Diaphragm Condition**. Diaphragms of concrete slabs or concrete filled metal deck with span-to-depth ratios of 3 or less in structures that have no horizontal irregularities are permitted to be idealized as rigid.

7.3 DIAPHRAGM DESIGN FORCE (F_{px})

Floor and roof diaphragms force F_{px} shall be determined from Equation 12.10-1 with the indicated upper and lower limits. It should be noted that the diaphragm force (F_{px}) is not the same as the seismic force (F_x) calculated in Chapter 4. The following figure illustrates the difference between these two forces.

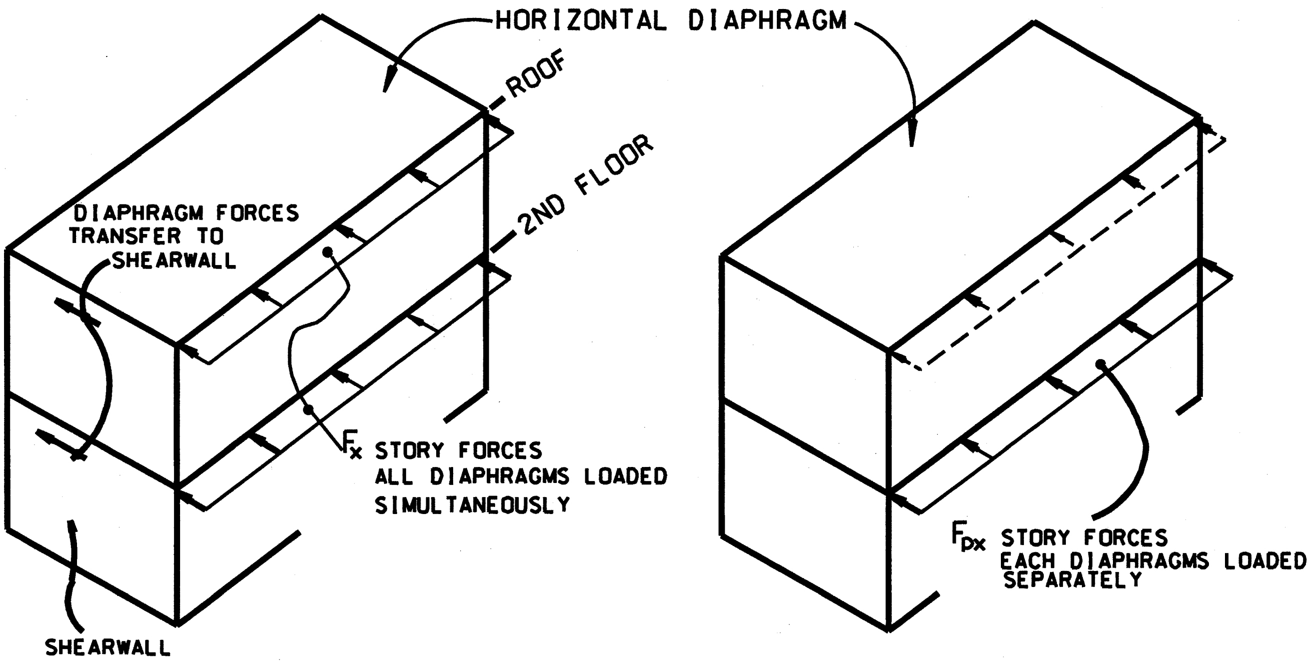


Figure 7-6 Story Shear Force(F_x) and Diaphragm Shear Force (F_{px})

§ 12.10.1 **Diaphragm Design**. Diaphragms shall be designed for <u>both the shear and bending stresses resulting from design forces</u>. At diaphragm discontinuities, such as openings and reentrant corners, the design shall assure that the dissipation or transfer of edge (chord) forces combined with other forces in the diaphragm is within shear and tension capacity of the diaphragm.

