THE REPUBLIC OF THE UNION OF MYANMAR

MINISTRY OF CONSTRUCTION

MYANMAR NATIONAL BUILDING CODE 2025

PART 3 - STRUCTURAL DESIGN

PART 4 - SOIL AND FOUNDATION

THE REPUBLIC OF THE UNION OF MYANMAR

MINISTRY OF CONSTRUCTION

MYANMAR NATIONAL BUILDING CODE

2025

PART 3 - STRUCTURAL DESIGN

PART 4 - SOIL AND FOUNDATION

JUNE, 2025

MINISTRY OF CONSTRUCTION
FOREWORD

In 2008, Cyclone Nargis, which hit Myanmar, caused extensive damage and necessitated the development of a Building Code to ensure the safety of buildings. The Ministry of Construction formed Technical Working Groups, in collaboration with United Nations Human Settlements Programme (UN-Habitat), the Myanmar Engineering Society, the Association of Myanmar Architects, related ministries, technological universities, and government organizations to develop the Myanmar National Building Code - 2012 (Provisional) (English Version) and distributed it to government departments and organizations.

In 2016, comments and advices sent from government departments and organizations were further incorporated and the directive was issued to follow Myanmar National Building Code - 2016 (English Version) in carrying out construction activities.

In 2020, the Technical Working Groups revised and modified the Building Code in accordance with the updated international standards. The Ministry of Construction distributed Myanmar National Building Code - 2020 [English and Myanmar Version] to the Union-level organizations, Union Ministries, Region and State governments and relevant organization to refer and follow them, and further issued the directive to the Myanmar Engineering Council, the Myanmar Architectural Council and the High-Rise and Public Building Projects Committee to follow them starting from 1st November, 2020.

In order to publish the Myanmar National Building Code (Updated Version) in line with the improved international standards, the Ministry of Construction has formed the Myanmar National Building Code Implementation Steering Committee, Implementation Working Committee, Drafting Sub-committee and Technical Working Groups (TWGs) in 2024. The publication of the Myanmar National Building Code – 2025 was made possible by joint efforts of the Ministry of Construction, the Ministry of Science and Technology and other related ministries, the Myanmar Engineering Council, the Myanmar Architect Council, the High-Rise and Public Building Projects Committee, the Federation of Myanmar Engineering Societies and its partner organizations, namely the Myanmar Earthquake Committee, the Myanmar Society of Civil Engineers, the Myanmar Society of Mechanical Engineers, the Myanmar Society of Welding Engineering, the Myanmar Green Building Society, the Myanmar International Consulting Engineers Group and the Environmental Conservation Consulting Engineers Association.

Following the Mandalay earthquake struck on 28th March, 2025, the Myanmar Earthquake Committee and the Myanmar Geoscience Association have undertaken the necessary preparations to build earthquake-resistant buildings.

Therefore, the Ministry of Construction acknowledged the contributions of the Ministries, the Myanmar Engineering Council, the Myanmar Architect Council, the High-rise and Public Building Projects Committee, the Federation of Myanmar Engineering Societies and partner organizations, the Myanmar Earthquake Committee, the Myanmar Geoscience Association and to those who have provided advice and assistance from various sectors to develop and publish the Myanmar National Building Code - 2025.

Ministry of Construction

MYANMAR NATIONAL BUILDING CODE 2025

PART 3
STRUCTURAL DESIGN

MYANMAR NATIONAL BUILDING CODE 2025

PART - 3 STRUCTURAL DESIGN

CONTENTS

SECTION 3.1
GENERAL

3.1.1 Definitions and Notations

3.1.1.1 Definitions

The following words and terms shall, for the purposes of this PART, have the meanings shown herein.

ALLOWABLE STRESS DESIGN: A method of proportioning structural members, such that elastically computed stresses produced in the members by nominal loads do not exceed specified allowable stresses (also called ―working stress design).

BALCONY, EXTERIOR: An exterior floor projecting from and supported by a structure without additional independent supports.

DEAD LOADS: The weight of materials of construction incorporated into the building, including but not limited to walls, floors, roofs, ceilings, stairways, built-in partitions, finishes, cladding and other similarly incorporated architectural and structural items, and the weight of fixed service equipment, such as cranes, plumbing stacks and risers, electrical feeders, heating, ventilating and air-conditioning systems and fire sprinkler systems.

DECK: An exterior floor supported on at least two opposing sides by an adjacent structure, and/or posts, piers or other independent supports.

DESIGN STRENGTH: The product of the nominal strength and a resistance factor (or strength reduction factor).

DIAPHRAGM: A horizontal or sloped system acting to transmit lateral forces to the vertical-resisting elements. When the term ―diaphragm is used, it shall include horizontal bracing systems.

DIAPHRAGM, BLOCKED: In light-frame construction, a diaphragm in which all sheathing edges not occurring on a framing member are supported on and fastened to blocking.

DIAPHRAGM BOUNDARY: In light-frame construction, a location where shear is transferred into or out of the diaphragm sheathing. 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.

DIAPHRAGM, FLEXIBLE: A diaphragm is flexible for the purpose of distribution of storey shear and torsional moment where so indicated in Section 12.3.1.1 of ASCE 7-05, as modified in Section 3.4.2.7.1 of this PART.

DIAPHRAGM, RIGID: A diaphragm is rigid for the purpose of distribution of storey shear and torsional moment when the lateral deformation of the diaphragm is less than or equal to two times the average storey drift

DURATION OF LOAD: The period of continuous application of a given load, or the aggregate of periods of intermittent applications of the same load.

ESSENTIAL FACILITIES: Buildings and other structures that are intended to remain operational in the event of extreme environmental loading from flood, wind or earthquakes.

FABRIC PARTITIONS: A partition consisting of a finished surface made of fabric, without a continuous rigid backing, that is directly attached to a framing system in which the vertical framing members are spaced greater than 4 feet (1219 mm) on center.

FACTORED LOAD: The product of a nominal load and a load factor.

IMPACT LOAD: The load resulting from moving machinery, elevators, crane ways, vehicles and other similar forces and kinetic loads, pressure and possible surcharge from fixed or moving loads.

LIMIT STATE: A condition beyond which a structure or member becomes unfit for service and is judged to be no longer useful for its intended function (serviceability limit state) or to be unsafe (strength limit state).

LIVE LOADS: Those loads produced by the use and occupancy of the building or other structure and do not include construction or environmental loads such as wind load, rain load, earthquake load, flood load or dead load.

LIVE LOADS (ROOF): Those loads produced (1) during maintenance by workers, equipment and materials; and (2) during the life of the structure by movable objects such as planters and by people.

LOAD AND RESISTANCE FACTOR DESIGN (LRFD): A method of proportioning structural members and their connections using load and resistance factors such that no applicable limit state is reached when the structure is subjected to appropriate load combinations. The term ―LRFD is used in the design of steel and timber structures.

LOAD EFFECTS: Forces and deformations produced in structural members by the applied loads.

LOAD FACTOR: A factor that accounts for deviations of the actual load from the nominal load, for uncertainties in the analysis that transforms the load into a load effect and for the probability that more than one extreme load will occur simultaneously.

LOADS: Forces or other actions that result from the weight of building materials, occupants and their possessions, environmental effects, differential movement and restrained dimensional changes. Permanent loads are those loads in which variations over time are rare or of small magnitude, such as dead loads. All other loads are variable loads (see also ―Nominal loads).

NOMINAL LOADS: The magnitudes of the loads specified in this PART (dead, live, soil, rain, wind, and earthquake).

OCCUPANCY CATEGORY: A category used to determine structural requirements based on occupancy.

OTHER STRUCTURES: Structures, other than buildings, for which loads are specified in this section.

PANEL (PART OF A STRUCTURE): The section of a floor, wall or roof comprised between the supporting frame of two adjacent rows of columns and girders or column bands of floor or roof construction.

RESISTANCE FACTOR: A factor that accounts for deviations of the actual strength from the nominal strength and the manner and consequences of failure (also called ―strength reduction factor).

STRENGTH, NOMINAL: The capacity of a structure or member to resist the effects of loads, as determined by computations using specified material strengths and dimensions and equations derived from accepted principles of structural mechanics or by field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions.

STRENGTH, REQUIRED: Strength of a member, cross section or connection required to resist factored loads or related internal moments and forces in such combinations as stipulated by these provisions.

STRENGTH DESIGN: A method of proportioning structural members such that the computed forces produced in the members by factored loads do not exceed the member design strength also called load and resistance factor design (LRFD). The term —strength design is used in the design of concrete and masonry structural elements.

VEHICLE BARRIER SYSTEM: A system of building components near open sides of a garage floor or ramp or building walls that act as restraints for vehicles.

3.1.1.2 Notation

D	=	Dead load
E	=	Combined effect of horizontal and vertical earthquake induced forces as defined in Section 12.4.2 of ASCE 7-05
E_m	=	Maximum seismic load effect of horizontal and vertical seismic forces as set forth in Section 12.4.3 of ASCE 7-05
F	=	Load due to fluids with well-defined pressures and maximum heights
F_a	=	Flood Load
H	=	Load due to lateral earth pressures, ground water pressure or pressure of bulk materials
L	=	Live load, except roof live load, including any permitted live load reduction
L_r	=	Roof live load including any permitted live load reduction
R	=	Rain load
T	=	Self-straining force arising from contraction or expansion resulting from temperature change, shrinkage, moisture change, creep in component materials, movement due to differential settlement or combinations thereof
W	=	Load due to wind pressure

3.1.2 Construction Documents

3.1.2.1 General

Construction documents shall show the size, section and relative locations of structural members with floor levels, column centers and offsets dimensioned. The design loads and other information pertinent to the structural design required by Sections 3.1.2.1.1 through 3.1.2.1.6 shall be indicated on the construction documents.

3.1.2.1.1 Floor Live Load

The uniformly distributed, concentrated and impact floor live load (if any) used in the design shall be indicated in the design document. Use of floor live load reduction in accordance with Section 3.2.3.9 is permitted in the design.

3.1.2.1.2 Roof Live Load

The roof live load used in the design shall be indicated for roof areas (section 3.2.3.11).

3.1.2.1.3 Wind Design Data

The following information related to wind loads shall be shown, regardless of whether wind loads govern the design of the lateral-force-resisting system of the building:

1. Basic wind speed (3-second gust), miles per hour
2. Wind Importance factor, I, and occupancy category
3. Wind exposure parameters and wind coefficients
4. The applicable internal pressure coefficient
5. Components and cladding. The design wind pressures in terms of psf (kN/m^2) to be used for the design of exterior component and cladding materials not specifically designed by the registered design professional.

3.1.2.1.4 Earthquake Design Data

The following information related to seismic loads shall be shown, regardless of whether seismic loads govern the design of the lateral force resisting system of the building:

1. Seismic importance factor, I, and occupancy category.
2. Mapped spectral response accelerations, S_S and S_1.
3. Site class.
4. Spectral response coefficients, S_{DS} and S_{D1}.
5. Seismic design category (SDC).
6. Basic seismic-force-resisting system(s).
7. Design base shear.
8. Seismic response coefficient(s), C_s.
9. Response modification factor(s), R.
10. Analysis procedure used.

3.1.2.1.5 Special Loads

Special loads that are applicable to the design of the building, structure or portions thereof shall be indicated in the design document.

3.1.2.1.6 Material Properties

The properties of the materials used in the design calculations shall be mentioned in the design document.

3.1.2.1.7 Soil and Foundation Data

The relevant soil and foundation data as used in the design calculations shall be mentioned in the design document.

3.1.2.2 Systems and Components Requiring Special Inspections for Seismic Resistance

Design and construction documents or specifications shall be prepared for those systems and components requiring special inspection for seismic resistance (if any).

3.1.2.3 Restrictions on Loading

It shall be unlawful to place, or cause or permit to be placed, on any floor or roof of a building, structure or portion thereof, a load greater than is permitted by the provisions of this PART, unless approved by the authority having jurisdiction for special situations.

3.1.2.4 Live Loads Posted

Where the live loads for which each floor or portion thereof of a commercial or industrial building is or has been designed to exceed 50 psf (2.40 kN/m), such design live loads shall be conspicuously posted by the owner in that part of each story in which they apply, using durable signs. It shall be unlawful to remove or deface such notices.

3.1.2.5 Occupancy Permits for Changed Loads

Occupancy permits for buildings hereafter erected shall not be issued until the floor load signs, required by Section 3.1.2.4, have been installed.

3.1.3 General Design Requirements

3.1.3.1. General

Buildings, and parts thereof, shall be designed and constructed in accordance with strength design, load and resistance factor design, allowable stress design, empirical design or conventional construction methods, as permitted by the applicable material sections of this PART. Analysis shall be carried out by following the guidelines of Section 3.1.3.4 and, where relevant, by using the methods permitted by this PART.

3.1.3.2 Strength

Buildings, and parts thereof, shall be designed and constructed to support safely the factored loads in load combinations defined in this PART without exceeding the appropriate strength limit states for the materials of construction.

Alternatively, buildings, and parts thereof, shall be designed and constructed to support safely the nominal loads in load combinations defined in this PART without exceeding the appropriate specified allowable stresses for the materials of construction.

Loads and forces for occupancies or uses not covered in this PART shall be subject to the approval of the building official.

3.1.3.3 Serviceability

Structural systems and members thereof shall be designed to have adequate stiffness to limit deflections and lateral drift. See Section 12.12.1 of ASCE7-05 for drift limits applicable to earthquake loading.

3.1.3.3.1 Deflections

The deflections of structural members shall not exceed the more restrictive of the limitations of Sections 3.1.3.3.2 through 3.1.3.3.5 or that permitted by Table 3.1.1.

TABLE 3.1.1 DEFLECTION LIMITS [a,b,g,h]

CONSTRUCTION	*L*	*W* [e]	*D + L* [c, f]
Roof members:[d]			
Supporting plaster ceiling	*l*/360	*l*/360	*l*/240
Supporting non-plaster ceiling	*l*/240	*l*/240	*l*/180
Not supporting ceiling	*l*/180	*l*/180	*l*/120
Floor members	*l*/360	—	*l*/240
Exterior walls and interior partitions:			
With brittle finishes	—	*l*/240	—
With flexible finishes	—	*l*/120	—
Farm buildings	—	—	*l*/180
Greenhouses	—	—	*l*/120

For SI: 1 foot = 304.8 mm

a. For structural roofing and siding made of formed metal sheets, the total load deflection shall not exceed l/60. For secondary roof structural members supporting formed metal roofing, the live load deflection shall not exceed l/150. For secondary wall members supporting formed metal siding, the design wind load deflection shall not exceed l/90. For roofs, this exception only applies when the metal sheets have no roof covering.
b. Interior partitions not exceeding 6 feet in height and flexible, folding and portable partitions are not governed by the provisions of this section. The deflection criterion for interior partitions is based on the horizontal load defined in section 3.2.3.13.
c. For wood structural members having a moisture content of less than 16 percent at time of installation and used under dry conditions, the deflection resulting from L + 0.5D is permitted to be substituted for the deflection resulting from L + D.
d. The above deflections do not ensure against ponding. Roofs that do not have sufficient slope or camber to assure adequate drainage shall be investigated for ponding. See Section 3.2.4 for rain and ponding requirements and Section 3.2.4.2 for roof drainage requirements.
e. The wind load is permitted to be taken as 0.7 times the ―component and cladding‖ loads for the purpose of determining deflection limits herein.
f. For steel structural members, the dead load shall be taken as zero.
g. For aluminum structural members or aluminum panels used in skylights and sloped glazing framing, roofs or walls of sunroom additions or patio covers, not supporting edge of glass or aluminum sandwich panels, the total load deflection shall not exceed l/120.
h. For cantilever members, l shall be taken as twice the length of the cantilever.

3.1.3.3.2 Reinforced Concrete

The deflection of reinforced concrete structural members shall not exceed that permitted by ACI 318- 08.

3.1.3.3.3 Steel

The deflection of steel structural members shall not exceed that permitted by AISC 360, AISI-NAS, AISI-General, AISI-Truss, ASCE 3, ASCE 8, SJI JG-1.1, SJI K-1.1 or SJI LH/DLH-1.1, as applicable.

3.1.3.3.4 Masonry

The deflection of masonry structural members shall not exceed that permitted by ACI 530/ASCE 5/TMS 402.

3.1.3.3.5 Aluminum

The deflection of aluminum structural members shall not exceed that permitted by AA ADM1.

3.1.3.3.6 Limits

Deflection of structural members over span, l shall not exceed that permitted by Table 3.1.1.

3.1.3.4 Analysis

Load effects on structural members and their connections shall be determined by methods of structural analysis that take into account equilibrium, general stability, geometric compatibility and both short- and long-term material properties.

Members that tend to accumulate residual deformations under repeated service loads shall have included in their analysis the added eccentricities expected to occur during their service life.

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 loads from their point of origin to the load-resisting elements.

The total lateral force shall be distributed to the various vertical elements of the lateral-force-resisting system in proportion to their rigidities, considering the rigidity of the horizontal bracing system or diaphragm. Rigid elements assumed not to be a part of the lateral-force-resisting system are permitted to be incorporated into buildings provided their effect on the action of the system is considered and provided for in the design. Except where diaphragms are flexible, or are permitted to be analyzed as flexible, provisions shall be made for the increased forces induced on resisting elements of the structural system resulting from torsion due to eccentricity between the centre of application of the lateral forces and the centre of rigidity of the lateral-force-resisting system.

Structures shall be designed to resist the overturning effects caused by the lateral forces specified in this PART if it is required to consider lateral loads. See Section 3.3 for wind loads, Section 3.2.2 for lateral soil loads and hydrostatic pressures and Section 3.4 for earthquake loads.

3.1.3.5 Occupancy Category

Buildings shall be assigned an occupancy category in accordance with Table 3.1.2.

3.1.3.5.1 Multiple Occupancies

Where a structure is occupied by two or more occupancies not included in the same occupancy category, the structure shall be assigned the classification of the highest occupancy category corresponding to the various occupancies. Where structures have two or more portions that are structurally separated, each portion shall be separately classified. Where a separated portion of a structure provides required access to, required egress from or shares life safety components with another portion having a higher occupancy category, both portions shall be assigned to the higher occupancy category.

3.1.3.6 In-Situ Load Tests

The building official is authorized to require an engineering analysis or a strength test or a load test, or any combination, of any construction whenever there is reason to question the safety of the construction for the intended occupancy.

3.1.3.7 Preconstruction Load Tests

Materials and methods of construction that are not capable of being designed by approved engineering analysis or that do not comply with the applicable material design standards listed shall be load tested or tested for strength and deformation characteristics.

3.1.3.8 Anchorage

3.1.3.8.1 General

Anchorage of the roof to walls and columns, and of walls and columns to foundations, shall be provided to resist the uplift and sliding forces that result from the application of the prescribed loads.

3.1.3.8.2 Concrete and Masonry Walls

Concrete and masonry walls shall be anchored to floors, roofs and other structural elements that provide lateral support for the wall. Such anchorage shall provide a positive direct connection capable of resisting the horizontal forces specified in this part but not less than a minimum strength design horizontal force of 280 plf (4.10 kN/m) of wall, substituted for "*E*"in the load combinations of Section 3.2.1.2 or 3.2.1.3. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 feet (1219 mm). Required anchors in masonry walls of hollow units or cavity walls shall be embedded in a reinforced grouted structural element of the wall. See Sections 3.3 for wind design requirements and see Section 3.4 for seismic design requirements.

TABLE 3.1.2 OCCUPANCY CATEGORY OF BUILDINGS AND OTHER STRUCTURES

Category	Occupancy	Nature of Occupancy
I	Temporary	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	Low	Buildings and other structures except those listed in Occupancy Categories I, III and IV.
III	High	Buildings and other structures, the failure of which could pose a substantial risk to human life, 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. • Healthcare 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	Essential	Buildings and other structures designated as essential facilities, including but not limited to: • Hospitals and other health care facilities having surgery or emergency treatment facilities. • Fire, rescue and police stations and emergency vehicle garages. • Designated earthquakes, hurricane or other emergency shelters. • Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response. • Power generating stations and other public utility facilities required as emergency backup facilities for Occupancy Category IV structures. • Structures containing highly toxic materials. • 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.

3.1.3.9 Decks

Where supported by attachment to an exterior wall, decks shall be positively anchored to the primary structure and designed for both vertical and lateral loads as applicable. Such attachment shall not be accomplished by the use of toenails or nails subject to withdrawal. Where positive connection to the primary building structure cannot be verified during inspection, decks shall be self-supporting.

3.1.3.10 Counteracting Structural Actions

Structural members, systems, components and cladding shall be designed to resist forces due to earthquake and wind, with consideration of overturning, sliding, and uplift. Continuous load paths shall be provided for transmitting these forces to the foundation. Where sliding is used to isolate the elements, the effects of friction between sliding elements shall be included as a force.

3.1.3.11 Wind and Seismic Detailing

Where required by the authority department, lateral-force-resisting systems shall meet seismic detailing requirements and limitations prescribed in this PART and ASCE 7-05, excluding Chapter 14 and Appendix 11A, even when wind code prescribed load effects are greater than seismic load effects.

SECTION 3.2
LOAD COMBINATIONS AND LOADS

3.2.1 Load Combinations

3.2.1.1 General

Buildings and other structures and portions thereof, shall be designed using the provisions of either Section 3.2.1.2 or 3.2.1.3. Either Section 3.2.1.2 or 3.2.1.3 shall be used exclusively for proportioning elements of a particular construction material throughout the structure. Each load combination shall also be investigated with one or more of the variable loads set to zero.

3.2.1.2 Combining Factored Loads Using Strength Design or Load and Resistance Factor Design

3.2.1.2.1 Applicability

The load combinations and load factors given in Section 3.2.1.2.2 shall be used only in those cases in which they are specifically authorized by the applicable material design standard. Otherwise, the provisions of the applicable material design standard shall be used.

3.2.1.2.2 Basic Load Combinations

Structures, components, and foundations shall be designed so that their design strength equals or exceeds the most critical effects of the factored loads in the following combinations:

1.	$1.4\,(D + F)$	*Eq. (3.2.1)*
2.	$1.2\,(D + F + T) + 1.6\,(L + H) + 0.5\,(L_r \text{ or } R)$	*Eq. (3.2.2)*
3.	$1.2\,D + 1.6\,(Lr \text{ or } R) + (L \text{ or } 0.8\,W)$	*Eq. (3.2.3)*
4.	$1.2\,D + 1.6\,W + L + 0.5\,(L_r \text{ or } R)$	*Eq. (3.2.4)*
5.	$1.2\,D + 1.0\,E + L$	*Eq. (3.2.5)*
6.	$0.9\,D + 1.6\,W + 1.6\,H$	*Eq. (3.2.6)*
7.	$0.9\,D + 1.0\,E + 1.6\,H$	*Eq. (3.2.7)*

EXCEPTIONS:

1. The load factor on L in combinations (3), (4), and (5) is equal to 0.5 for all occupancies in which L_0 in Table 3.2.2 is less than or equal to 100 psf, with the exception of garages or areas occupied as places of public assembly.
2. The load factor on H shall be set equal to zero in combinations (6) and (7) if the structural action due to H counteracts that due to W or E. Where lateral earth pressure provides resistance to structural actions from other forces, it shall not be included in H but shall be included in the design resistance.

Each relevant strength limit state shall be investigated. Effects of one or more loads not acting shall be investigated. The most unfavorable effects from both wind and earthquake loads shall be investigated, where appropriate, but they need not be considered to act simultaneously.

As an exception, where other factored load combinations are specifically required by the provisions of this PART, such combinations shall take precedence.

3.2.1.2.3 Load Combinations Including Flood Load

When a structure is located in a flood zone (Section 3.2.5.3.1), the following load combinations shall be considered:

1. In V-Zones or Coastal A-Zones, 1.6W in combinations (4) and (6) shall be replaced by 1.6 W + 2.0 F_a.
2. In non-coastal A-Zones, 1.6W in combinations (4) and (6) shall be replaced by 0.8W + 1.0 F_a.

3.2.1.3 Combining Nominal Loads Using Allowable Stress Design or Working Stress Design

3.2.1.3.1 Basic Load Combinations

Loads listed herein shall be considered to act in the following combinations; whichever produces the most unfavorable effect in the building, foundation, or structural member being considered. Effects of one or more loads not acting shall be considered.

1.	$D + F$	*Eq. (3.2.8)*
2.	$D + H + F + L + T$	*Eq. (3.2.9)*
3.	$D + H + F + (L_r \text{ or } R)$	*Eq. (3.2.10)*
4.	$D + H + F + 0.75\,(L + T) + 0.75\,(L_r \text{ or } R)$	*Eq. (3.2.11)*
5.	$D + H + F + (W \text{ or } 0.7\,E)$	*Eq. (3.2.12)*
6.	$D + H + F + 0.75\,(W \text{ or } 0.7\,E) + 0.75\,L + 0.75\,(L_r \text{ or } R)$	*Eq. (3.2.13)*
7.	$0.6\,D + W + H$	*Eq. (3.2.14)*
8	$0.6\,D + 0.7\,E + H$	*Eq. (3.2.15)*

The most unfavorable effects from both wind and earthquake loads shall be considered, where appropriate, but they need not be assumed to act simultaneously.

3.2.1.3.2 Stress Increases

Increases in allowable stress shall not be used with the loads or load combinations given in Section 3.2.1.3.1 unless it can be demonstrated that such an increase is justified by structural behavior caused by rate or duration of load.

3.2.1.3.3 Load Combinations Including Flood Load

When a structure is located in a flood zone, the following load combinations shall be considered:

1. In V-Zones or Coastal A-Zones (Section 3.2.5.3. l), 1.5 Fa shall be added to other loads in combinations (5), (6), and (7), and E shall be set equal to zero in (5) and (6).
2. In non-coastal A-Zones, 0.75 Fa shall be added to combinations (5), (6), and (7), and E shall be set equal to zero in (5) and (6).

3.2.1.4 Load Combinations for Extraordinary Events

Where required by the applicable code, standard, or the authority having jurisdiction, strength and stability shall be checked to ensure that structures are capable of withstanding the effects of extraordinary (i.e., low-probability) events, such as fires, explosions, and vehicular impact.

3.2.1.5 Special Seismic Load Combinations

For both strength and allowable stress design methods where specifically required by relevant material design standards, elements and components shall be designed to resist the forces calculated using Eq. (3.2.16) when the effects of the seismic ground motion are additive to gravity forces and those calculated using Eq. (3.2.17) when the effects of the seismic ground motion counteract gravity forces.

1.	$1.2\,D + f_1 L + E_m$	*Eq. (3.2.16)*

2. $0.9\,D + E_m$ *Eq. (3.2.17)*

where

Em = the maximum effect of horizontal and vertical forces as set forth in Section12.4.3 of ASCE 7-05.

The load factor f_l for L in combination Eq. (3.2.16) is equal to 0.5 for all occupancies when live load is less than or equal to 100 psf (4.79 kN/m^2), with the exception of garages or areas of public assembly. Otherwise, f_l is equal to 1.

3.2.2 Dead Loads, Soil Loads and Hydrostatic Pressure

3.2.2.1 Dead Loads

3.2.2.1.1 Definition

Dead loads consist of the weight of all materials of construction incorporated into the building including, but not limited to, walls, floors, roofs, ceilings, stair ways, built-in partitions, finishes, cladding, and other similarly incorporated architectural and structural items, and fixed service equipment including the weight of cranes.

3.2.2.1.2 Weights of Materials and Constructions

In determining dead loads for purposes of design, the actual weights of materials and constructions shall be used provided that in the absence of definite information, values approved by the authority having jurisdiction shall be used.

3.2.2.1.3 Weight of Fixed Service Equipment

In determining dead loads for purposes of design, the weight of fixed service equipment, such as plumbing stacks and risers, electrical feeders, and heating, ventilating, and air conditioning systems shall be included.

3.2.2.2 Soil Loads and Hydrostatic Pressure

3.2.2.2.1 Lateral Pressures

In the design of structures below grade, provision shall be made for the lateral pressure of adjacent soil. If soil loads are not given in a soil investigation report approved by the authority having jurisdiction, then the soil loads specified in Table 3.2.1 shall be used as the minimum design lateral loads. Due allowance shall be made for possible surcharge from fixed or moving loads. When a portion or the whole of the adjacent soil is below a free-water surface, computations shall be based upon the weight of the soil diminished by buoyancy, plus full hydrostatic pressure.

The lateral pressure shall be increased if soils with expansion potential are present at the site as determined by a geotechnical investigation.

Basement walls and other walls in which horizontal movement is restricted at the top shall be designed for at-rest pressure. Retaining walls free to move and rotate at the top are permitted to be designed for active pressure. As an exception, basement walls extending not more than 8 feet (2438 mm) below grade and supporting flexible floor system shall be permitted to be designed for active pressure.

TABLE 3.2.1 DESIGN LATERAL SOIL LOAD

DESCRIPTION OF BACKFILL MATERIAL[c]	UNIFIED SOIL CLASSIFICATION	DESIGN LATERAL SOIL LOAD[a] (psf per foot of depth)	
		Active pressure	At-rest pressure
Well-graded, clean gravels; gravel–sand mixes	GW	30	60
Poorly graded clean gravels; gravel–sand mixes	GP	30	60
Silty gravels, poorly graded gravel–sand mixes	GM	40	60
Clayey gravels, poorly graded gravel-and-clay mixes	GC	45	60
Well-graded, clean sands; gravelly–sand mixes	SW	30	60
Poorly graded clean sands; sand–gravel mixes	SP	30	60
Silty sands, poorly graded sand–silt mixes	SM	45	60
Sand–silt clay mix with plastic fines	SM–SC	45	100
Clayey sands, poorly graded sand–clay mixes	SC	60	100
Inorganic silts and clayey silts	ML	45	100
Mixture of inorganic silt and clay	ML–CL	60	100
Inorganic clays of low to medium plasticity	CL	60	100
Organic silts and silt–clays, low plasticity	OL	Note b	Note b
Inorganic clayey silts, elastic silts	MH	Note b	Note b
Inorganic clays of high plasticity	CH	Note b	Note b
Organic clays and silty clays	OH	Note b	Note b

For SI: 1 pound per square foot per foot of length = 0.157 kPa/m, 1 foot = 304.8 mm.

a. Design lateral soil loads are given for moist conditions for the specified soils at their optimum densities. Actual field conditions shall govern. Submerged or saturated soil pressures shall include the weight of the buoyant soil plus the hydrostatic loads.
b. Unsuitable as backfill material.
c. The definition and classification of soil materials shall be in accordance with ASTM D2487.

3.2.2.2.2 Uplift on Floors and Foundations

In the design of basement floors and similar approximately horizontal elements below grade, the upward pressure of water, where applicable, shall be taken as the full hydrostatic pressure applied over the entire area. The hydrostatic load shall be measured from the underside of the construction. Any other upward loads shall be included in the design.

Where expansive soils are present under foundations or slabs-on-ground, the foundations, slabs, and other components shall be designed to tolerate the movement or resist the upward loads caused by the expansive soils, or the expansive soil shall be removed or stabilized around and beneath the structure.

3.2.3 – Live Loads

3.2.3.1 Definitions

The following definitions apply only to the provision of Section 3.2.3.

LIVE LOAD: A load produced by the use and occupancy of the building or other structure that does not include construction or environmental loads, such as wind load, snow load, rain load, earthquake load, flood load, or dead load.

ROOF LIVE LOAD: A load on a roof produced (1) during maintenance by workers, equipment, and materials and (2) during the life of the structure by movable objects, such as planters or other similar small decorative appurtenances that are not occupancy related.

FIXED LADDER: A ladder that is permanently attached to a structure, building, or equipment.

GRAB BAR SYSTEM: A bar provided to support body weight in locations such as toilets, showers, and tub enclosures.

GUARDRAIL SYSTEM: A system of building components near open sides of an elevated surface for the purpose of minimizing the possibility of a fall from the elevated surface by people, equipment, or material.

HANDRAIL: A rail grasped by hand for guidance and support. A handrail assembly includes the handrail, supporting attachments, and structures.

VEHICLE BARRIER SYSTEM: A system of building components near open sides of a garage floor or ramp or building walls that act as restraints for vehicles.

3.2.3.2 Uniformly Distributed Loads

3.2.3.2.1 Required Live Loads

The live loads used in the design of buildings and other structures shall be the maximum loads expected by the intended use or occupancy but shall in no case be less than the minimum uniformly distributed unit loads required by Table 3.2.2.

TABLE 3.2.2 MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_0 , AND MINIMUM CONCENTRATED LIVE LOAD

OCCUPANCY OR USE	UNIFORM (PSF)	CONCENTRATED (LBS)
1. Apartments (see residential)	—	—
2. Access floor systems		
Office use	50	2,000
Computer use	100	2,000
3. Armories and drill rooms	150	—
4. Assembly areas and theaters		
Fixed seats (fastened to floor)	60	
Follow spot, projections and control rooms	50	
Lobbies	100	—
Movable seats	100	
Stages and platforms	125	
5. Balconies	100	
On one- and two-family residences only, and not exceeding 100 sqft	60	—
6. Bowling alleys	75	—
7. Catwalks	40	300
8. Dance halls and ballrooms	100	—
9. Decks	Same as occupancy served [g]	—
10. Dining rooms and restaurants	100	—
11. Dwellings (see residential)	—	—
12. Cornices	60	—
13. Corridors, except as otherwise indicated	100	—
14. Elevator machine room grating (on area of 4 in^2)	—	300
15. Finish light floor plate construction (on area of 1 in^2)	—	200
16. Fire escapes	100	
On single-family dwellings only	40	—
17. Garages (passenger vehicles only)	40	Note [a]
Trucks and buses	See Section 3.2.3.4	
18. Grandstands (see stadium and arena bleachers)	—	—
19. Gymnasiums, main floors and balconies	100	—
20. Handrails, guards and grab bars	See Section 3.2.3.5.1	
21. Hospitals		
Corridors above first floor	80	1,000
Operating rooms, laboratories	60	1,000
Patient rooms	40	1,000
22. Hotels (see residential)	—	—

OCCUPANCY OR USE	UNIFORM (PSF)	CONCEN-TRATED (LBS)
23. Libraries		
Corridors above first floor	80	1,000
Reading rooms	60	1,000
Stack rooms	150[b]	1,000
24. Manufacturing		
Heavy	250	3,000
Light	125	2,000
25. Marquees	75	—
26. Office buildings		
Corridors above first floor	80	2,000
File and computer rooms shall be designed for heavier loads based on anticipated occupancy	—	—
Lobbies and first-floor corridors	100	2,000
Offices	50	2,000
27. Penal institutions		
Cell blocks	40	—
Corridors	100	
28. Residential		
One- and two-family dwellings		
Uninhabitable attics without storage [h]	10	
Uninhabitable attics with limited storage [h,i,j]	20	
Habitable attics and sleeping areas	30	
All other areas except balconies and decks	40	—
Hotels and multiple-family dwellings		
Private rooms and corridors serving them	40	
Public rooms and corridors serving them	100	
29. Reviewing stands, grandstands and bleachers	Note [c]	
30. Roofs		
All roof surfaces subject to maintenance workers		300
Awnings and canopies		
Fabric construction supported by a lightweight rigid skeleton structure	5 Non-reduceable	
All other construction	20	
Ordinary flat, pitched, and curved roofs	20	
Primary roof members, exposed to a work floor		
Single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs Over manufacturing, storage, Warehouse, and repair garages		2,000
All other occupancies		300
Roofs used for other special purposes	Note [k]	Note [k]
Roofs used for promenade purposes	60	
Roofs used for roof gardens or assembly purposes	100	

OCCUPANCY OR USE	UNIFORM (PSF)	CONCEN-TRATED (LBS)
31. Schools		
Classrooms	40	1,000
Corridors above first floor	80	1,000
First-floor corridors	100	1,000
32. Scuttles, skylight ribs and accessible ceilings	—	200
33. Sidewalks, vehicular driveways and yards, subject to trucking	250[d]	8,000[e]
34. Skating rinks	100	—
35. Stadiums and arenas		
Bleachers	100[c]	
Fixed seats (fastened to floor)	60[c]	—
36. Stairs and exits		Note [f]
One- and two-family dwellings	40	
All other	100	
37. Storage warehouses (shall be designed for heavier loads if required for anticipated storage)		
Heavy	250	
Light	125	
38. Stores		
Retail		
First floor	100	1,000
Upper floors	75	1,000
Wholesale, all floors	125	1,000
39. Vehicle barriers	See Section 3.2.3.5.3	
40. Walkways and elevated platforms (other than exit ways)	60	—
41. Yards and terraces, pedestrians	100	—

For SI: 1 inch = 25.4 mm, 1 square inch = 645.16 mm^2, 1 square foot = 0.0929 m^2,
1 pound per square foot = 0.0479 kN/m^2, 1 pound = 0.004448 kN,
1 pound per cubic foot = 16 kg/m^3

a. Floors in garages or portions of buildings used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of Table 3.2.2 or the following concentrated loads: (1) for garages restricted to vehicles accommodating not more than nine passengers, 3,000 pounds acting on an area of 4.5 inches by 4.5 inches; (2) for mechanical parking structures without slab or deck which are used for storing passenger vehicles only, 2,250 pounds per wheel.

b. The loading applies to stack room floors that support non mobile, double-faced library books stacks, subject to the following limitations:
   1. The nominal books stack unit height shall not exceed 90 inches;
   2. The nominal shelf depth shall not exceed 12 inches for each face; and
   3. Parallel rows of double-faced books stacks shall be separated by aisles not less than 36 inches wide.

c. Design in accordance with the ICC Standard on Bleachers, Folding and Telescopic Seating and Grandstands.

d. Other uniform loads in accordance with an approved method which contains provisions for truck loadings shall also be considered where appropriate.

e. The concentrated wheel load shall be applied on an area of 20 square inches.

f. Minimum concentrated load on stair treads (on area of 4 square inches) is 300 pounds
g. See Section 3.1.3.9 for decks attached to exterior walls.
h. Attics without storage are those where the maximum clear height between the joist and rafter is less than 42 inches, or where there are not two or more adjacent trusses with the same web configuration capable of containing a rectangle 42 inches high by 2 feet wide, or greater, located within the plane of the truss. For attics without storage, this live load need not be assumed to act concurrently with any other live load requirements.
i. For attics with limited storage and constructed with trusses, this live load need only be applied to those portions of the bottom chord where there are two or more adjacent trusses with the same web configuration capable of containing a rectangle 42 inches high by 2 feet wide or greater, located within the plane of the truss. The rectangle shall fit between the top of the bottom chord and the bottom of any other truss member, provided that each of the following criteria is met:
    1. The attic area is accessible by a pull-down stairway or framed opening and
    2. The truss shall have a bottom chord pitch less than 2:12.
    3. Bottom chords of trusses shall be designed for the greater of actual imposed dead load or 10 psf, uniformly distributed over the entire span.
j. Attic spaces served by a fixed stair shall be designed to support the minimum live load specified for habitable attics and sleeping rooms.
k. Roofs used for other special purposes shall be designed for appropriate loads as approved by the building official.