§12.10.1.1 **Diaphragm Design Forces.** Floor and roof diaphragms shall be designed to resist design seismic forces from the structural analysis, but shall not be less than that determined in accordance with Eq. 12.10-1 as follows:

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \qquad (12.10-1)$$

$$(0.2S_{DS}Iw_{px})_{\min} \le F_{px} \le (0.4S_{DS}Iw_{px})_{\max}$$

where
F_{px} = the diaphragm design force
F_i = the design force applied to Level i
w_i = the weight tributary to Level i
w_{px} = the weight tributary to the diaphragm at Level x

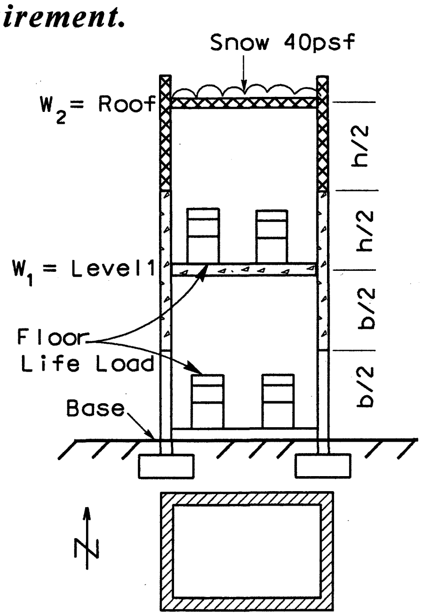
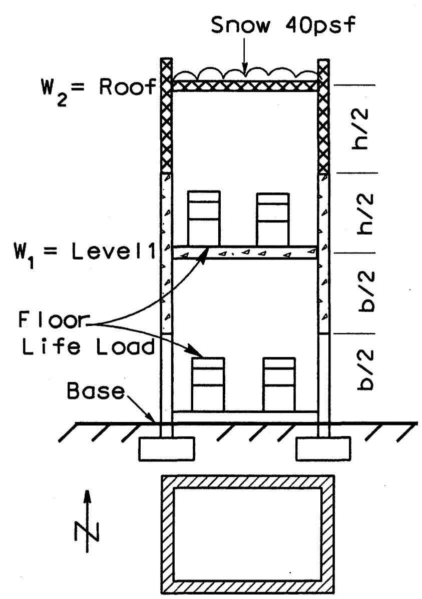
It should be noted that w_{px} is not the same as w_x. Sometimes for simplicity they are assumed to be equal and this is conservative because it will produce larger value of F_{px}. The following table shows the difference between w_x and w_{px}. w_{px} does not include walls parallel to the direction of analysis because they do not contribute to the load on the diaphragm. This is analogous to concentrated loads acting at the supports of a simply supported beam where these concentrated loads do not change the shear and moment values of the beam.

The redundancy factor, ρ, applies to the design of diaphragms in structures assigned to Seismic Design Category (SDC) D, E, or F. For inertial forces calculated in accordance with Eq. 12.10-1, the redundancy factor shall equal 1.0.

Table 7- 2 COMPARISON BETWEEN w_x & w_{px}

<table>
<tr><th>w_x</th><th>w_{px}</th></tr>
<tr><td>➢ Seismic dead load to be used in calculating the distribution of the base shear F_x

$$F_x = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} V \qquad (12.8-11)$$</td><td>➢ Seismic dead load to be used in calculating the diaphragm force F_{px} (Eq. 12.10-1)

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \qquad (12.10-1)$$</td></tr>
<tr><td>➢ It consist of the dead load of the level (floor or roof) <u>plus</u> the exterior tributary walls dead load <u>plus</u> any portion of the FOUR items mentioned above
➢ <u>It includes all walls</u> (parallel and perpendicular) because all walls contribute to the distribution of the base shear.
➢ All <u>foundation weight and half of the first story weight</u> are <u>ignored</u>. This is a common practice NOT Code requirement.</td><td>➢ It consist of the dead load of the diaphragm <u>plus</u> the exterior tributary perpendicular (normal) walls dead load <u>plus</u> any portion of the FOUR items mentioned above
➢ <u>Does not</u> include <u>walls parallel to the direction of analysis</u> because it do not contribute to the load on the diaphragm.
➢ All <u>foundation weight and half of the first story weight</u> are <u>ignored</u>. This is a common practice NOT Code requirement.</td></tr>
<tr><td></td><td></td></tr>
<tr><td>W_1 = Dead load of the floor + Applicable portion(s) of the FOUR items §12.7.2 + (b/2 +h/2) (building perimeter)(unit weight of the wall)</td><td>W_{p1} = Dead load of the floor + Applicable portion(s) of the FOUR items §12.7.2 + (b/2 +h/2) (walls ⊥ direction of analysis)(unit weight of the wall)</td></tr>
<tr><td>W_2 = Dead load of the roof + Applicable portion(s) of the FOUR items §12.7.2 + (h/2) (building perimeter)(unit weight of the wall)</td><td>W_{p2} = Dead load of the roof + Applicable portion(s) of the FOUR items §12.7.2 + (b/2 +h/2) (walls ⊥ direction of analysis)(unit weight of the wall)</td></tr>
</table>