3.2.3.2.2 Provision for Partitions

In office buildings or other buildings where partitions will be erected or rearranged, provision for partition weight shall be made, whether or not partitions are shown on the construction documents. Partition load shall not be less than uniformly distributed live load of 15 psf (0.74kN/m^2).

EXCEPTION:

A partition live load is not required where the minimum specified live load exceeds 80 psf (3.83kN/m^2).

3.2.3.3 Concentrated Loads

Floors, roofs, and other similar surfaces shall be designed to support safely the uniformly distributed live loads prescribed in Section 3.2.3.2 or the concentrated load, in pounds or kilo newtons (kN), given in Table 3.2.2, whichever produces the greater load effects. Unless otherwise specified, the indicated concentration shall be assumed to be uniformly distributed over an area 2.5 ft (762 mm) square [6.25 ft^2 (0.58 m^2)] and shall be located so as to produce the maximum load effects in the structural members.

3.2.3.4 Truck and Bus Garages

Minimum live loads for garages having trucks or buses shall be as specified in Table 3.2.3, but shall not be less than 50 psf (2.40 kN/m^2), unless other loads are specifically justified and approved by the building official. Actual loads shall be used where they are greater than the loads specified in the table.

3.2.3.4.1 Truck and Bus Garage Live Load Application

The concentrated load and uniform load shall be uniformly distributed over a 10-foot (3048 mm) width on a line normal to the centre line of the lane placed within a 12-foot-wide (3658 mm) lane. The loads shall be placed within their individual lanes so as to produce the maximum stress in each structural member. Single spans shall be designed for the uniform load in Table 3.2.3 and one simultaneous concentrated load positioned to produce the maximum effect. Multiple spans shall be designed for the uniform load in Table3.2.3 on the spans and two simultaneous concentrated loads in two spans positioned to produce the maximum negative moment effect. Multiple span design loads, for other effects, shall be the same as for single spans.

TABLE 3.2.3 UNIFORM AND CONCENTRATED LOADS

LOADING CLASS[a]	UNIFORM LOAD (pounds/linear foot of lane)	CONCENTRATED LOAD (pounds)[b]	
		For moment design	For shear design
H20-44 and HS20-44	640	18,000	26,000
H15-44 and HS15-44	480	13,500	19,500

For SI: 1 pound per linear foot = 0.01459 kN/m, 1 pound = 0.004448 kN,1 ton = 8.90 kN.

a. An H loading class designates a two-axle truck with a semitrailer. An HS loading class designates a tractor truck with a semitrailer. The numbers following the letter classification indicate the gross weight in tons of the standard truck and the year the loadings were instituted.
b. See Section 3.2.3.4.1 for the loading of multiple spans.

3.2.3.5 Loads on Handrails, Guardrail Systems, Grab Bar Systems, Vehicle Barrier Systems, and Fixed Ladders

3.2.3.5.1 Loads on Handrails and Guardrail Systems

All handrail assemblies and guardrail systems shall be designed to resist a single concentrated load of 200 lb (0.89 kN) applied in any direction at any point along the top and to transfer this load through the supports to the structure.

Further, all handrail assemblies and guardrail systems shall be designed to resist a load of 50 lb/ft (pound-force per linear foot) (0.73 kN/m) applied in any direction at the top and to transfer this load through the supports to the structure. This load need not be assumed to act concurrently with the load specified in the preceding paragraph, and this load need not be considered for the following occupancies:

1. One- and two-family dwellings.
2. Factory, industrial, and storage occupancies, in areas that are not accessible to the public and that serve an occupant load not greater than 50, the minimum load in that are shall be 20 lb/ft (0.29kN/m).

Intermediate rails (all those except the handrail), balusters, and panel fillers shall be designed to withstand a horizontally applied normal load of 50 lb (0.22 kN) on an area not to exceed 1 ft square (305 mm square) including openings and space between rails. Reactions due to this loading are not required to be superimposed with those of either preceding paragraph.

Where handrails and guards are designed using working stress design exclusively for the loads specified in this section, the allowable stress for the members and their attachments are permitted to be increased by one-third.

3.2.3.5.2 Loads on Grab Bar Systems

Grab bar systems shall be designed to resist a single concentrated load of 250 lb (1.11 kN) applied in any direction at any point.

3.2.3.5.3 Loads on Vehicle Barrier Systems

Vehicle barrier systems for passenger cars shall be designed to resist a single load of 6,000 lb (26.70 kN) applied horizontally in any direction to the barrier system, and shall have anchorages or attachments capable of transferring this load to the structure. For design of the system, the load shall be assumed to act at a minimum height of 1 ft 6 in. (460 mm) above the floor or ramp surface on an area not to exceed 1 foot square (305 mm square), and is not required to be assumed to act concurrently with any handrail or guardrail loadings specified in Section 3.2.3.4.1. Garages accommodating trucks and buses shall be designed in accordance with an approved method, which contains provision for traffic railings.

3.2.3.5.4 Loads on Fixed Ladders

The minimum design live load on fixed ladders with rungs shall be a single concentrated load of 300 lb (1.33 kN), and shall be applied at any point to produce the maximum load effect on the element being considered. The number and position of additional concentrated live load units shall be a minimum of 1 unit of 300 lb (1.33 kN) for every 10 ft (3,048 mm) of ladder height.

Where rails of fixed ladders extend above a floor or platform at the top of the ladder, each side rail extension shall be designed to resist a concentrated live load of 100 lb (0.445 kN) in any direction at any height up to the top of the side rail extension. Ship ladders with treads instead of rungs shall have minimum design loads as stairs, defined in Table 3.2.2.

3.2.3.6 Loads Not Specified

For occupancies or uses not designated in Sections 3.2.3.2 or 3.2.3.3, the live load shall be determined in accordance with a method approved by the authority having jurisdiction.

3.2.3.7 Partial Loading

The full intensity of the appropriately reduced live load applied only to a portion of a structure or member shall be accounted for if it produces a more unfavorable effect than the same intensity applied over the full structure or member. Roof live loads are to be distributed as specified in Table 3.2.2.

3.2.3.8 Impact Loads

The live loads specified in Sections 3.2.3.2 shall be assumed to include adequate allowance for ordinary impact conditions. Provision shall be made in the structural design for uses and loads that involve unusual vibration and impact forces.

3.2.3.8.1 Elevators

All elevator loads shall be increased by 100 percent for impact and the structural supports shall be designed within the limits of deflection prescribed by ANSI A17.2 and ANSI/ASME A17.1.

3.2.3.8.2 Machinery

For the purpose of design, the weight of machinery and moving loads shall be increased as follows to allow for impact: (I) elevator machinery, 100 percent; (2) light machinery, shaft- or motor-driven, 20 percent; (3) reciprocating machinery or power-driven units, 50 percent; and (4) hangers for floors or balconies, 33 percent. All percentages shall be increased where specified by the manufacturer.

3.2.3.9 Reduction in Live Loads

Except for roof uniform live loads, all other minimum uniformly distributed live loads, L_0 in Table 3.2.2, may be reduced according to the following provisions.

3.2.3.9.1 General

Subject to the limitations of Sections 3.2.3.9.1.1 through 3.2.3.9.1.4, members for which a value of $K_{LL}A_T$ is 400 ft^2 (37.16 m^2) or more are permitted to be designed for a reduced live load in accordance with the following equation:

$$L = L_0 \left(0.25 + \frac{15}{\sqrt{K_{LL}\, A_T}} \right) \qquad Eq. (3.2.18)$$

In SI:

$$L = L_0 \left(0.25 + \frac{4.57}{\sqrt{K_{LL}\, A_T}} \right)$$

where

L	=	reduced design live load per ft^2 (m^2) of area supported by the member
L_0	=	unreduced design live load per ft^2 (m^2) of area supported by the member (see Table 3.2.2)
K_{LL}	=	live load element factor (see Table 3.2.4)
A_T	=	tributary area in ft^2 (m^2)

L shall not be less than 0.50 L_0 for members supporting one floor and L shall not be less than 0.40 L_0 for members supporting two or more floors.

3.2.3.9.1.1 Heavy Live Loads

Live loads that exceed 100 lb/ft^2 (4.79 kN/m^2) shall not be reduced.

EXCEPTIONS:

1. Live loads for members supporting two or more floors may be reduced by a maximum of 20 percent, but the live load shall not be less than L as calculated in Section 3.2.3.9.1.
2. For uses other than storage, where approved, additional live load reductions shall be permitted where shown by the registered design engineer that a rational approach has been used and that such reduction are warranted.

TABLE 3.2.4 LIVE LOAD ELEMENT FACTOR, K_{LL}

Element	K_{LL}
Interior columns Exterior columns without cantilever slabs	4 4
Edge columns with cantilever slabs	3
Corner columns with cantilever slabs Edge beams without cantilever slabs Interior beams	2 2 2
All other members not identified above including: • Edge beams with cantilever slabs • Cantilever beams • Two-way slabs • Members without provisions for continuous shear transfer normal to their span	1

3.2.3.9.1.2 Passenger Car Garages

The live loads shall not be reduced in passenger car garages.

EXCEPTION:

Live loads for members supporting two or more floors may be reduced by a maximum of 20 percent, but the live load shall not be less than L as calculated in Section 3.2.3.9.1.

3.2.3.9.1.3 Special Occupancies

Live loads of 100 lb/ft^2 (4.79 kN/m^2) or less shall not be reduced in public assembly occupancies.

3.2.3.9.1.4 Special Structural Elements

Live load shall not be reduced for one-way slabs except as permitted in Section 3.2.3.9.1.1. Live loads of 100 psf (4.79 kN/m^2) or less shall not be reduced for roof members except as specified in Section 3.2.3.11.2.

3.2.3.9.2 Alternative Floor Live Load Reduction

As an alternative to Section 3.2.3.9.1, floor live loads are permitted to be reduced in accordance with the following provisions. Such reductions shall apply to slab systems, beams, girders, columns, piers, walls and foundations.

1. A reduction shall not be permitted in group A occupancies. (i.e., Assembly Group A)
2. A reduction shall not be permitted where the live load exceeds 100 psf (4.79 kN/m^2) except that the design live load for members supporting two or more floors is permitted to be reduced by 20 percent.
3. A reduction shall not be permitted in passenger vehicle parking garages except that the live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent.
4. For live loads not exceeding 100 psf (4.79 kN/m^2), the design live load for any structural member supporting 150 square feet (13.94 m^2) or more is permitted to be reduced in accordance with the following equation:

$$R = 0.08\ (A - 150) \qquad \text{Eq. (3.2.19)}$$

In SI:

$$R = 0.861\ (A - 13.94)$$

Such reduction shall not exceed the smallest of:

1. 40 percent for horizontal members;
2. 60 percent for vertical members; or
3. R as determined by the following equation.

$$R = 23.1\ (1 + D/L_0) \qquad Eq.\ (3.2.20)$$

where:

A = Area of floor supported by the member, ft^2 (m^2)
D = Dead load per ft2 (m^2) of area supported
L_0= Unreduced live load per ft2 (m^2) of area supported
R = Reduction in percent

3.2.3.10 Distribution of Floor Loads

Where uniform floor live loads are involved in the design of structural members arranged so as to create continuity, the minimum applied loads shall be the full dead loads on all spans in combination with the floor live loads on spans selected to produce the greatest effect at each location under consideration. It shall be permitted to reduce floor live loads in accordance with Section 3.2.3.9.

3.2.3.11 Roof Loads

The structural supports of roofs and marquees shall be designed to resist wind and, where applicable, earthquake load, in addition to the dead load of construction and the appropriate live loads as prescribed in this section, or set forth in Table 3.2.2. The live loads acting on a sloping surface shall be assumed to act vertically on the horizontal projection of that surface.

3.2.3.11.1 Distribution of Roof Loads

Where uniform roof live loads are reduced to less than 20 psf (0.958 kN/m^2) in accordance with Section 3.2.3.11.2.1 and are involved in the design of structural members arranged so as to create continuity, the minimum applied loads shall be the full dead loads on all spans in combination with the roof live loads on adjacent spans or on alternate spans, whichever produces the greatest effect. See Section 3.2.3.11.2 for minimum roof live loads.

3.2.3.11.2 Reduction in Roof Live Loads

The minimum uniformly distributed roof live loads, L0 in Table 3.2.2, are permitted to be reduced according to the following provisions.

3.2.3.11.2.1 Flat, Pitched, and Curved Roofs

Ordinary flat, pitched, and curved roofs are permitted to be designed for a reduced roof live load, as specified in Eq. (3.2.21) or other controlling combinations of loads, as discussed in Section 3.2.1, whichever produces the greater load. In structures such as greenhouses, where special scaffolding is used as a work surface for workmen and materials during maintenance and repair operations, a lower roof load than specified in Eq. (3.2.21) shall not be used unless approved by the authority having jurisdiction. On such structures, the minimum roof live load shall be 12 psf (0.58 kN/m^2).

$$L_r = L_0 R_1 R_2 \qquad \text{where } 12 \leq L_r \leq 20 \qquad \text{Eq. (3.2.21)}$$

In SI:

$$L_r = L_0 R_1 R_2 \qquad \text{where } 0.58 \leq L_r \leq 0.96$$

where,

L_r =reduced roof live load per ft2 (m^2) of horizontal projection in pounds per ft2 (kN/m^2)

The reduction factors R_1 and R_2 shall be determined as follows:

$$R_1 = 1 \qquad \text{for } A_t \leq 200\, ft^2 \qquad \text{Eq. (3.2.22a)}$$

$$R_1 = 1.2 - 0.001A_t \qquad \text{for } 200\, ft^2 < A_t < 600\, ft^2 \qquad \text{Eq. (3.2.22b)}$$

$$R_1 = 0.6 \qquad \text{for } A_t \geq 600\, ft^2 \qquad \text{Eq. (3.2.22c)}$$

In SI:

$$R_1 = 1 \qquad \text{for } A_t \leq 18.58\, m^2$$

$$R_1 = 1.2 - 0.001A_t \qquad \text{for } 18.58m2 < A_t < 55.74m^2$$

$$R_1 = 0.6 \qquad \text{for } A_t \geq 55.74m^2$$

where A_t = tributary area (i.e., span length multiplied by effective width) in ft^2 (m^2) supported by any structural member, and

$$R_2 = 1 \qquad \text{for } F \leq 4 \qquad \text{Eq. (3.2.23a)}$$

$$R_2 = 1.2 - 0.05F \qquad \text{for } 4 < F < 12 \qquad \text{Eq. (3.2.23b)}$$

$$R_2 = 0.6 \qquad \text{for } F \geq 12 \qquad \text{Eq. (3.2.23c)}$$

where

- for a pitched roof, F = number of inches of rise per foot (in SI: F = 0.12 × slope, with slope expressed as a percentage)
- for an arch or dome, F = rise-to-span ratio multiplied by 32.

3.2.3.11.2.2 Special Purpose Roofs

Roofs that have an occupancy function, such as roof gardens, assembly purposes, promenade purposes, or other special purposes shall be designed for a minimum live load as required in Table 3.2.2 and are permitted to have their uniformly distributed live load reduced in accordance with the requirements of Section 3.2.3.9.

3.2.3.11.2.3 Landscaped Roofs

Where roofs are to be landscaped, the uniform design live load in the landscaped area shall be 20 psf (0.958 kN/m^2). The weight of the landscaping materials shall be considered as dead load and shall be computed on the basis of saturation of the soil.

3.2.3.11.2.4 Awnings and Canopies

Awnings and canopies shall be designed for uniform live loads as required in Table 3.2.2 as well as for wind loads as specified in Section 3.

3.2.3.12 Crane Loads

The crane live load shall be the rated capacity of the crane. Design loads for the runway beams, including connections and support brackets, of moving bridge cranes and monorail cranes shall include the maximum wheel loads of the crane and the vertical impact, lateral, and longitudinal forces induced by the moving crane.

3.2.3.12.1 Maximum Wheel Load

The maximum wheel loads shall be the wheel loads produced by the weight of the bridge, as applicable, plus the sum of the rated capacity and the weight of the trolley with the trolley positioned on its runway at the location where the resulting load effect is maximum.

3.2.3.12.2 Vertical Impact Force

The maximum wheel loads of the crane shall be increased by the percentages shown below to determine the induced vertical impact or vibration force:

- Monorail cranes (powered) 25%
- Cab-operated or remotely operated bridge cranes (powered) 25%
- Pendant-operated bridge cranes (powered) 10%
- Bridge cranes or monorail cranes with hand-geared bridge, trolley, and hoist 0%

3.2.3.12.3 Lateral Force

The lateral force on crane runway beams with electrically powered trolleys shall be calculated as 20 percent of the sum of the rated capacity of the crane and the weight of the hoist and trolley. The lateral force shall be assumed to act horizontally at the traction surface of a runway beam, in either direction perpendicular to the beam, and shall be distributed according to the lateral stiffness of the runway beam and supporting structure.

3.2.3.12.4 Longitudinal Force

The longitudinal force on crane runway beams, except for bridge cranes with hand-geared bridges, shall be calculated as 10 percent of the maximum wheel loads of the crane. The longitudinal force shall be assumed to act horizontally at the traction surface of a runway beam in either direction parallel to the beam.

3.2.3.13 Interior Walls and Partitions

Interior walls and partitions that exceed 6 feet (1829 mm) in height, including their finish materials, shall have adequate strength to resist the loads to which they are subjected but not less than a horizontal load of 5 psf (0.240 kN/m^2).

EXCEPTION:

Fabric partitions complying with Section 3.2.3.13.1 shall not be required to resist the minimum horizontal load of 5 psf (0.240 kN/m^2).

3.2.3.13.1 Fabric Partition

Fabric partitions that exceed 6 ft (1829 mm) in height, including their finish materials, shall have adequate strength to resist the following load conditions:

1. A horizontal distributed load of 5 psf (0.240 kN/m^2) applied to the partition framing. The total area used to determine the distributed load shall be the area of the fabric face between the framing members to which the fabric is attached. The total distributed load shall be uniformly applied to such framing members in proportion to the length of each member.
2. A concentrated load of 40 pounds (0.176 kN) applied to an 8-in. diameter (203 mm) area [(50.3 in^2 (32452 mm^2)] of the fabric face at a height of 54 inches (1372 mm) above the floor.

3.2.4 Rain Loads

3.2.4.1 Symbols and Notation

R = rain load on the undeflected roof, in lb/ft^2 (kN/m^2). When the phrase "undeflected roof is used, deflections from loads (including dead loads) shall not be considered when determining the amount of rain on the roof.

d_s = depth of water on the undeflected roof up to the inlet of the secondary drainage system when the primary drainage system is blocked (i.e., the static head), in inches (mm).

d_h = additional depth of water on the undeflected roof above the inlet of the secondary drainage system at its design flow (i.e., the hydraulic head), in inches (mm).

3.2.4.2 Roof Drainage

Roof drainage systems shall be designed in accordance with the provisions of the code having jurisdiction. The flow capacity of secondary (overflow) drains or scuppers shall not be less than that of the primary drains or scuppers.

3.2.4.3 Design Rain Loads

Each portion of a roof shall be designed to sustain the load of all rainwater that will accumulate on it if the primary drainage system for that portion is blocked plus the uniform load caused by water that rises above the inlet of the secondary drainage system at its design flow.

$$R = 5.2\,(d_s + d_h) \qquad Eq. (3.2.24)$$

In SI:

$$R = 0.0098\,(d_s + d_h)$$

If the secondary drainage systems contain drain lines, such lines and their point of discharge shall be separate from the primary drain lines.

3.2.4.4 Ponding Instability

"Ponding" refers to the retention of water due solely to the deflection of relatively flat roofs. Roofs with a slope less than ¼ "per feet [1.19 degrees (0.0208 rad)] shall be investigated by structural analysis to assure that they possess adequate stiffness to preclude progressive deflection (i.e., instability) as rain falls on them. The primary drainage system within an area subjected to ponding shall be considered to be blocked in this analysis.

3.2.4.5 Controlled Drainage

Roofs equipped with hardware to control the rate of drainage shall be equipped with a secondary drainage system at a higher elevation that limits accumulation of water on the roof above that elevation. Such roofs shall be designed to sustain the load of all rainwater that will accumulate on them to the elevation of the secondary drainage system plus the uniform load caused by water that rises above the inlet of the secondary drainage system at its design flow (determined from Section 3.2.4.3).

Such roofs shall also be checked for ponding instability (determined from Section 3.2.4.4).

3.2.5 Flood Load

3.2.5.1 GENERAL

The provisions of this section apply to buildings and other structures located in areas prone to flooding.

3.2.5.2 DEFINITIONS

The following definitions apply to the provisions of this chapter:

APPROVED: Acceptable to the authority having jurisdiction.

BASE FLOOD: The flood having a 1 percent chance of being equaled or exceeded in any given year.

BASE FLOOD ELEVATION (BFE): The elevation of flooding, including wave height, having a 1 percent chance of being equaled or exceeded in any given year.

BREAKAWAY WALL: Any type of wall subject to flooding that is not required to provide structural support to a building or other structure, and that is designed and constructed such that, under base flood or lesser flood conditions, it will collapse in such a way that: (1) it allows the free passage of floodwaters, and (2) it does not damage the structure or supporting foundation system.

COASTAL A-ZONE: An area within a special flood hazard area, landward of a V-Zone or landward of an open coast without mapped V-Zones. To be classified as a Coastal A-Zone, the principal source of flooding must be astronomical tides, storm surges, seiches, or tsunamis, not riverine flooding, and the potential for breaking wave heights greater than or equal to 1.5 ft (0.46 m) must exist during the base flood.

COASTAL HIGH HAZARD AREA (V-ZONE): An area within a Special Flood Hazard Area, extending from offshore to the inland limit of a primary frontal dune along an open coast, and any other area that is subject to high-velocity wave action from storms or seismic sources.

DESIGN FLOOD ELEVATION (DFE): The elevation of the design flood, including wave height, relative to the datum specified on a community's flood hazard map.

FLOOD HAZARD AREA: The area subject to flooding during the design flood.

SPECIAL FLOOD HAZARD AREA (AREA OF SPECIAL FLOOD HAZARD): The land in the floodplain subject to a 1 percent or greater chance of flooding in any given year.

3.2.5.3 Design Requirements

3.2.5.3.1 Design Loads

Structural systems of buildings or other structures shall be designed, constructed, connected, and anchored to resist flotation, collapse, and permanent lateral displacement due to action of flood loads associated with the design flood (see Section 3.2.5.3.3) and other loads in accordance with the load combinations of Section 3.2.1.

3.2.5.3.2 Erosion and Scour

The effects of erosion and scour shall be included in the calculation of loads on buildings and other structures in flood hazard areas.

3.2.5.3.3 Loads on Breakaway Walls

Walls and partitions required by ASCE/SEI 24, to break away, including their connections to the structure, shall be designed for the largest of the following loads acting perpendicular to the plane of the wall:

1. The wind load specified in Section 3.3.
2. The earthquake load specified in Section 3.4.
3. 10 psf (0.48 kN/m^2).

The loading at which breakaway walls are intended to collapse shall not exceed 20 psf (0.96 kN/m^2) unless the design meets the following conditions:

1. Breakaway wall collapse is designed to result from a flood load less than that which occurs during the base flood.
2. The supporting foundation and the elevated portion of the building shall be designed against collapse, permanent lateral displacement, and other structural damage due to the effects of flood loads in combination with other loads as specified in Section 3.2.1.

3.2.5.4 Loads During Flooding

3.2.5.4.1 Load Basis

In flood hazard areas, the structural design shall be based on the design flood.

3.2.5.4.2 Hydrostatic Loads

Hydrostatic loads caused by a depth of water to the level of the DFE shall be applied over all surfaces involved, both above and below ground level, except that for surfaces exposed to free water, the design depth shall be increased by 1 ft (0.30 m).

Reduced uplift and lateral loads on surfaces of enclosed spaces below the DFE shall apply only if provision is made for entry and exit of floodwater.

3.2.5.4.3 Hydrodynamic Loads

Dynamic effects of moving water shall be determined by a detailed analysis utilizing basic concepts of fluid mechanics.

EXCEPTION:

Where water velocities do not exceed 10 ft/s (3.05 m/s), dynamic effects of moving water shall be permitted to be converted into equivalent hydrostatic loads by increasing the DFE for design purposes by an equivalent surcharge depth, d_h, on the headwater side and above the ground level only, equal to

$$D_h = \frac{aV^2}{2g} \qquad Eq.\ (3.2.25)$$

where,

V = average velocity of water in ft/s (m/s)
g = acceleration due to gravity, 32.2 ft/s (9.81 m/s^2)
a = coefficient of drag or shape factor (not less than 1.25)

The equivalent surcharge depth shall be added to the DFE design depth and the resultant hydrostatic pressures applied to, and uniformly distributed across, the vertical projected area of the building or structure that is perpendicular to the flow. Surfaces parallel to the flow or surfaces wetted by the tail water shall be subject to the hydrostatic pressures for depths to the DFE only.

3.2.5.4.4 Wave Loads

Wave loads shall be determined by one of the following three methods:

1. by using the analytical procedures outlined in this section,
2. by more advanced numerical modeling procedures,
3. by laboratory test procedures (physical modeling).

Wave loads are those loads that result from water waves propagating over the water surface and striking a building or other structure. Design and construction of buildings and other structures subject to wave loads shall account for the following loads: waves breaking on any portion of the building or structure; uplift forces caused by shoaling waves beneath a building or structure, or portion thereof; wave runup striking any portion of the building or structure; wave-induced drag and inertia forces; and wave-induced scour at the base of a building or structure, or its foundation. Wave loads shall be included for both V-Zones and A-Zones. In V-Zones, waves are 3 ft (0.91 m) high, or higher; in coastal floodplains landward of the V-Zone, waves are less than 3 ft high (0.91 m).

Nonbreaking and broken wave loads shall be calculated using the procedures described in Sections 3.2.5.4.2 and 3.2.5.4.3 that show how to calculate hydrostatic and hydrodynamic loads.

Breaking wave loads shall be calculated using the procedures described in Sections 3.2.5.4.4.1 through 3.2.5.4.4.4. Breaking wave heights used in the procedures described in Sections 3.2.5.4.4.1 through 3.2.5.4.4.4 shall be calculated for V-Zones and Coastal A-Zones using Eqs. 3.2.26 and 3.2.27.

$$H_b = 0.78 d_s \qquad Eq.\ (3.2.26)$$

where,

H_b = breaking wave height in ft (m)
d_s = local still water depth in ft (m)

The local still water depth shall be calculated using Eq. 3.2.27, unless more advanced procedures or laboratory tests permitted by this section are used.

$$d_s = 0.65(BFE - G) \qquad Eq.\ (3.2.27)$$

where,

BFE = BFE in ft (m)

G = ground elevation in ft (m)

3.2.5.4.4.1 Breaking Wave Loads on Vertical Pilings and Columns

The net force resulting from a breaking wave acting on a rigid vertical pile or column shall be assumed to act at the still water elevation and shall be calculated by the following:

$$F_D = 0.5\, \gamma_w\, C_D\, D\, H_b^2 \qquad \textit{Eq. (3.2.28)}$$

where,

F_D = net wave force, in lb (kN)

γ_w = unit weight of water, in lb per cubic ft (kN/m^3),
= 62.4 pcf (9.80 kN/m^3) for fresh water and 64.0 pcf (10.05 kN/m^3) for salt water

C_D = coefficient of drag for breaking waves,
= 1.75 for round piles or columns, and
= 2.25 for square piles or columns

D = pile or column diameter, in ft (m) for circular sections, or for a square pile or column, 1.4 times the width of the pile or column in ft (m)

H_b = breaking wave height, in ft (m)

3.2.5.4.4.2 Breaking Wave Loads on Vertical Walls

Maximum pressures and net forces resulting from a normally incident breaking wave (depth-limited in size, with $H_b = 0.78d_s$) acting on a rigid vertical wall shall be calculated by the following:

$$P_{max} = C_p\gamma_w d_s + 1.2\gamma_w d_s \qquad \textit{Eq (3.2.29)}$$

and

$$F_t = 1.1C_p\gamma_w d_s^2 + 2.4\gamma_w d_s^2 \qquad \textit{Eq (3.2.30)}$$

where,

P_{max}= maximum combined dynamic ($C_p\gamma_w d_s$) and static ($1.2\gamma_w d_s$) wave pressures, also referred to as shock pressures in lb/ft^2 (kN/m^2)

F_t = net breaking wave force per unit length of structure, also referred to as shock, impulse, or wave impact force in lb/ft (kN/m), acting near the still water elevation

C_p = dynamic pressure coefficient ($1.6 < C_p < 3.5$) (see Table 3.2.5-1)

γ_w = unit weight of water, in lb per cubic ft (kN/m^3), = 62.4 pcf (9.80 kN/m^3) for fresh water and 64.0 pcf (10.05 kN/m^3) for salt water

d_s = still water depth in ft (m) at base of building or other structure where the wave breaks

This procedure assumes the vertical wall causes a reflected or standing wave against the waterward side of the wall with the crest of the wave at a height of $1.2d_s$ above the still water level. Thus, the dynamic static and total pressure distributions against the wall are as shown in Fig. 3.2.5-1.

This procedure also assumes the space behind the vertical wall is dry, with no fluid balancing the static component of the wave force on the outside of the wall. If free water exists behind the wall, a portion of the hydrostatic component of the wave pressure and force disappears (see Fig. 3.2.5-2) and the net force shall be computed by Eq. 3.2.31 (the maximum combined wave pressure is still computed with Eq. 3.2.29).

$$F_t = 1.1C_p\gamma_w d^2_s + 1.9\gamma_w d_s^2 \qquad Eq.\ (3.2.31)$$

where,

F_t = net breaking wave force per unit length of structure, also referred to as shock, impulse, or wave impact force in lb/ft (kN/m), acting near the still water elevation

C_p = dynamic pressure coefficient ($1.6 < C_p < 3.5$) (see Table 3.2.5-1)

γ_w = unit weight of water, in lb per cubic ft (kN/m^3), = 62.4 pcf (9.80 kN/m^3) for fresh water and 64.0 pcf (10.05 kN/m^3) for salt water

d_s = still water depth in ft (m) at base of building or other structure where the wave breaks

3.2.5.4.4.3 Breaking Wave Loads on Non Vertical Walls

Breaking wave forces given by Eqs. 3.2.30 and 3.2.31 shall be modified in instances where the walls or surfaces upon which the breaking waves act are non-vertical. The horizontal component of breaking wave force shall be given by

$$F_{nv} = F_t \sin^2 a \qquad Eq.\ (3.2.32)$$

where,

F_{nv} = horizontal component of breaking wave force in lb/ft (kN/m)

F_t = net breaking wave force acting on a vertical surface in lb/ft (kN/m)

a = vertical angle between non-vertical surface and the horizontal

3.2.5.4.4.4 Breaking Wave Loads from Obliquely Incident Waves

Breaking wave forces given by Eqs. 3.2.30 and 3.2.31 shall be modified in instances where waves are obliquely incident. Breaking wave forces from non-normally incident waves shall be given by

$$F_{oi} = F_t \sin^2 a \qquad Eq.\ (3.2.33)$$

where

F_{oi} = horizontal component of obliquely incident breaking wave force in lb/ft (kN/m)

F_t = net breaking wave force (normally incident waves) acting on a vertical surface in lb/ft (kN/m)

a = horizontal angle between the direction of wave approach and the vertical surface

3.2.5.4.5 Impact Loads

Impact loads are those that result from debris, ice, and any object transported by floodwaters striking against buildings and structures, or parts thereof. Impact loads shall be determined using a rational approach as concentrated loads acting horizontally at the most critical location at or below the DFE.

TABLE 3.2.5-1 VALUE OF DYNAMIC PRESSURE COEFFICIENT, Cp

Building Category	C_p
I	1.6
II	2.8
III	3.2
IV	3.5

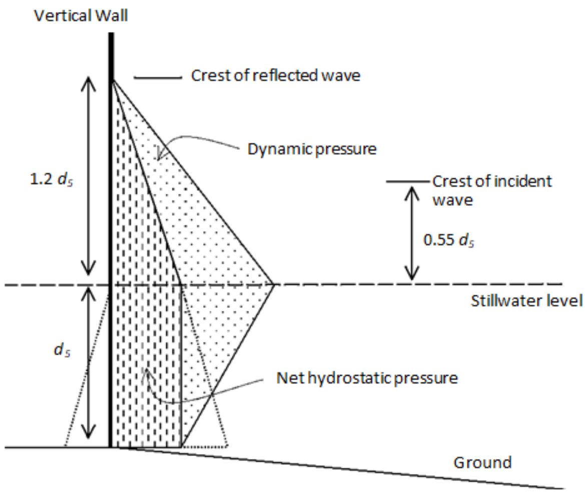


FIGURE 3.2.5- 1 NORMALLY INCIDENT BREAKING WAVE PRESSURES AGAINST A VERTICAL WALL (space behind vertical wall is dry)

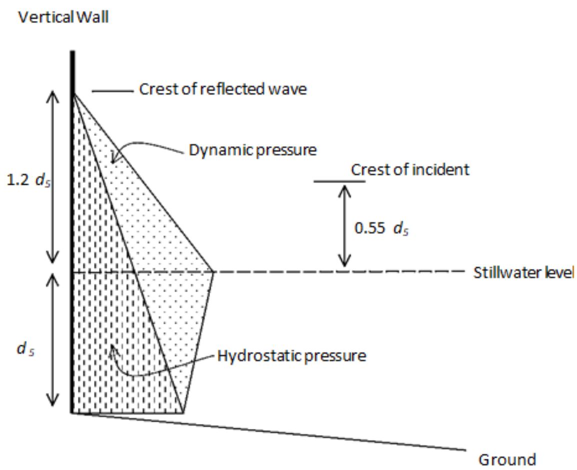


FIGURE 3.2.5- 2 NORMALLY INCIDENT BREAKING WAVE PRESSURES AGAINST A VERTICAL WALL (still water level equal on both sides of wall)

SECTION 3.3
WIND DESIGN CRITERIA

3.3.1 General

3.3.1.1 Scope

Buildings and other structures, including the Main Wind-Force Resisting System (MWFRS) and all components and cladding thereof, shall be designed and constructed to resist wind loads as specified herein. Decreases in wind loads shall not be made for the effect of shielding by other structures.

3.3.1.2 Allowed Procedures

The design wind loads for buildings and other structures, including the MWFRS and component and cladding elements thereof, shall be determined using one of the following procedures:(1) Method 1 – Simplified Procedure as specified in Section 3.3.4 for buildings meeting the requirements specified therein; (2) Method 2 – Analytical Procedure as specified in Section 6.5 of ASCE 7-05 for buildings meeting the requirements specified therein; (3) Method 3 – Wind Tunnel Procedure as specified in Section 6.6 of ASCE 7-05.

3.3.1.3 Wind Pressures Acting on Opposite Faces of Each Building Surface

In the calculation of design wind loads for the MWFRS and for components and cladding of buildings, the algebraic sum of the pressures acting on opposite faces of each building surface shall be taken into account.

3.3.1.4 Minimum Design Wind Loading

The design wind load, determined by any one of the procedures specified in Section 3.3.1.2, shall be not less than that specified in this section.

3.3.1.4.1 Main Wind-Force Resisting System

The wind load to be used in the design of the MWFRS for an enclosed or partially enclosed building shall not be less than 10 lb/ft^2 (0.48 kN/m^2) multiplied by the area of the building or structure projected onto a vertical plane normal to the assumed wind direction. The design wind force for open buildings and other structures shall be not less than 10 lb/ft^2 (0.48 kN/m^2) multiplied by the area A_f.

3.3.1.4.2 Components and Cladding

The design wind pressure for components and cladding of buildings shall not be less than net pressure of10lb/ft^2 (0.48kN/m^2) acting in either direction normal to the surface.

3.3.2 Definitions

The following definitions apply only to the provisions of Section 3.3.

APPROVED: Acceptable to the authority having jurisdiction.

BASIC WIND SPEED, V: Three-second gust speed at 33 ft (10 m) above the ground in Exposure C (see Section 3.3.4.6.3) as determined in accordance with Section 3.3.4.4.

BUILDING, ENCLOSED: A building that does not comply with the requirements for open or partially enclosed buildings.

BUILDING ENVELOPE: Cladding, roofing, exterior walls, glazing, door assemblies, window assemblies, skylight assemblies, and other components enclosing the building.

BUILDING AND OTHER STRUCTURE, FLEXIBLE: Slender buildings that have a fundamental natural frequency less than 1Hz.
BUILDING, LOW-RISE: Enclosed or partially enclosed buildings that comply with the following conditions:

1. Mean roof height h less than or equal to 60 ft (18 m).
2. Mean roof height h does not exceed least horizontal dimension.

BUILDING, OPEN: A building having each wall at least 80 percent open. This condition is expressed for each wall by the equation $A_0 \geq 0.8A_g$where

A_0 = total area of openings in a wall that receives positive external pressure, in ft^2 (m^2)

A_x = the gross area of that wall in which A_0 is identified, in ft^2 (m^2)

BUILDING PARTIALLY ENCLOSED: A building that complies with both of the following conditions:

1. The total area of openings in a wall that receives positive external pressure exceeds the sum of the areas of openings in the balance of the building envelope (walls and roof) by more than 10 percent.
2. The total area of openings in a wall that receives positive external pressure exceeds 4 ft^2 (0.37 m^2) or 1 percent of the area of that wall, whichever is smaller, and the percentage of openings in the balance of the building envelope does not exceed 20 percent.

These conditions are expressed by the following equations:

1. $A_0 > 1.10 A_{0i}$;
2. $A_0 >$ 4 sqft (0.37 m^2) or $> 0.01 A_g$, whichever is smaller, and$A_{0i} / A_{gi} \leq 0.20$

where
A_0, A_g are as defined for Open Building
A_{0i} = the sum of the areas of openings in the building envelope (walls and roof) not including A_0, in ft^2 (m^2)
A_{gi} = the sum of the gross surface areas of the building envelope (walls and roof) not including A_g, in ft^2 (m^2)

BUILDING OR OTHER STRUCTURES, REGULAR-SHAPED: A building having no unusual geometrical irregularity in spatial form.