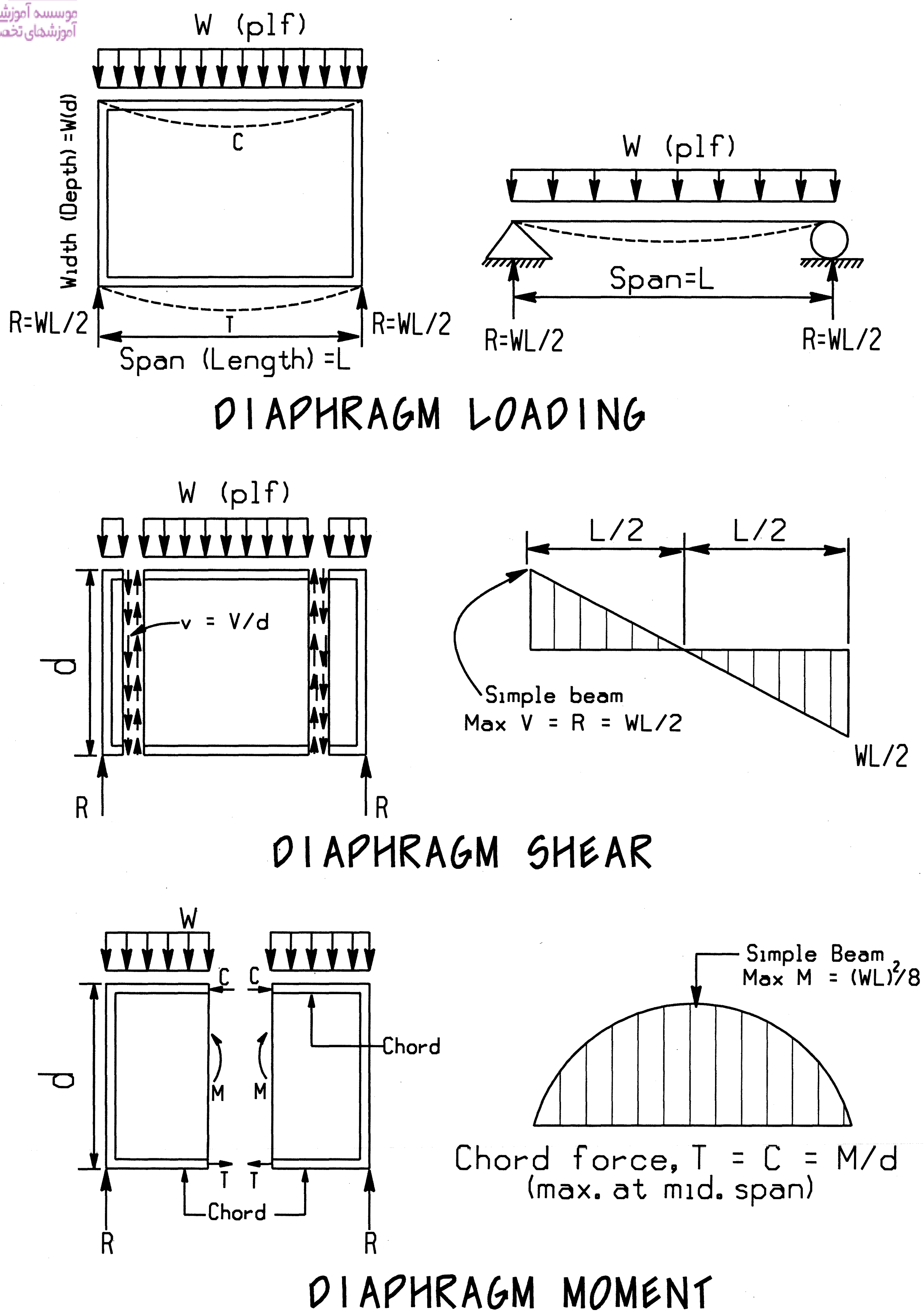


Figure 7-7 Flexible Diaphragm Loading

7.4 FORCES IN DIAPHRAGMS

For flexible diaphragms, loading, shear and moment diagrams are shown in Figure 7-5. For a diaphragm loaded with a uniform distributed load, the maximum shear (WL/2) occurs at the end supports while the maximum bending moment ($WL^2/8$) occurs at the center at L/2. Two forces will be considered: chord force and drag force. These two forces are applicable to both flexible and rigid diaphragms.

I- Chord Force:

The diaphragm chord is defined as "diaphragm boundary element perpendicular to the applied load that is assumed to take axial stresses due to the diaphragm moment. Because of the type of loads and hence stresses on those boundary elements, chords are analogous to the flanges of a steel beam and resist the compression and tension developed in the boundary member due to bending (flexure) of the diaphragm.

Therefore, the chord force is the tension (one side of the diaphragm) or compression (the opposite side of the diaphragm) force in the diaphragm boundary member perpendicular to the direction of lateral load under consideration.