BUILDING OR OTHER STRUCTURES, RIGID: A building whose fundamental frequency is greater than or equal to**1Hz.**

BUILDING, SIMPLE DIAPHRAGM: A building in which both windward and leeward wind loads are transmitted through floor and roof diaphragms to the same vertical MWFRS (e.g., no structural separations).

COMPONENTS AND CLADDING: Elements of the building envelope that do not qualify as part of the MWFRS.

DESIGNFORCE, F: Equivalent static force to be used in the determination of wind loads for open buildings.

DESIGN PRESSURE, p: Equivalent static pressure to be used in the determination of wind loads for buildings.

EAVE HEIGHT, h: The distance from the ground surface adjacent to the building to the roof eave line at a particular wall.

If the height of the eave varies along the wall, the average height shall be used.

EFFECTIVE WIND AREA, A: The area used to determine GC_p. For component and cladding elements, the effective wind area in Fig: 6-11 through 6-17 and Fig: 6-19 in Appendix is the span length multiplied by an effective width that need not be less than one-third the span length. For cladding fasteners, the effective wind area shall not be greater than the area that is tributary to an individual fastener.

ESCARPMENT: Also known as scarp, with respect to topographic effects in Section 3.3.5.7, a cliff or steep slope generally separating two levels or gently sloping areas (see Fig.3.3.3).

FREE ROOF: Roof with a configuration generally conforming to those shown in Fig 6-18A through 6-18D in Appendix (mono slope, pitched, or troughed) in an open building with no enclosing walls underneath the roof surface.

GLAZING: Glass or transparent or translucent plastic sheet used in windows, doors, skylights, or curtain walls.

GLAZING, IMPACT RESISTANT: Glazing that has been shown by testing in accordance with ASTM El886 and ASTM El996 or other approved test methods to withstand the impact of wind-borne missiles likely to be generated in wind-borne debris regions during design winds.

HILL: With respect to topographic effects in Section 3.3.5.7, a land surface characterized by strong relief in any horizontal direction (see Fig. 3.3.3).

IMPORTANCE FACTOR, I: A factor that accounts for the degree of hazard to human life and damage to property.

MAIN WIND-FORCE RESISTING SYSTEM (MWFRS): An assemblage of structural elements assigned to provide support and stability for the overall structure. The system generally receives wind loading from more than one surface.

MEAN ROOF HEIGHT, h: The average of the roof eave height and the height to the highest point on the roof surface, except that, for roof angles of less than or equal to **10°,** the mean roof height shall be the roof eave height.

OPENINGS: Apertures or holes in the building envelope that allow air to flow through the building envelope and that are designed as "open" during design winds as defined by these provisions.

RECOGNIZED LITERATURE: Published research findings and technical papers that are approved.

RIDGE: With respect to topographic effects in Section 3.3.4.7 an elongated crest of a hill characterized by strong relief in two directions (see Fig. 3.3.3).

3.3.3 Symbols and Notation

The following symbols and notation apply only to the provisions of Section 3.3.

A = effective wind area, in ft^2 (m^2)

A_f = area of open buildings either normal to the wind direction or projected on a plane normal to the wind direction, in ft^2 (m^2)

A_g = the gross area of that wall in which A_0 is identified, in ft^2 (m^2)

A_{gi} = the sum of the gross surface areas of the building envelope (walls and roof) not including A_g, in ft^2 (m^2)

A_0 = total area of openings in a wall that receives positive external pressure, in ft^2 (m^2)

A_{0i} = the sum of the areas of openings in the building envelope (walls and roof) not including A_0, in ft^2 (m^2)

A_{0g} = total area of openings in the building envelope, in ft^2 (m^2)

A_s = gross area of the solid freestanding wall or solid sign, in ft^2 (m^2)

a = width of pressure coefficient zone, in ft (m)

B = horizontal dimension of building measured normal to wind direction, in ft (m)

$\bar{b}$ = mean hourly wind speed factor in Eq. 6-14 from Table 6-2 (ASCE 7-05)

$\hat{b}$ = 3-s gust speed factor from Table 6-2 (ASCE 7-05)

C_f = force coefficient to be used in determination of wind loads for other structures

C_N = net pressure coefficient to be used in determination of wind loads for open buildings

C_p = external pressure coefficient to be used in determination of wind loads for buildings

c = turbulence intensity factor in Eq. 6-5 from Table 6-2 (ASCE 7-05)

D = diameter of a circular structure or member, in ft (m)

D' = depth of protruding elements such as ribs and spoilers, in ft (m)

F = design wind force for other structures, in lb (N)

G = gust effect factor

G_f = gust effect factor for MWFRSs of flexible buildings

GC_{pn} = combined net pressure coefficient for a parapet

GC_p = product of external pressure coefficient and gust effect factor to be used in determination of wind loads for buildings

GC_{pf} = product of the equivalent external pressure coefficient and gust-effect factor to be used in determination of wind loads for MWFRS of low-rise buildings

GC_{pi} = product of internal pressure coefficient and gust-effect factor to be used in determination of wind loads for buildings

g_Q = peak factor for background response Eq. 6-4 and 6-8 (ASCE 7-05)

g_R = peak factor for resonant response Eq. 6-8 (ASCE 7-05)

g_v = peak factor for wind response Eq. 6-4 and 6-8 (ASCE 7-05)

H = height of hill or escarpment in Fig. 3.3.3, in ft (m)

h = mean roof height of a building, except that eave height shall be used for roof angle θ of less than or equal to 10°, in ft(m)

h_e = roof eave height at a particular wall, or the average height if the eave varies long the wall

I = importance factor

$I_{\bar{z}}$ = intensity of turbulence from Eq. 6-5 (ASCE 7-05)

K_1, K_2, K_3= multipliers in Fig.3.3.3 to obtain K_{zt}

K_d = wind directionality factor in Table 6-4 (ASCE 7-05)

K_h = velocity pressure exposure coefficient evaluated at height $z=h$

K_z = velocity pressure exposure coefficient evaluated at height z

K_{zt} = topographic factor as defined in Section 3.3.4.7

L = horizontal dimension of a building measured parallel the wind direction, in ft(m)

L_h = distance upwind of crest of hill or escarpment in Fig.3.3.3 to where the difference in ground elevation is half the height of hill or escarpment, in ft (m)

$L_{\bar{z}}$ = integral length scale of turbulence, in ft (m)

L_r = horizontal dimension of return corner for a solid freestanding wall or solid sign from Fig.6-20 (ASCE 7-05), in ft (m)

ℓ = integral length scale factor from Table 3.3.3, ft (m)

N_1 = reduced frequency from Eq. 6-12 (ASCE 7-05)

n_1 = building natural frequency, Hz

p = design pressure to be used in determination of wind loads for buildings, in lb/ft^2 (N/m^2)

p_L = wind pressure acting on leeward face in Fig 6-9 (ASCE 7-05), in lb/ft^2 (N/m^2)

p_{net} = net design wind pressure from Eq. (3.3.2), in lb/ft^2 (N/m^2)

p_{net30} = net design wind pressure for Exposure B at h=30ft and I= 1.0 from Fig.3.3.2, in lb/ft^2 (N/m^2)

p_p = combined net pressure on a parapet from Eq 6-20 (ASCE 7-05) in lb/ft^2 (N/m^2)

p_s = net design wind pressure from Eq. (3.3.1), in lb/ft^2 (N/m^2)

$p_{s\,30}$ = simplified design wind pressure for Exposure B at h= 30ft and I=1.0 from Fig.3.3.1, in lb/ft^2 (N/m^2)

p_w = wind pressure acting on windward face in Fig 6-9 (ASCE 7-05), in lb/ft^2 (N/m^2)

Q = background response factor from Eq. 6-6 (ASCE 7-05)

q = velocity pressure, in lb/ft^2 (N/m^2)

q_h = velocity pressure evaluated at height $z= h$, in lb/ft^2 (N/m^2)

q_i = velocity pressure for internal pressure determination, in lb/ft^2 (N/m^2)

q_p = velocity pressure at top of parapet, in lb/ft^2 (N/m^2)

q_z = velocity pressure evaluated at height z above ground, in lb/ft^2 (N/m^2)

R = resonant response factor from Eq. 6-10 (ASCE 7-05)

R_B, R_h, R_L =values from Eq. 6-13 (ASCE 7-05)

R_i = reduction factor from Eq. 6-16 (ASCE 7-05)

R_n = value from Eq. 6-11 (ASCE 7-05)

s = vertical dimension of the solid freestanding wall or solid sign from Fig 6-20 (ASCE 7-05), in ft (m)

r = rise-to-span ratio for arched roofs

V =basic wind speed obtained from Table 3.3.1, in mi/h (m/s). The basic wind speed corresponds to a 3-s gust speed at 33 ft (10 m) above ground in exposure Category C

V_i = un partitioned internal volume, ft^3 (m^3)

$\overline{V}_{\bar{z}}$ = mean hourly wind speed at height $\bar{z}$, ft/s (m/s)

W = width of building in Figs. 6-12 and 6-14A (ASCE 7-05) and B and width of span in Figs.6-13 and 6-15 (ASCE 7-05), in ft(m)

X = distance to centre of pressure from wind ward edge in Fig.6-18 (ASCE 7-05), in ft (m)

x = distance up wind or downwind of crest in Fig.3.3.3, in ft (m)

z = height above ground level, in ft (m)

$\bar{z}$ = equivalent height of structure, in ft (m)

z_g = nominal height of the atmospheric boundary layer used in this standard. Values appear in Table 6-2 (ASCE 7-05)

z_{min} = exposure constant from Table 6-2 (ASCE 7-05)

α = 3-s gust-speed power law exponent from Table 6-2 (ASCE 7-05)

$\tilde{\alpha}$ = reciprocal of α from Table 6-2 (ASCE 7-05)

$\bar{\alpha}$ = mean hourly wind-speed power law exponent in Eq.6-14 from Table 6-2 (ASCE 7-05)

β = damping ratio, percent critical for buildings

ϵ = ratio of solid area to gross area for solid free-standing wall, solid sign, open sign, face of a trussed tower, or lattice structure

λ = adjustment factor for building height and exposure from Figs.3.3.1 and 3.3.2

$\bar{\epsilon}$ = integral length scale power law exponent in Eq. 6-7 from Table 6-2 (ASCE 7-05)

η = value used in Eq. 6-13 (see Section 6.5.8.2) (ASCE 7-05)

θ = angle of plane of roof from horizontal, in degrees

v = height-to-width ratio for solid sign

3.3.4 – Method 1 - Simplified Procedure

3.3.4.1 Scope

A building whose design wind loads are determined in accordance with this section shall meet all the conditions of 3.3.4.1.1 or 3.3.4.1.2. If a building qualifies only under 3.3.4.1.2 for design of its components and cladding, then its MWFRS shall be designed by Method 2 – Analytical Procedure as specified in Section 6.5 of ASCE 7-05 (or) Method 3 – Wind Tunnel Procedure as specified in Section 6.6 of ASCE 7-05.

3.3.4.1.1 Main Wind-Force Resisting Systems

For the design of MWFRSs the building must meet all of the following conditions:

1. The building is a simple diaphragm building as defined in Section 3.3.2.
2. The building is a low-rise building as defined in Section 3.3.2.
3. The building is enclosed as defined in Section 3.3.2
4. The building is a regular-shaped building or structure as defined in Section 3.3.2.
5. The building is not classified as a flexible building as defined in Section 3.3.2.
6. The building does not have response characteristics making it subject to across wind loading, vortex shedding, instability due to galloping or flutter; and does not have a site location for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.
7. The building has an approximately symmetrical cross section in each direction with either a flat roof or a gable or hip roof with $\theta \leq 45°$.
8. The building is exempted from torsional load cases as indicated in Note 5 of Fig.6-10 (ASCE 7-05), or the torsional load cases defined in Note 5 do not control the design of any of the MWFRSs of the building.

3.3.4.1.2 Components and Cladding

For the design of components and cladding the building must meet all the following conditions:

1. The mean roof height h must be less than or equal to 60 ft ($h \leq 60$ ft).
2. The building is enclosed as defined in Section 3.3.2
3. The building is a regular-shaped building or structure as defined in Section 3.3.2.
4. The building does not have response characteristics making it subject to across wind loading, vortex shedding, instability due to galloping or flutter; and does not have a site location for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.
5. The building has a flat roof, a gable roof with, or a hip roof with $\theta \leq 45°$, or a hip roof with $\theta \leq 27°$.

3.3.4.2 Design Procedure

1. The basic wind speed V shall be determined in accordance with Section 3.3.4.4. The wind shall be assumed to come from any horizontal direction.
2. An importance factor I shall be determined in accordance with Section 3.3.4.5.
3. An exposure category shall be determined in accordance with Section 3.3.4.6.
4. A height and exposure adjustment coefficient, λ, shall be determined from Fig. 3.3.1.

3.3.4.2.1 Main Wind-Force Resisting System

Simplified design wind pressures, p_s, for the MWFRSs of low-rise simple diaphragm buildings represent the net pressures (sum of internal and external) to be applied to the horizontal and vertical projections of building surfaces as shown in Fig. 3.3.1. For the horizontal pressures (zones A, B, C, D), p_s is the combination of the windward and leeward net pressures. p_s, shall be determined by the following equation:

$$p_s = \lambda K_{zt} I p_{S30} \qquad \textit{Eq. (3.3.1)}$$

where

λ = adjustment factor for building height and exposure from Fig. 3.3.1

K_{zt} = topographic factor as defined in Section 3.3.4.7 evaluated at mean roof height, h

I = importance factor as defined in Section 3.3.2

p_{S30} = simplified design wind pressure for Exposure B, at h = 30 ft, and for I = 1.0, from Fig. 3.3.1

3.3.4.2.1.1 Minimum Pressures

The load effects of the design wind pressures from Section 3.3.4.2.1 shall not be less than the minimum load case from Section 3.3.1.4.1 assuming the pressures, p_s, for zones A, B, C, and D all equal to +10 psf, while assuming zones E, F, G, and H all equal to 0 psf.

3.3.4.2.2 Components and Cladding

Net design wind pressures, p_{net}, for the components and cladding of buildings designed using Method 1 represent the net pressures (sum of internal and external) to be applied normal to each building surface as shown in Fig. 3.2. p_{net} shall be determined by the following equation:

$$p_{net} = \lambda K_{zt} I p_{net30} \qquad \textit{Eq. (3.3.2)}$$

where

λ = adjustment factor for building height and exposure from Fig. 3.3.2

K_{zt} = topographic factor as defined in Section 3.3.4.7 evaluated at mean roof height, h

I = importance factor as defined in Section 3.3.2

p_{net30} = net design wind pressure for exposure B, at h = 30 ft, and for I = 1.0, from Fig. 3.3.2

3.3.4.2.2.1 Minimum pressures

The positive design wind pressures, p_{net}, from Section 3.3.4.2.2 shall not be less than +10 psf, and the negative design wind pressures, p_{net}, from Section 3.3.4.2.2 shall not be less than - 10 psf.

3.3.4.3 Air Permeable Cladding

Design wind loads determined from Fig. 3.3.2 shall be used for all air permeable cladding unless approved test data or the recognized literature demonstrate lower loads for the type of air permeable cladding being considered.

3.3.4.4 Basic Wind Speed

The basic wind speed, V, used in the determination of design wind loads on buildings shall be as given in Table 3.3.1 except as provided in Sections 3.3.4.4.1 and 3.3.4.4.2. The wind shall be assumed to come from any horizontal direction.

3.3.4.4.1 Special Wind Regions

The basic wind speed shall be increased where records or experience indicate that the wind speeds are higher than those reflected in Table 3.3.1. Mountainous terrain gorges and special regions shall be examined for unusual wind conditions. The authority having jurisdiction shall, if necessary, adjust the values given in Table 3.3.1 to account for higher local wind speeds. Such adjustment shall be based on meteorological information and an estimate of the basic wind speed obtained in accordance with the provisions of Section 3.3.4.4.2.

3.3.4.4.2 Estimation of Basic Wind Speeds from Regional Climatic Data

Regional climatic data shall only be used in lieu of the basic wind speeds given in Table 3.3.1 when (1) approved extreme-value statistical-analysis procedures have been employed in reducing the data; and (2) the length of record, sampling error, averaging time, anemometer height, data quality, and terrain exposure of the anemometer have been taken into account. Reduction in basic wind speed below that of Table 3.3.1 shall not be permitted.

When the basic wind speed is estimated from regional climatic data, the basic wind speed shall be not less than the wind speed associated with an annual probability of 0.02 (50- year mean recurrence interval), and the estimate shall be adjusted for equivalence to a 3-s gust wind speed at 33 ft (10 m) above ground in exposure Category C. The data analysis shall be performed in accordance with this section.

3.3.4.5 Importance Factor

An importance factor, I, for the building shall be determined from Table 3.3.2 based on building categories listed in Table 3.1.2.

3.3.4.6 Exposure

For each wind direction considered, the upwind exposure category shall be based on ground surface roughness that is determined from natural topography, vegetation, and constructed facilities.

3.3.4.6.1 Wind Directions and Sectors

For each selected wind direction at which the wind loads are to be evaluated, the exposure of the building or structure shall be determined for the two upwind sectors extending 45° either side of the selected wind direction. The exposures in these two sectors shall be determined in accordance with Sections 3.3.4.6.2 and 3.3.4.6.3 and the exposure resulting in the highest wind loads shall be used to represent the winds from that direction.

3.3.4.6.2 Surface Roughness Categories

A ground surface roughness within each 45° sector shall be determined for a distance upwind of the site as defined in Section 3.3.4.6.3 from the categories defined in the following text, for the purpose of assigning an exposure category as defined in Section 3.3.4.6.3.

Surface Roughness B: Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger.

Surface Roughness C: Open terrain with scattered obstructions having heights generally less than 30 ft (9.1 m).

Surface Roughness D: Flat, unobstructed areas and water surfaces. This category includes smooth mud flats and salt flats.

3.3.4.6.3 Exposure categories

Exposure B: Exposure B shall apply where the ground surface roughness condition, as defined by Surface Roughness B, prevails in the upwind direction for a distance of at least 2,600 ft (792 m) or 20 times the height of the building, whichever is greater.

EXCEPTION:

For buildings whose mean roof height is less than or equal to 30 ft, the upwind distance may be reduced to 1,500 ft (457 m).

Exposure C: Exposure C shall apply for all cases where Exposures B or D do not apply.

Exposure D: Exposure D shall apply where the ground surface roughness, as defined by Surface Roughness D, prevails in the upwind direction for a distance greater than 5,000 ft (1,524 m) or 20 times the building height, whichever is greater. Exposure D shall extend into downwind areas of Surface Roughness B or C for a distance of 600 ft (200 m) or 20 times the height of the building, whichever is greater.
For a site located in the transition zone between exposure categories, the category resulting in the largest wind forces shall be used.

EXCEPTION:

An intermediate exposure between the preceding categories is permitted in a transition zone provided that it is determined by a rational analysis method defined in the recognized literature.

3.3.4.7 Topographic Effects

3.3.4.7.1 Wind Speed-Up Over Hills, Ridges, and Escarpments

Wind speed-up effects at isolated hills, ridges, and escarpments constituting abrupt changes in the general topography, located in any exposure category, and shall be included in the design when buildings and other site conditions and locations of structures meet all of the following conditions:

1. The hill, ridge, or escarpment is isolated and unobstructed upwind by other similar topographic features of comparable height for 100 times the height of the topographic feature (100H) or 2 mi (3.22 km), whichever is less. This distance shall be measured horizontally from the point at which the height H of the hill, ridge, or escarpment is determined.
2. The hill, ridge, or escarpment protrudes above the height of upwind terrain features within a 2-mi (3.22 km) radius in any quadrant by a factor of two or more.
3. The structure is located as shown in Fig. 3.3.3 in the upper one-half of a hill or ridge or near the crest of an escarpment
4. $H/L_h \geq 0.2$.
5. H is greater than or equal to 15 ft (4.5 m) for Exposures C and D and 60 ft (18 m) for Exposure B.

3.3.4.7.2 Topographic Factor

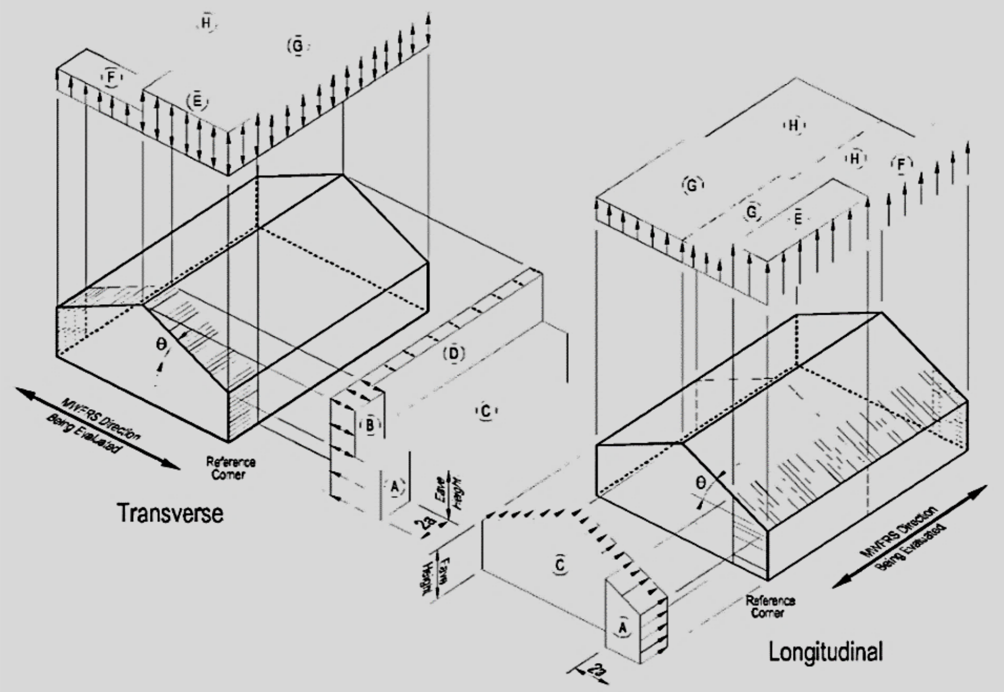
The wind speed-up effect shall be included in the calculation of design wind loads by using the factor K_{zt}:

$$K_{zt} = (1 + K_1 K_1 K_3)^2 \qquad \textit{Eq. (3.3.3)}$$

where
K_1, K_2, and K_3 are given in Fig. 3.3.3

If site conditions and locations of structures do not meet all the conditions specified in section 3.3.4.7.1 then K_{zt}= 1.0.

Main Wind Force Resisting System — Method 1		$h \leq 60$ ft.
Figure 3.3.1 (cont'd)	**Design Wind Pressure**	**Walls & Roofs**
Enclosed Buildings		



Notes:

1. Pressures shown are applied to the horizontal and vertical projections, for exposure B, at h=30 ft (9.1 m), I = 1.0, and K_{zt} = 1.0. Adjust to other condition using Equation 3.3.1.
2. The load patterns shown shall be applied to each corner of the building in turn as the reference corner
3. For the design of the longitudinal MWFRS use θ= 0°, and locate the zone E/F, G/H boundary at the mid-length of the building.
4. Load cases 1 and 2 must be checked for 25° < θ ≤ 45°. Load case 2 at 25° is provided only for interpolation between 25° to 30°.
5. Plus and minus signs signify pressures acting toward and away from the projected surfaces, respectively.
6. For roof slopes other than those shown, linear interpolation is permitted.
7. The total horizontal load shall not be less than that determined by assuming p_s = 0 in zones B & D.
8. The zone pressures represent the following:
9. Horizontal pressure zones - Sum of the windward and leeward net (sun of internal and external) pressures on vertical projection of:
   a. A - End zone of wall C - Interior zone of wall
   b. B - End zone of roof D - Interior zone of roof
10. Vertical pressure zones - Net (sum of internal and external) pressures on horizontal projection of:
    a. E - End zone of windward roof G - Interior zone of windward roof
    b. F - End zone of leeward roof H - Interior zone of leeward roof
11. Where zone E or G falls on a roof overhang on the windward side of the building, use E_{OH} and G_{OH} for the pressure on the horizontal projection of the overhang. Overhangs on the leeward and side edges shall have the basic zone pressure applied.
12. Notation:
13. a: 10 percent of least horizontal dimension or 0.4h, whichever is smaller, but not less than either 4 % of least horizontal dimension or 3 ft (0.9 m)
14. h: mean roof height, in feet (meters), except that eave height shall be used for roof angles < 10°.
15. θ: angle of plane of roof from horizontal, in degrees.

Main Wind Force Resisting System — Method 1											h ≤ 60 ft.	
Figure 3.3.1 (cont'd)		Design Wind Pressure									Walls & Roofs	
Enclosed Buildings												
Simplified Design Wind Pressure, p_{s30} (psf) *(Exposure B at h = 30 ft., K_{z1} = 1.0, with I = 1.0)*												
Basic Wind Speed (mph	Roof Angle (deg)	Load Case	Zones									
			Horizontal Pressures				Vertical Pressures				Overhangs	
			A	B	C	D	E	F	G	H	E_{OH}	G_{OH}
85	0 to 5°	1	11.5	-5.9	7.6	-3.5	-13.8	-7.8	-9.6	-6.1	-19.3	-15.1
	10°	1	12.9	-5.4	8.6	-3.1	-13.8	-8.4	-9.6	-6.5	-19.3	-15.1
	15°	1	14.4	-4.8	9.6	-2.7	-13.8	-9.0	-9.6	-6.9	-19.3	-15.1
	20°	1	15.9	-4.2	10.6	-2.3	-13.8	-9.6	-9.6	-7.3	-19.3	-15.1
	25°	1	14.4	2.3	10.4	2.4	-6.4	-8.7	-4.6	-7	-11.9	-10.1
		2	------	------	------	------	-2.4	-4.7	-0.7	-3	------	------
	30° to 45°	1	12.9	8.8	10.2	7.0	1.0	-7.8	0.3	-6.7	-4.5	-5.2
		2	12.9	8.8	10.2	7.0	5.0	-3.9	4.3	-2.8	-4.5	-5.2
90	0 to 5°	1	12.8	-6.7	8.5	-4.0	-15.4	-8.8	-10.7	-6.8	-21.6	-16.9
	10°	1	14.5	-6.0	9.6	-3.5	-15.4	-9.4	-10.7	-7.2	-21.6	-16.9
	15°	1	16.1	-5.4	10.7	-3.0	-15.4	-10.1	-10.7	-7.7	-21.6	-16.9
	20°	1	17.8	-4.7	11.9	-2.6	-15.4	-10.7	-10.7	-8.1	-21.6	-16.9
	25°	1	16.1	2.6	11.7	2.7	-7.2	-9.8	-5.2	-7.8	-13.3	-11.4
		2	------	------	------	------	-2.7	-5.3	-0.7	-3.4	------	------
	30° to 45°	1	14.4	9.9	11.5	7.9	1.1	-8.8	0.4	-7.5	-5.1	-5.8
		2	14.4	9.9	11.5	7.9	5.6	-4.3	4.8	-3.1	-5.1	-5.8
100	0 to 5°	1	15.9	-8.2	10.5	-4.9	-19.1	-10.8	-13.3	-8.4	-26.7	-20.9
	10°	1	17.9	-7.4	11.9	-4.3	-19.1	-11.6	-13.3	-8.9	-26.7	-20.9
	15°	1	19.9	-6.6	13.3	-3.8	-19.1	-12.4	-13.3	-9.5	-26.7	-20.9
	20°	1	22.0	-5.8	14.6	-3.2	-19.1	-13.3	-13.3	-10.1	-26.7	-20.9
	25°	1	19.9	3.2	14.4	3.3	-8.8	-12	-6.4	-9.7	-16.5	-14
		2	------	------	------	------	-3.4	-6.6	-0.9	-4.2	------	------
	30° to 45°	1	17.8	12.2	14.2	9.8	1.4	-10.8	0.5	-9.3	-6.3	-7.2
		2	17.8	12.2	14.2	9.8	6.9	-5.3	5.9	-3.8	-6.3	-7.2
105	0 to 5°	1	17.5	-9.0	11.6	-5.4	-21.1	-11.9	-14.7	-9.3	-29.4	-23
	10°	1	19.7	-8.2	13.1	-4.7	-21.1	-12.8	-14.7	-9.8	-29.4	-23
	15°	1	21.9	-7.3	14.7	-4.2	-21.1	-13.7	-14.7	-10.5	-29.4	-23
	20°	1	24.3	-8.4	16.1	-3.5	-21.1	-14.7	-14.7	-11.1	-29.4	-23
	25°	1	21.9	3.5	15.9	3.5	-9.7	-13.2	-7.1	-10.7	-18.2	-15.4
		2	------	------	------	------	-3.7	-7.3	-1.0	-4.6	------	------
	30° to 45°	1	19.6	13.5	15.7	10.8	1.5	-11.9	0.6	-10.3	-6.9	-7.9
		2	19.6	13.5	15.7	10.8	7.6	-5.8	6.5	-4.2	-6.9	-7.9
110	0 to 5°	1	19.2	-10.0	12.7	-5.9	-23.1	-13.1	-16.0	-10.1	-32.3	-25.3
	10°	1	21.6	-9.0	14.4	-5.2	-23.1	-14.1	-16.0	-10.8	-32.3	-25.3
	15°	1	24.1	-8.0	16.0	-4.6	-23.1	-15.1	-16.0	-11.5	-32.3	-25.3
	20°	1	26.6	-7.0	17.7	-3.9	-23.1	-16.0	-16.0	-12.2	-32.3	-25.3
	25°	1	24.1	3.9	17.4	4.0	-10.7	-14.6	-7.7	-11.7	-19.9	-17.0
		2	------	------	------	------	-4.1	-7.9	-1.1	-5.1	------	------
	30° to 45°	1	21.6	14.8	17.2	11.8	1.7	-13.1	0.6	-11.3	-7.6	-8.7
		2	21.6	14.8	17.2	11.8	8.3	-6.5	7.2	-4.6	-7.6	-8.7
120	0 to 5°	1	22.8	-11.9	15.1	-7.0	-27.4	-15.6	-19.1	-12.1	-38.4	-30.1
	10°	1	25.8	-10.7	17.1	-6.2	-27.4	-16.8	-19.1	-12.9	-38.4	-30.1
	15°	1	28.7	-9.5	19.1	-5.4	-27.4	-17.9	-19.1	-13.7	-38.4	-30.1
	20°	1	31.6	-8.3	21.1	-4.6	-27.4	-19.1	-19.1	-14.5	-38.4	-30.1
	25°	1	28.6	4.6	20.7	4.7	-12.7	-17.3	-9.2	-13.9	-23.7	-20.2
		2	------	------	------	------	-4.8	-9.4	-1.3	-6.0	------	------
	30° to 45°	1	25.7	17.6	20.4	14.0	2.0	-15.6	0.7	-13.4	-9.0	-10.3
		2	25.7	17.6	20.4	14.0	9.9	-7.7	8.6	-5.5	-9.0	-10.3

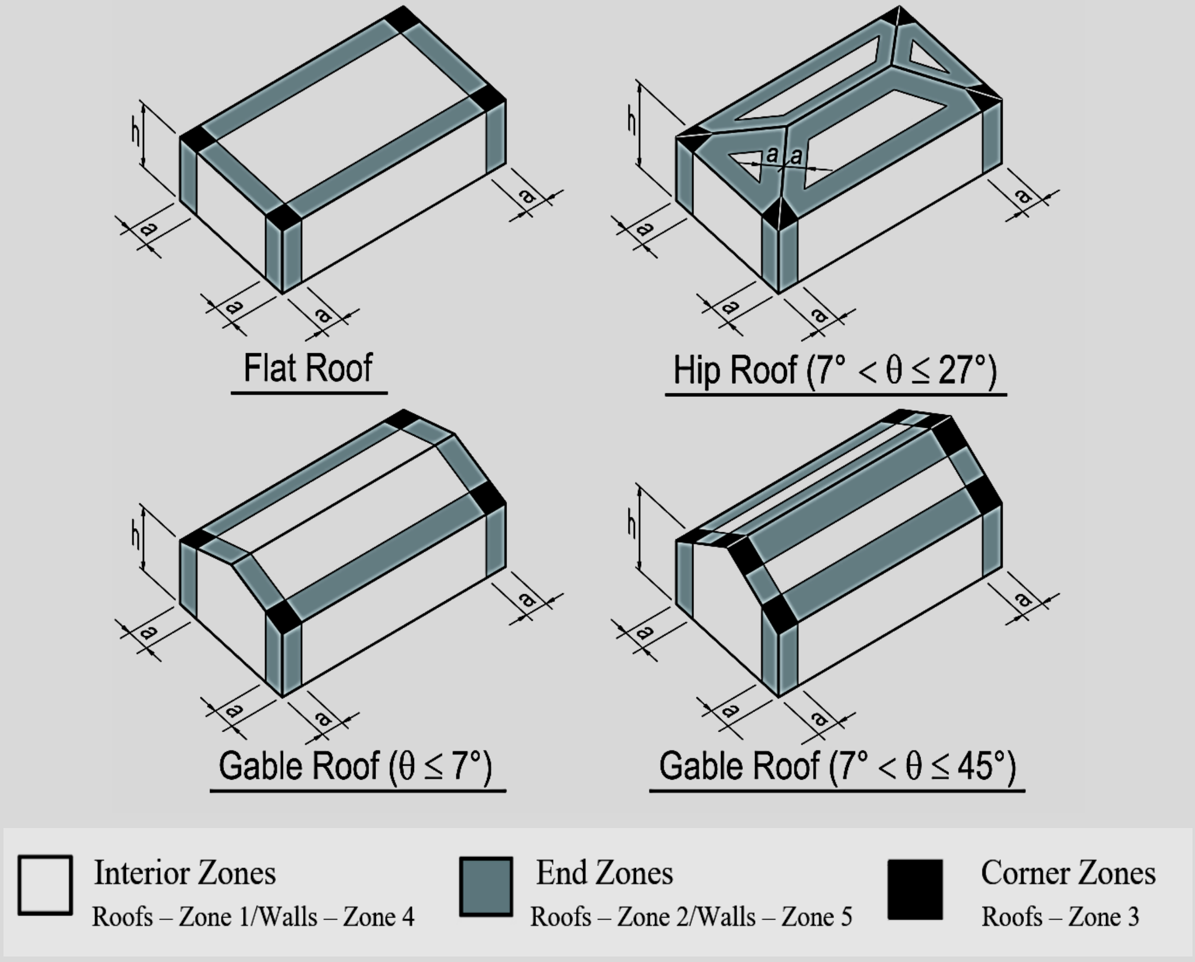
Main Wind Force Resisting System — Method 1											**h ≤ 60 ft.**	
Figure 3.3.1 (cont'd)			**Design Wind Pressure**								**Walls & Roofs**	
Enclosed Buildings												
Simplified Design Wind Pressure, p_{s30} (psf) ***(Exposure B at h = 30 ft., K_{21} = 1.0, with I = 1.0)***												
Basic Wind Speed (mph	**Roof Angle (deg)**	**Load Case**	**Zones**									
			Horizontal Pressures				**Vertical Pressures**				**Overhangs**	
			A	**B**	**C**	**D**	**E**	**F**	**G**	**H**	**E_{OH}**	**G_{OH}**
125	**0 to 5°**	1	24.7	-12.9	16.4	-7.6	-29.7	-16.9	-20.7	-13.1	-41.7	-32.7
	10°	1	28.0	-11.6	18.6	-6.7	-29.7	-18.2	-20.7	-14.0	-41.7	-32.7
	15°	1	31.1	-10.3	20.7	-5.9	-29.7	-19.4	-20.7	-14.9	-41.7	-32.7
	20°	1	34.3	-9.0	22.9	-5.0	-29.7	-20.7	-20.7	-15.7	-41.7	-32.7
	25°	1	31.0	5.0	22.5	5.1	-13.8	-18.8	-10.0	-15.1	-25.7	-21.9
		2	------	------	------	------	-5.2	-10.2	-1.4	-6.5	------	------
	30° to 45°	1	27.9	19.1	22.1	15.2	2.2	-16.9	0.8	-14.5	-9.8	-11.2
		2	27.9	19.1	22.1	15.2	10.7	-8.4	9.3	-6.0	-9.8	-11.2
130	**0 to 5°**	1	26.8	-13.9	17.8	-8.2	-32.2	-18.3	-22.4	-14.2	-45.1	-35.3
	10°	1	30.2	-12.5	20.1	-7.3	-32.2	-19.7	-22.4	-15.1	-45.1	-35.3
	15°	1	33.7	-11.2	22.4	-6.4	-32.2	-21.0	-22.4	-16.1	-45.1	-35.3
	20°	1	37.1	-9.8	24.7	-5.4	-32.2	-22.4	-22.4	-17.0	-45.1	-35.3
	25°	1	33.6	5.4	24.3	5.5	-14.9	-20.4	-10.8	-16.4	-27.8	-23.7
		2	------	------	------	------	-5.7	-11.1	-1.5	-7.1	------	------
	30° to 45°	1	30.1	20.6	24.0	16.5	2.3	-18.3	0.8	-15.7	-10.6	-12.1
		2	30.1	20.6	24.0	16.5	11.6	-9.0	10	-6.4	-10.6	-12.1
140	**0 to 5°**	1	31.1	-16.1	20.6	-9.6	-37.3	-21.2	-26.0	-16.4	-52.3	-40.9
	10°	1	35.1	-14.5	23.3	-8.5	-37.3	-22.8	-26.0	-17.5	-52.3	-40.9
	15°	1	39.0	-12.9	26	-7.4	-37.3	-24.4	-26.0	-18.6	-52.3	-40.9
	20°	1	43.0	-11.4	28.7	-6.3	-37.3	-26.0	-26.0	-19.7	-52.3	-40.9
	25°	1	39.0	6.3	28.2	6.4	-17.3	-23.6	-12.5	-19	-32.3	-27.5
		2	------	------	------	------	-6.6	-12.8	-1.8	-8.2	------	------
	30° to 45°	1	35.0	23.9	27.8	19.1	2.7	-21.2	0.9	-18.2	-12.3	-14.0
		2	35.0	23.9	27.8	19.1	13.4	-10.5	11.7	-7.5	-12.3	-14.0
145	**0 to 5°**	1	33.4	-17.3	22.1	-10.3	-40.0	-22.7	-27.9	-17.6	-56.1	-43.9
	10°	1	37.7	-15.6	25.0	-9.1	-40.0	-24.5	-27.9	-18.8	-56.1	-43.9
	15°	1	41.8	-13.8	27.9	-7.9	-40.0	-26.2	-27.9	-20.0	-56.1	-43.9
	20°	1	46.1	-12.2	30.8	-6.8	-40.0	-27.9	-27.9	-21.1	-56.1	-43.9
	25°	1	41.8	6.8	30.3	6.9	-18.6	-25.3	-13.4	-20.4	-34.6	-29.5
		2	------	------	------	------	-7.1	-13.7	-1.9	-8.8	------	------
	30° to 45°	1	37.5	25.6	29.8	20.5	2.9	-22.7	1.0	-19.5	-13.2	-15.0
		2	35.7	25.6	29.8	20.5	14.4	-11.3	12.6	-8.0	-13.2	-15.0
150	**0 to 5°**	1	35.7	-18.5	23.7	-11.0	-42.9	-24.4	-29.8	-18.9	-60.0	-47.0
	10°	1	40.2	-16.7	26.8	-9.7	-42.9	-26.2	-29.8	-20.1	-60.0	-47.0
	15°	1	44.8	-14.9	29.8	-8.5	-42.9	-28.0	-29.8	-21.4	-60.0	-47.0
	20°	1	49.4	-13.0	32.9	-7.2	-42.9	-29.8	-29.8	-22.6	-60.0	-47.0
	25°	1	44.8	7.2	32.4	7.4	-19.9	-27.1	-14.4	-21.8	-37.0	-31.6
		2	------	------	------	------	-7.5	-14.7	-2.1	-9.4	------	------
	30° to 45°	1	40.1	27.4	31.9	22.0	3.1	-24.4	1.0	-20.9	-14.1	-16.1
		2	40.1	27.4	31.9	22.0	15.4	-12.0	13.4	-8.6	-14.1	-16.1
170	**0 to 5°**	1	45.8	-23.8	30.4	-14.1	-55.1	-31.3	-38.3	-24.2	-77.1	-60.4
	10°	1	51.7	-21.4	34.4	-12.5	-55.1	-33.6	-38.3	-25.8	-77.1	-60.4
	15°	1	57.6	-19.1	38.3	-10.9	-55.1	-36.0	-38.3	-27.5	-77.1	-60.4
	20°	1	63.4	-16.7	42.3	-9.3	-55.1	-38.3	-38.3	-29.1	-77.1	-60.4
	25°	1	57.5	9.3	41.6	9.5	-25.6	-34.8	-18.5	-28	-47.6	-40.5
		2	------	------	------	------	-9.7	-18.9	-2.6	-12.1	------	------
	30° to 45°	1	51.5	35.2	41.0	28.2	4.0	-31.3	1.3	-26.9	-18.1	-20.7
		2	51.5	35.2	41.0	28.2	19.8	-15.4	17.2	-11.0	-18.1	-20.7