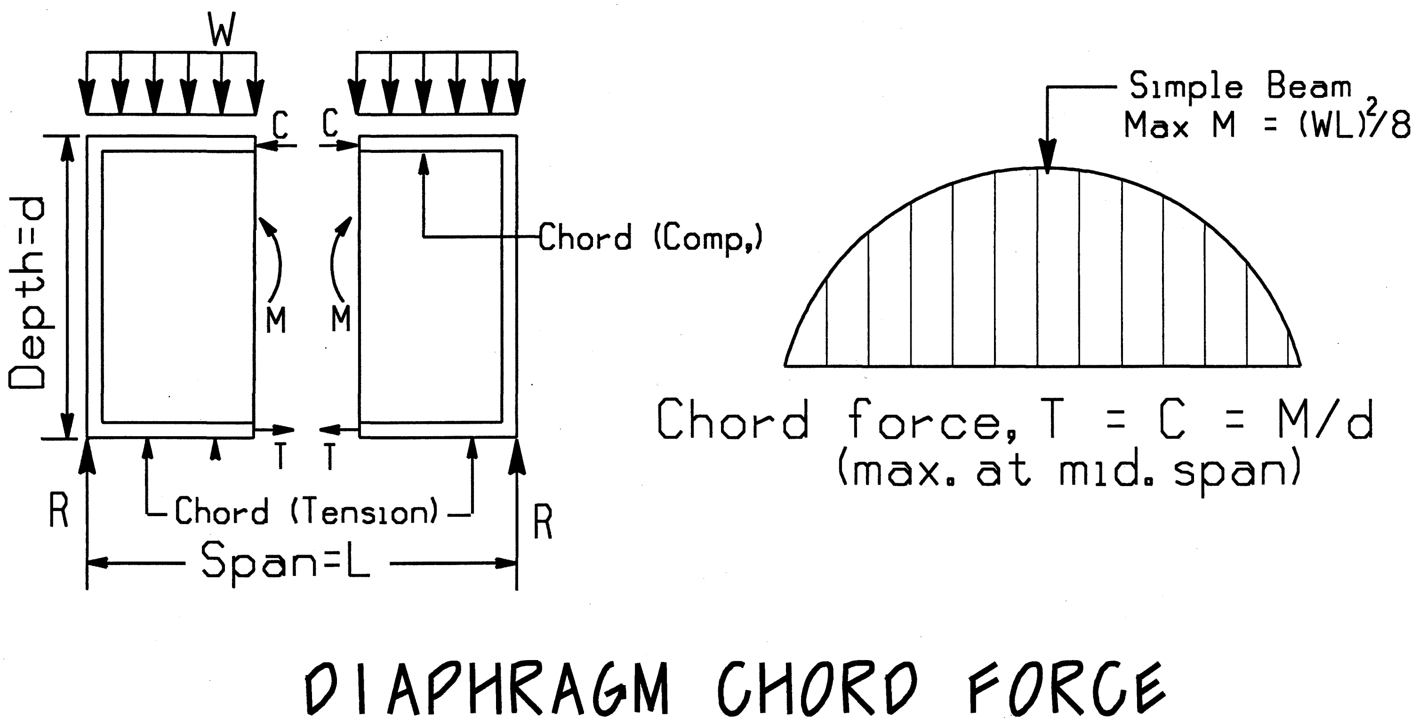


Figure 7- 8 Diaphragm Chord Force

The chord force (either tension or compression) at any distance along the chord is given by the following equation:

$$Chrod\ Force = \frac{Bending\ Moment\ \ Along\ the\ Chord}{Depth\ of\ the\ Diaphragm} = \frac{M}{d} \qquad (7-1)$$

The bending moment along the diaphragm chord changes from zero at both ends to the maximum at the mid-span of the diaphragm where the moment is ($WL^2/8$). The distribution of the moment along the chord is parabolic (2nd degree) and therefore, the chord force will be also nonlinear.

$$(Chrod\ Force)_{\max} = \frac{Max.\ Bending\ Moment}{Depth\ of\ the\ Diaphragm} = \frac{M_{\max}}{d} = \frac{wL^2/8}{d} = \frac{wL^2}{8d} \qquad (7-2)$$

II- Collector (Drag) Force:

Drag Strut (Collector, Tie, Diaphragm Strut) is defined as "a diaphragm or shear wall boundary element parallel to the applied load that collects and transfers diaphragm shear forces to the vertical force-resisting elements or distributes forces within the diaphragm or shear wall."

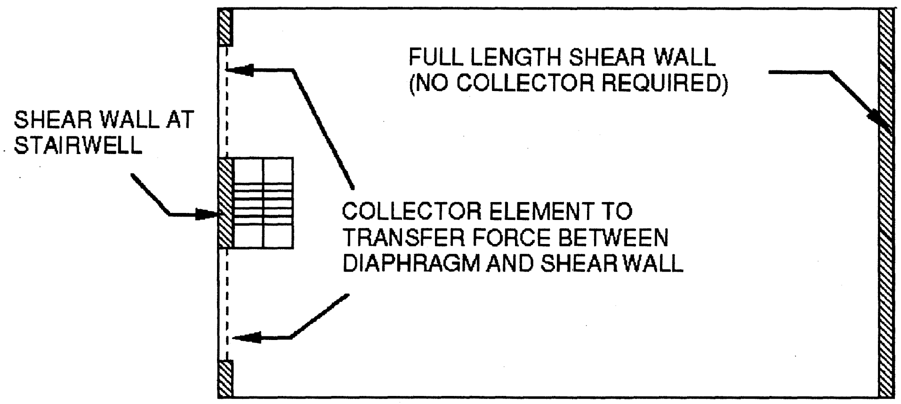


Figure 7- 9 Collectors
(FIGURE 12.10-1 COLLECTORS – ASCE 7-05)

§12.10.2 **Collector Elements**. Collector elements shall be provided that are capable of transferring the seismic forces originating in other portions of the structure to the element providing the resistance to those forces.

§12.10.2.1 **Collector Elements Requiring Load Combinations with Overstrength Factor for Seismic Design Categories C through F**. In structures assigned to Seismic Design Category C, D, E, or F, collector elements (see Fig. 12.10-1), splices, and their connections to resisting elements shall resist the load combinations with overstrength of Section 12.4.3.2.