Figure 3.3.1 (cont'd)	**Design Wind Pressure**	**Walls & Roofs**
Enclosed Buildings		

Adjustment Factor
for Building Height and Exposure, λ

Mean roof height (ft)	**Exposure**		
	B	**C**	**D**
15	1.00	1.21	1.47
20	1.00	1.29	1.55
25	1.00	1.35	1.61
30	1.00	1.40	1.66
35	1.05	1.45	1.70
40	1.09	1.49	1.74
45	1.12	1.53	1.78
50	1.16	1.56	1.81
55	1.19	1.59	1.84
60	1.22	1.62	1.87

Components and Cladding — Method 1		**h ≤ 60 ft.**
Figure 3.3.2	**Design Wind Pressure**	**Walls & Roofs**
Enclosed Buildings		



Notes:

1. *Pressures shown are applied normal to the surface, for exposure B, at h=30 ft (9.1 m), I = 1.0, and K_{zt} = 1.0. Adjust to other condition using Equation 3.3.2.*
2. *Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.*
3. *For hip roofs with θ ≤ 25°, Zone 3 shall be treated as Zone 2.*
4. *For effective wind areas between those given, value may be interpolated, otherwise use the value associated with the lower effective wind area.*
5. *Notation:*
   a: 10 percent of least horizontal dimension or 0.4h, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m)

   h: mean roof height, in feet (meters), except that eave height shall be used for roof angles < 10°.

   θ : angle of plane of roof from horizontal, in degrees.

Components and Cladding — Method 1		**h ≤ 60 ft.**
Figure 3.3.2 (cont'd)	**Net Design Wind Pressure**	**Walls & Roofs**
Enclosed Buildings		

Net Design Wind Pressure, p_{net30} (psf) *(Exposure B at h = 30 ft., K_{21} = 1.0, with I = 1.0, K_{zt} = 1.0)*

	Zone	Effective wind area (sq. ft)	Basic Wind Speed V (mph) 85		90		100		105		110		120	
Roof 0 to 7 degrees	1	10	5.3	-13.0	5.9	-14.6	7.3	-18.0	8.1	-19.8	8.9	-21.8	10.5	-25.9
	1	20	5.0	-12.7	5.6	-14.2	6.9	-17.5	7.6	-19.3	8.3	-21.2	9.9	-25.2
	1	50	4.5	-12.2	5.1	-13.7	6.3	-16.9	6.9	-18.7	7.6	-20.5	9.0	-24.4
	1	100	4.2	-11.9	4.7	-13.3	5.8	-16.5	6.4	-18.2	7.0	-19.9	8.3	-23.7
	2	10	5.3	-21.8	5.9	-24.4	7.3	-30.2	8.1	-33.3	8.9	-36.5	10.5	-43.5
	2	20	5.0	-19.5	5.6	-21.8	6.9	-27.0	7.6	-29.7	8.3	-32.6	9.9	-38.8
	2	50	4.5	-16.4	5.1	-18.4	6.3	-22.7	6.9	-25.1	7.6	-27.5	9.0	-32.7
	2	100	4.2	-14.1	4.7	-15.8	5.8	-19.5	6.4	-21.5	7.0	-23.6	8.3	-28.1
	3	10	5.3	-32.8	5.9	-36.8	7.3	-45.4	8.1	-50.1	8.9	-55.0	10.5	-65.4
	3	20	5.0	-27.2	5.6	-30.5	6.9	-37.6	7.6	-41.5	8.3	-45.5	9.9	-54.2
	3	50	4.5	-19.7	5.1	-22.1	6.3	-27.3	6.9	-30.1	7.6	-33.1	9.0	-39.3
	3	100	4.2	-14.1	4.7	-15.8	5.8	-19.5	6.4	-21.5	7.0	-23.6	8.3	-28.1
Roof >7 to 27 degrees	1	10	7.5	-11.9	8.4	-13.3	10.4	-16.5	11.4	-18.2	12.5	-19.9	14.9	-23.7
	1	20	6.8	-11.6	7.7	-13.0	9.4	-16.0	10.4	-17.6	11.4	-19.4	13.6	-23.0
	1	50	6.0	-11.1	6.7	-12.5	8.2	-15.4	9.1	-17.0	10.0	-18.6	11.9	-22.2
	1	100	5.3	-10.8	5.9	-12.1	7.3	-14.9	8.1	-16.5	8.9	-18.1	10.5	-21.5
	2	10	7.5	-20.7	8.4	-23.2	10.4	-28.7	11.4	-31.6	12.5	-34.7	14.9	-41.3
	2	20	6.8	-19.0	7.7	-21.4	9.4	-26.4	10.4	-29.1	11.4	-31.9	13.6	-38.0
	2	50	6.0	-16.9	6.7	-18.9	8.2	-23.3	9.1	-25.7	10.0	-28.2	11.9	-33.6
	2	100	5.3	-15.2	5.9	-17.0	7.3	-21.0	8.1	-23.2	8.9	-25.5	10.5	-30.3
	3	10	7.5	-30.6	8.4	-34.3	10.4	-42.4	11.4	-46.7	12.5	-51.3	14.9	-61.0
	3	20	6.8	-28.6	7.7	-32.1	9.4	-39.6	10.4	-43.7	11.4	-47.9	13.6	-57.1
	3	50	6.0	-26.0	6.7	-29.1	8.2	-36.0	9.1	-39.7	10.0	-43.5	11.9	-51.8
	3	100	5.3	-24.0	5.9	-26.9	7.3	-33.2	8.1	-36.6	8.9	-40.2	10.5	-47.9
Roof >27 to 45 degrees	1	10	11.9	-13.0	13.3	-14.6	16.5	-18.0	18.2	-19.8	19.9	-21.8	23.7	-25.9
	1	20	11.6	-12.3	13.0	-13.8	16.0	-17.1	17.6	-18.8	19.4	-20.7	23.0	-24.6
	1	50	11.1	-11.5	12.5	-12.8	15.4	-15.9	17.0	-17.5	18.6	-19.2	22.2	-22.8
	1	100	10.8	-10.8	12.1	-12.1	14.9	-14.9	16.5	-16.5	18.1	-18.1	21.5	-21.5
	2	10	11.9	-15.2	13.3	-17.0	16.5	-21.0	18.2	-23.2	19.9	-25.5	23.7	-30.3
	2	20	11.6	-14.5	13.0	-16.3	16.0	-20.1	17.6	-22.2	19.4	-24.3	23.0	-29.0
	2	50	11.1	-13.7	12.5	-15.3	15.4	-18.9	17.0	-20.8	18.6	-22.9	22.2	-27.2
	2	100	10.8	-13.0	12.1	-14.6	14.9	-18.0	16.5	-19.8	18.1	-21.8	21.5	-25.9
	3	10	11.9	-15.2	13.3	-17.0	16.5	-21.0	18.2	-23.2	19.9	-25.5	23.7	-30.3
	3	20	11.6	-14.5	13.0	-16.3	16.0	-20.1	17.6	-22.2	19.4	-24.3	23.0	-29.0
	3	50	11.1	-13.7	12.5	-15.3	15.4	-18.9	17.0	-20.8	18.6	-22.9	22.2	-27.2
	3	100	10.8	-13.0	12.1	-14.6	14.9	-18.0	16.5	-19.8	18.1	-21.8	21.5	-25.9
Wall	4	10	13.0	-14.1	14.6	-15.8	18.0	-19.5	19.8	-21.5	21.8	-23.6	25.9	-28.1
	4	20	12.4	-13.5	13.9	-15.1	17.2	-18.7	18.9	-20.6	20.8	-22.6	24.7	-26.9
	4	50	11.6	-12.7	13.0	-14.3	16.1	-17.6	17.8	-19.4	19.5	-21.3	23.2	-25.4
	4	100	11.1	-12.2	12.4	-13.6	15.3	-16.8	16.9	-18.5	18.5	-20.4	22.0	-24.2
	4	500	9.7	-10.8	10.9	-12.1	13.4	-14.9	14.8	-16.5	16.2	-18.1	19.3	-21.5
	5	10	13.0	-17.4	14.6	-19.5	18.0	-24.1	19.8	-26.6	21.8	-29.1	25.9	-34.7
	5	20	12.4	-16.2	13.9	-18.2	17.2	-22.5	18.9	-24.8	20.8	-27.2	24.7	-32.4
	5	50	11.6	-14.7	13.0	-16.5	16.1	-20.3	17.8	-22.4	19.5	-24.6	23.2	-29.3
	5	100	11.1	-13.5	12.4	-15.1	15.3	-18.7	16.9	-20.6	18.5	-22.6	22.0	-26.9
	5	500	9.7	-10.8	10.9	-12.1	13.4	-14.9	14.8	-16.5	16.2	-18.1	19.3	-21.5

Components and Cladding — Method 1														
Figure 3.3.2 (cont'd)			**Net Design Wind Pressure**								**h ≤ 60 ft.**			
Enclosed Buildings											**Walls & Roofs**			
Net Design Wind Pressure, p_{net30} (psf) ***(Exposure B at h = 30 ft., K_{21} = 1.0, with I = 1.0, K_{zt} = 1.0)***														
	Zone	**Effective wind area (sq. ft)**	**Basic Wind Speed V (mph)**											
			125		**130**		**140**		**145**		**150**		**170**	
Roof 0 to 7 degrees	1	10	11.4	-28.1	12.4	-30.4	14.3	-35.3	15.4	-37.8	16.5	-40.5	21.1	-52.0
	1	20	10.7	-27.4	11.6	-29.6	13.4	-34.4	14.4	-36.9	15.4	-39.4	19.8	-50.7
	1	50	9.8	-26.4	10.6	-28.6	12.3	-33.2	13.1	-35.6	14.1	-38.1	18.1	-48.9
	1	100	9.1	-25.7	9.8	-27.8	11.4	-32.3	12.2	-34.6	13	-37.0	16.7	-47.6
	2	10	11.4	-47.2	12.4	-51.0	14.3	-59.2	15.4	-63.5	16.5	-67.9	21.1	-87.2
	2	20	10.7	-42.1	11.6	-45.6	13.4	-52.9	14.4	-56.7	15.4	-60.7	19.8	-78.0
	2	50	9.8	-35.5	10.6	-38.4	12.3	-44.5	13.1	-47.8	14.1	-51.1	18.1	-65.7
	2	100	9.1	-30.5	9.8	-33.0	11.4	-38.2	12.2	-41.0	13.0	-43.9	16.7	-56.4
	3	10	11.4	-71.0	12.4	-76.8	14.3	-89.0	15.4	-95.5	16.5	-102.2	21.1	-131.3
	3	20	10.7	-58.5	11.6	-63.6	13.4	-73.8	14.4	-79.1	15.4	-84.7	19.8	-108.7
	3	50	9.8	-42.7	10.6	-46.2	12.3	-53.5	13.1	-57.4	14.1	-61.5	18.1	-78.9
	3	100	9.1	-30.5	9.8	-33.0	11.4	-38.2	12.2	-41.0	13.0	-43.9	16.7	-56.4
Roof >7 to 27 degrees	1	10	16.2	-25.7	17.5	-27.8	20.3	-32.3	21.8	-34.6	23.3	-37.0	30.0	-47.6
	1	20	14.8	-25.0	16.0	-27.0	18.5	-31.4	19.9	-33.7	21.3	-36.0	27.3	-46.3
	1	50	12.9	-24.1	13.9	-26.0	16.1	-30.2	17.3	-32.4	18.5	-34.6	23.8	-44.5
	1	100	11.4	-23.2	12.4	-25.2	14.3	-29.3	15.4	-31.4	16.5	-33.6	21.1	-43.2
	2	10	16.2	-44.8	17.5	-48.4	20.3	-56.2	21.8	-60.3	23.3	-64.5	30.0	-82.8
	2	20	14.8	-41.2	16.0	-44.6	18.5	-51.7	19.9	-55.4	21.3	-59.3	27.3	-76.2
	2	50	12.9	-36.5	13.9	-39.4	16.1	-45.7	17.3	-49.1	18.5	-52.5	23.8	-67.4
	2	100	11.4	-32.9	12.4	-35.6	14.3	-41.2	15.4	-44.2	16.5	-47.3	21.1	-60.8
	3	10	16.2	-66.2	17.5	-71.6	20.3	-83.1	21.8	-89.1	23.3	-95.4	30.0	-122.5
	3	20	14.8	-61.9	16.0	-67.0	18.5	-77.7	19.9	-83.3	21.3	-89.2	27.3	-114.5
	3	50	12.9	-56.2	13.9	-60.8	16.1	-70.5	17.3	-75.7	18.5	-81.0	23.8	-104.0
	3	100	11.4	-51.9	12.4	-56.2	14.3	-65.1	15.4	-69.9	16.5	-74.8	21.1	-96.0
Roof >27 to 45 degrees	1	10	25.7	-28.1	27.8	-30.4	32.3	-35.3	34.6	-37.8	37.0	-40.5	47.6	-52.0
	1	20	25.0	-26.7	27.0	-28.9	31.4	-33.5	33.7	-35.9	36.0	-38.4	46.3	-49.3
	1	50	24.1	-24.8	26.0	-26.8	30.2	-31.1	32.4	-33.3	34.6	-35.7	44.5	-45.8
	1	100	23.3	-23.3	25.2	-25.2	29.3	-29.3	31.4	-31.4	33.6	-33.6	43.2	-43.2
	2	10	25.7	-32.9	27.8	-35.6	32.3	-41.2	34.6	-44.2	37.0	-47.3	47.6	-60.8
	2	20	25.0	-31.4	27.0	-34.0	31.4	-39.4	33.7	-42.3	36.0	-45.3	46.3	-58.1
	2	50	24.1	-29.5	26.0	-32.0	30.2	-37.1	32.4	-39.8	34.6	-42.5	44.5	-54.6
	2	100	23.2	-28.1	25.2	-30.4	29.3	-35.3	31.4	-37.8	33.6	-40.5	43.2	-52.0
	3	10	25.7	-32.9	27.8	-35.6	32.3	-41.2	34.6	-44.2	37.0	-47.3	47.6	-60.8
	3	20	25.0	-31.4	27.0	-34.0	31.4	-39.4	33.7	-42.3	36.0	-45.3	46.3	-58.1
	3	50	24.1	-29.5	26.0	-32.0	30.2	-37.1	32.4	-39.8	34.6	-42.5	44.5	-54.6
	3	100	23.3	-28.1	25.2	-30.4	29.3	-35.3	31.4	-37.8	33.6	-40.5	43.2	-52.0
Wall	4	10	28.1	-30.5	30.4	-33.0	35.3	-38.2	37.8	-41	40.5	-43.9	52.0	-56.4
	4	20	26.8	-29.2	29.0	-31.6	33.7	-36.7	36.1	-39.3	38.7	-42.1	49.6	-54.1
	4	50	25.2	-27.5	27.2	-29.8	31.6	-34.6	33.9	-37.1	36.2	-39.7	46.6	-51.0
	4	100	23.9	-26.3	25.9	-28.4	30.0	-33.0	32.2	-35.4	34.4	-37.8	44.2	-48.6
	4	500	21.0	-23.3	22.7	-25.2	26.3	-29.3	28.2	-31.4	30.2	-33.6	38.8	-43.2
	5	10	28.1	-37.6	30.4	-40.7	35.3	-47.2	37.8	-50.6	40.5	-54.2	52.0	-69.6
	5	20	26.8	-35.1	29.0	-38.0	33.7	-44.0	36.1	-47.2	38.7	-50.5	49.6	-64.9
	5	50	25.2	-31.8	27.2	-34.3	31.6	-39.8	33.9	-42.7	36.2	-45.7	46.6	-58.7
	5	100	23.9	-29.2	25.9	-31.6	30.0	-36.7	32.2	-39.3	34.4	-42.1	44.2	-54.1
	5	500	21.0	-23.2	22.7	-25.2	26.3	-29.3	28.2	-31.1	30.2	-33.6	38.8	-43.2

Components and Cladding — Method 1		h ≤ 60 ft.
Figure 3.3.2 (cont'd)	Net Design Wind Pressure	Walls & Roofs
Enclosed Buildings		

Roof Overhang Net Design Wind Pressure, p_{net30} (psf)
(Exposure B at h = 30 ft. with I = 1.0)