§1605.4 Special seismic load combinations. For both allowable stress design and strength design methods where specifically required by Section 1605.1 or by Chapters 18 through 23, elements and components shall be designed to resist the forces calculated using Equation 16-22 when the effects of the seismic ground motion are additive to gravity forces and those calculated using Equation 16-23 when the effects of the seismic ground motion counteract gravity forces.

$$1.2D + f_1 L + E_m \qquad \text{(Equation 16-22)}$$

$$0.9D + E_m \qquad \text{(Equation 16-23)}$$

where:

E_m = The maximum effect of horizontal and vertical forces as set forth in Section 12.4.3 of ASCE 7.

= $\Omega_0 \times Q_E$ (Q_E = the drag force due to the F_{px} assuming $\rho = 1$)

f_1 = 1 for floors in places of public assembly, for live loads in excess of 100 psf (4.79 kN/m^2) and for garage live load, or

= 0.5 for other live loads

EXCEPTION: In structures or portions thereof braced entirely by light-frame shear walls, collector elements, splices ,and connections to resisting elements need only be designed to resist forces in accordance with Section 12.10.1.1.

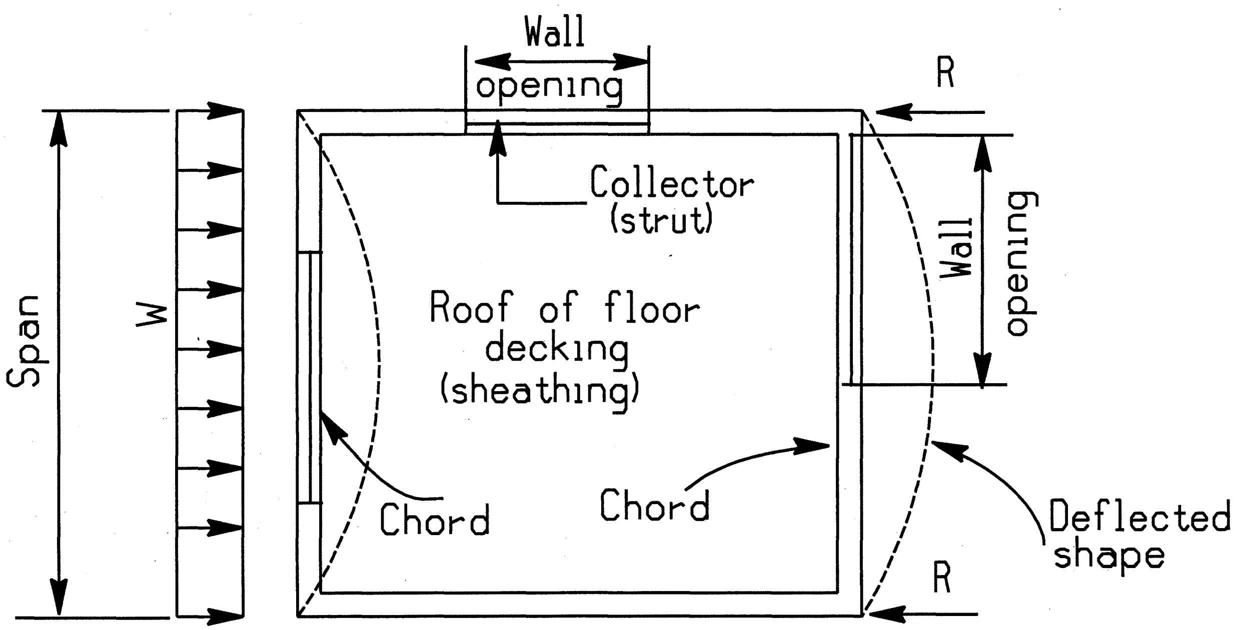


Figure 7- 10 Collector (Strut) Over Wall Opening

The drag force is a tension or compression force in the diaphragm boundary members parallel to the direction of the seismic lateral force.

The diaphragm shear (shear along the diaphragm) is given by the following Equation using Figure 7-7:

$$\tau_{diaphragm} = \frac{F_{total}/2}{d} = \frac{F_{total}}{2d} = \frac{F_{diaphragm}}{2d} \qquad (7-3)$$

Where:

F_{total} = Total load in the direction of analysis = $W \cdot L$

L = Span (length) of the diaphragm

d = Depth (dimension parallel to the load) of the diaphragm