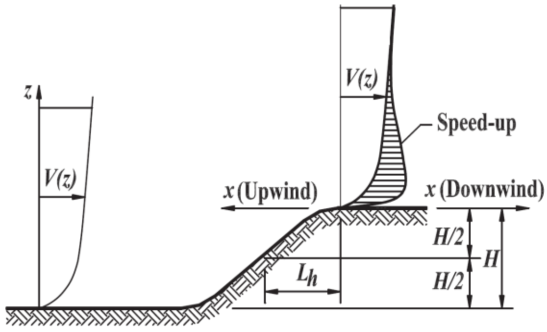
	Zone	Effective wind area (sq. ft)	Basic Wind Speed V (mph) 90	100	110	120	130	140	150	170
Roof 0 to 7 degrees	2	10	-21.0	-25.9	-31.4	-37.3	-43.8	-50.8	-58.3	-74.9
	2	20	-20.6	-25.5	-30.8	-36.7	-43.0	-49.9	-57.3	-73.6
	2	50	-20.1	-24.9	-30.1	-35.8	-42.0	-48.7	-55.9	-71.8
	2	100	-19.8	-24.4	-29.5	-35.1	-41.2	-47.8	-54.9	-70.5
	3	10	-34.6	-42.7	-51.6	-61.5	-72.1	-83.7	-96.0	-123.4
	3	20	-27.1	-33.5	-40.5	-48.3	-56.6	-65.7	-75.4	-96.8
	3	50	-17.3	-21.4	-25.9	-30.8	-36.1	-41.9	-48.1	-61.8
	3	100	-10.0	-12.2	-14.8	-17.6	-20.6	-23.9	-27.4	-35.2
Roof >7 to 27 degrees	2	10	-27.2	-33.5	-40.6	-48.3	-56.7	-65.7	-75.5	-96.9
	2	20	-27.2	-33.5	-40.6	-48.3	-56.7	-65.7	-75.5	-96.9
	2	50	-27.2	-33.5	-40.6	-48.3	-56.7	-65.7	-75.5	-96.9
	2	100	-27.2	-33.5	-40.6	-48.3	-56.7	-65.7	-75.5	-96.9
	3	10	-45.7	-56.4	-68.3	-81.2	-95.3	-110.6	-126.9	-163.0
	3	20	-41.2	-50.9	-61.6	-73.3	-86.0	-99.8	-114.5	-147.1
	3	50	-35.3	-43.6	-52.8	-62.8	-73.7	-85.5	-98.1	-126.1
	3	100	-30.9	-38.1	-46.1	-54.9	-64.4	-74.7	-85.8	-110.1
Roof >27 to 45 degrees	2	10	-24.7	-30.5	-36.9	-43.9	-51.5	-59.8	-68.6	-88.1
	2	20	-24.0	-29.6	-35.8	-42.6	-50.0	-58.0	-66.5	-85.5
	2	50	-23.0	-28.4	-34.3	-40.8	-47.9	-55.6	-63.8	-82.0
	2	100	-22.2	-27.4	-33.2	-39.5	-46.4	-53.8	-61.7	-79.3
	3	10	-24.7	-30.5	-36.9	-43.9	-51.5	-59.8	-68.6	-88.1
	3	20	-24.0	-29.6	-35.8	-42.6	-50.0	-58.0	-66.5	-85.5
	3	50	-23.0	-28.4	-34.3	-40.8	-47.9	-55.6	-63.8	-82.0
	3	100	-22.2	-27.4	-33.2	-39.5	-46.4	-53.8	-61.7	-79.3

Adjustment Factor
for Building Height and Exposure, λ

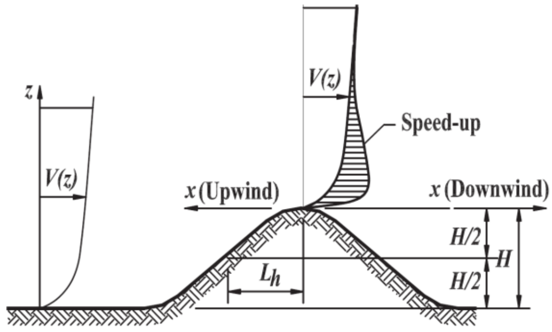
Mean roof height (ft)	Exposure B	C	D
15	1.00	1.21	1.47
20	1.00	1.29	1.55
25	1.00	1.35	1.61
30	1.00	1.40	1.66
35	1.05	1.45	1.70
40	1.09	1.49	1.74
45	1.12	1.53	1.78
50	1.16	1.56	1.81
55	1.19	1.59	1.84
60	1.22	1.62	1.87

Topographic Factor, Kzt

Figure 3.3.3



ESCARPMENT



2-D RIDGE OR 3-D AXISYMMETRICAL HILL

Topographic multipliers for Exposure C										
	K_1 multiplier				K_2 multiplier			K_3 multiplier		
H/L_h	2-D Ridge	2-D Escarp.	3-D Axisym. Hill	x/L_h	2-D Escarp.	All Other Cases	z/L_h	2-D Ridge	2-D Escarp.	3-D Axisym. Hill
0.20	0.29	0.17	0.21	0.00	1.00	1.00	0.00	1.00	1.00	1.00
0.25	0.36	0.21	0.26	0.50	0.88	0.67	0.10	0.74	0.78	0.67
0.30	0.43	0.26	0.32	1.00	0.75	0.33	0.20	0.55	0.61	0.45
0.35	0.51	0.30	0.37	1.50	0.63	0.00	0.30	0.41	0.47	0.30
0.40	0.58	0.34	0.42	2.00	0.50	0.00	0.40	0.30	0.37	0.20
0.45	0.65	0.38	0.47	2.50	0.38	0.00	0.50	0.22	0.29	0.14
0.50	0.72	0.43	0.53	3.00	0.25	0.00	0.60	0.17	0.22	0.09
				3.50	0.13	0.00	0.70	0.12	0.17	0.06
				4.00	0.00	0.00	0.80	0.09	0.14	0.04
							0.90	0.07	0.11	0.03
							1.00	0.05	0.08	0.02
							1.50	0.01	0.02	0.00
							2.00	0.00	0.00	0.00

Notes:

1. For values of H/L_h, x/L_h and z/L_h other than those shown, linear interpolation is permitted.
2. For $H/L_h > 0.5$, assume $H/L_h = 0.5$ for evaluating K_1 and substitute 2Hfor L_h for evaluating K_2 and K_3.
3. Multipliers are based on the assumption that wind approaches the hill or escarpment along the direction of maximum slope.
4. Notation:
   - *H* : Height of hill or escarpment relative to the upwind terrain, in feet (meters).
   - L_h : Distance upwind of crest to where the difference in ground elevation is half the height of hill or escarpment, in feet (meters).
   - K_1 : Factor to account for shape of topographic feature and maximum speed-up effect.
   - K_2 : Factor to account for reduction in speed-up with distance upwind or downwind of crest.
   - K_3 : Factor to account for reduction in speed-up with height above local terrain.
   - *x* : Distance (upwind or downwind) from the crest to the building site, in feet (meters).
   - *z* : Height above local ground level, in feet (meters).
   - μ : Horizontal attenuation factor.
   - γ : Height attenuation factor.

Topographic Factor, Kzt

Figure 3.3.3 (cont'd)

Equations:

$$K_{zt} = (1 + K_1 K_2 K_3)^2$$

$$\mathrm{K}_1 = \text{determined from table below}$$

$$K_2 = (1 - \frac{|x|}{\mu L_h})$$

$$K_3 = e^{-gz/L_h}$$

Parameters for Speed-Up Over Hills and Escarpments						
Hill Shape	**$K_1/(H/L_h)$**			**γ**	**μ**	
	Exposure				**Upwind of Crest**	**Downwind of Crest**
	B	**C**	**D**			
2-dimensional ridges (or valleys with negative H in $K_1/(H/L_h)$)	1.30	1.45	1.55	3	1.5	1.5
2-dimensional escarpments	0.75	0.85	0.95	2.5	1.5	4
3-dimensional axisym. hill	0.95	1.05	1.15	4	1.5	1.5

TABLE 3.3.1 BASIC WIND SPEED (3 SEC GUST WIND SPEED IN MPH)

Sr. No.	City	Lat.	Long.	Basic Wind Speed (mph)
1	**Bago**	17.33	96.48	80
2	**Bamaw**	24.27	97.20	70
3	**Bogalay**	16.30	95.40	100
4	**Chauk**	20.90	94.83	70
5	**Dawei**	14.10	98.22	90
6	**Falam**	22.91	93.68	70
7	**Hakha**	22.65	93.62	90
8	**Henzada**	17.65	95.46	90
9	**Homalin**	24.87	94.92	50
10	**Hpa-An**	16.75	97.67	70
11	**Kale**	23.18	94.05	70
12	**Kawthaung**	10.00	98.55	90
13	**Kengtung**	21.30	99.62	70
14	**Kyaukpyu**	19.42	93.55	130
15	**Lashio**	22.93	97.75	70
16	**Loikaw**	19.68	97.22	70
17	**Magwe**	20.15	94.17	70
18	**Mandalay**	21.93	96.10	80
19	**Mawlamyine**	16.50	97.62	90
20	**Meiktila**	20.83	95.83	70
21	**Monywa**	22.08	95.00	70
22	**Muse**	23.98	97.90	70
23	**Myeik**	12.43	98.60	90
24	**Myitkyina**	25.37	97.40	70
25	**Nansam**	20.90	97.71	70
26	**Naypyitaw**	19.75	96.10	80
27	**Pakokku**	21.33	95.08	70
28	**Pathein**	16.78	94.73	100
29	**Putao**	27.33	97.42	70
30	**Pyay**	18.80	95.22	70
31	**Sittwe**	20.13	92.88	130
32	**Taunggyi**	20.78	97.03	70
33	**Thandwe**	18.47	94.35	130
34	**Yangon**	16.77	96.17	100
35	**Ye**	15.26	97.85	90
36	**Yenangyaung**	20.46	94.87	70
Note: For cities not included in the table, wind speed of the nearest city in the list shall be used.				

TABLE 3.3.2 IMPORTANCE FACTOR (WIND LOADS)

Category	Non-Cyclone Prone Regions and Cyclone Prone Regions with V = 85-100 mph	Cyclone Prone Regions with V > 100 mph
I	0.87	0.77
II	1	1
III	1.15	1.15
IV	1.15	1.15
Note: The building categories are listed in Table 3.1.2		

SECTION 3.4

SEISMIC DESIGN CRITERIA AND DESIGN REQUIREMENTS FOR BUILDINGS

3.4.1 Seismic Design Criteria

3.4.1.1 General

3.4.1.1.1 Purpose

This section presents criteria for the design and construction of buildings and other structures subject to earthquake ground motions. 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.

3.4.1.1.2 Scope

Every structure, and portion thereof, including nonstructural components that are permanently attached to structures and their supports and attachments, shall be designed and constructed to resist the effects of earthquake motions in accordance with this code and ASCE 7-05, excluding Chapter 14 and Appendix 11A. The seismic design category for a structure is permitted to be determined in accordance with Section 3.4 or ASCE 7-05.

EXCEPTIONS:

1. Detached one- and two-family dwellings, assigned to Seismic Design Category A, B or C, or located where the mapped short-period spectral response acceleration, S_S, is less than 0.4 g.
2. The seismic-force-resisting system of wood-frame buildings that conform to the provisions of Section 2308 (IBC-2006) are not required to be analyzed as specified in this section.
3. Agricultural storage structures intended only for incidental human occupancy.
4. Structures that require special consideration of their response characteristics and environment that are not addressed by this code or ASCE 7-05 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.

3.4.1.1.3 Applicability

Structures and their nonstructural components shall be designed and constructed in accordance with the requirement of Section 3.4.2.

3.4.1.1.4 Alternate Materials and Methods of Construction

Alternate materials and methods of construction to those prescribed in the seismic requirements of this standard shall not be used unless approved by the authority having jurisdiction. Substantiating evidence shall be submitted demonstrating that the proposed alternate, for the purpose intended, will be at least equal in strength, durability, and seismic resistance.

3.4.1.2 Definitions

The following definitions apply only to the seismic requirements of this standard.

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 authority Department).

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

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

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.

ATTACHMENTS: 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.

BASE: The level at which the horizontal seismic ground motions are considered to be imparted to the structure.

BASEMENT: A basement is any storey below the lowest storey above grade.

BASE SHEAR: Total design lateral force or shear at the base.

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.

BUILDING: Any structure whose intended use includes shelter of human occupants.

CANTILEVERED COLUMN SYSTEM: A seismic force-resisting system in which lateral forces are resisted entirely by columns acting as cantilevers from the base.

CHARACTERISTIC EARTHQUAKE: An earthquake assessed for an active fault having a magnitude equal to the best estimate of the maximum magnitude capable of occurring on the fault, but not less than the largest magnitude that has occurred historically on the fault.

COMPONENT: A part of element of an architectural, electrical, mechanical, or structural system.
Component, Equipment: A mechanical or electrical component or element that is part of a mechanical and/or electrical system within or without a building system.

Component, Flexible: Component, including its attachments, having a fundamental period greater than 0.06s.

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

COMPONENT SUPPORT: Those structural members or assemblies of members, including braces, frames, struts, and attachments that transmit all loads and forces between systems, components, or elements and the structures.

CONCRETE, PLAIN: Concrete that is either unreinforced or contains less reinforcement than the minimum amount specified in ACI 318-08 for reinforced concrete.

CONCRETE, REINFORCED: Concrete reinforced with no less reinforcement than the minimum amount required by ACI 318-08 prestressed or non-prestressed, and designed on the assumption that the two materials act together in resisting forces.

CONSTRUCTION DOCUMENTS: The written, graphic, electronic and pictorial documents describing design, locations and physical characteristics of the project required to verify compliance with this standard.

COUPLING BEAM: A beam that is used to connect adjacent concrete wall elements to make them act together as a unit to resist lateral loads.

DEFORMABILITY: The ratio of the ultimate deformation to the limit deformation.

High-Deformability Element: An element whose deformability is not less than 3.5 where subjected to four fully reversed cycles at the limit deformation.

Limited-Deformability Element: An element that is neither low- deformability or a high-deformability element.

Low-Deformability Element: An element whose deformability is 1.5 or less.

DEFORMATION:

Limit Deformation: Two times the initial deformation that occurs at a load equal to 40 percent of the maximum strength.

Ultimate Deformation: The deformation at which failure occurs and that shall be deemed to occur if the sustainable load reduces to 80 percent or less of the maximum strength.

DESIGNATED SEISMIC SYSTEMS: The seismic force-resisting system and those architectural, electrical, and mechanical system or their components and for which the component importance factor, *Ip*, is greater than 1.0.

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.

ENCLOSURE: An interior space surrounded by walls.

EQUIPMENT SUPPORT: Those structural members or assemblies of members or manufactured elements, including braces, frames, legs, lugs, snuggers, hangers, or saddles that transmit gravity loads and operating loads between the equipment and the structure.

FLEXIBLE EQUIPMENT CONNECTIONS: Those connections between equipment components that permit rotational and/or translational movement without degradation of performance. Examples include universal joints, bellows expansion joints, and flexible metal hose.

FRAME:

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.

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).

Eccentrically Braced Frame (EBF): A diagonally braced frame in which at least one end of each frames into a beam a short distance from a beam-column or from another diagonal brace.

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).

STRUCTURAL SYSTEM:

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.

Dual System: A structural system with an essentially complete space frame providing support for vertical loads. Seismic force resistance is provided by moment resisting fames and shear walls or braced frames as prescribed in Section 3.4.2.5.1.

Shear Wall-Frame Interactive System: A structural system that uses combinations of ordinary reinforced concrete shear walls 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.

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.

GLAZED CURTAIN WALL: A nonbearing wall that extends beyond the edges of building floor slabs, and includes a glazing material installed in the curtain wall framing.

GLAZED STOREFRONT: A nonbearing wall that is installed between floor slabs, typically including entrances, and includes a glazing material installed in the storefront framing.

GRADE PLANE: A reference plane representing the average of finished ground level adjoining the structure at all exterior walls. Where the finished ground level slopes away from the exterior walls, the reference plane shall be established by the lowest points within the area between the buildings and the lot line or, where the lot line is more than 6 ft (1,829 mm) from the structure, between the structure and points 6 ft (1,829 mm) from the structure.

HAZARDOUS CONTENTS: A material that is highly toxic or potentially explosive and in sufficient quantity to pose a significant life- safety threat to the general public if an uncontrolled release were to occur.

IMPORTANCE FACTOR: A factor assigned to each structure according to its Occupancy Category as prescribed in Section 3.4.1.5.

INSPECTION, SPECIAL: The observation of the work by a special inspector to determine compliance with the approved construction documents and these standards in accordance with the quality assurance plan.

Continuous Special Inspection: The full-time or intermittent observation of the work by a special inspector who is present in the area where work is being performed.

Periodic Special Inspection: The part-time or intermittent observation of the work by a special inspector who is present in the area where work has been or is being performed.

INSPECTOR, SPECIAL (who shall be identified as the owner's inspector): A person approved by the authority having jurisdiction to perform special inspection.

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.

JOINT: The geometric volume common to intersecting members.

LIGHT-FRAME CONSTRUCTION: A method of construction where the structural assemblies (e.g., walls, floors, ceilings and roofs) are primarily formed by a system of repetitive wood or cold- formed steel framing members of subassemblies of these members (e.g., trusses).

LONGITUDINAL REINFORCEMENT RATIO: Area of longitudinal reinforcement divided by the cross-sectional area of the concrete.

MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION: The most severe earthquake effects considered by this standard as defined in Section 3.4.1.4.

MECHANICALLY ANCHORED TANKS OR VESSELS: Tanks or vessels provided with mechanical anchors to resist overturning moments.

NONBUILDING STRUCTURE: A structure, other than a building, constructed of a type included in Chapter 15 (ASCE 7-05) and within the limits of Section 15.1.1 (ASCE 7-05).

NONBUILDING STRUCTURE SIMILAR TO A BUILDING: A non-building 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 Table 15.4-1(ASCE 7-05).

ORTHOGONAL: To be in two horizontal directions, at 90° to each other.

OWNER: Any person, agent, firm, or corporation having a legal or equitable interest in the property.

PARTITION: A nonstructural interior wall that spans horizontally or vertically from support to support. The supports may be the basic building frame, subsidiary structural members, or other portions of the partition system.

P-DELTA EFFECT: The secondary effect on shears and moments of structural members due to the action of the vertical loads induced by horizontal displacement of the structure resulting from various loading conditions.

PILE: Deep foundation components including piers, caissons, and piles.

PILE CAP: Foundation elements to which piles are connected including grade beams and mats.

REGISTERED STRUCTURAL DESIGN PROFESSIONAL: An engineer, registered or licensed to practice professional engineering, as defined by the statutory requirements of the professional registrations laws of the state in which the project is to be constructed.

SEISMIC DESIGN CATEGORY: A classification assigned to a structure based on its Occupancy Category and the severity of the design earthquake ground motion at the site as defined in Section 3.4.1.6.

SEISMIC FORCE-RESISTING SYSTEM: That part of the structural system that has been considered in the design to provide the required resistance to the seismic forces prescribed herein.

SEISMIC FORCES: The assumed forces prescribed herein, related to the response of the structure to earthquake motions, to be used in the design of the structure and its components.

SELF-ANCHORED TANKS OR VESSELS: Tanks or vessels that is stable under design overturning moment without the need for mechanical anchors to resist uplift.

SHEAR PANEL: A floor, roof, or wall component sheathed to act as a shear wall or diaphragm.

SITE CLASS: A classification assigned to a site based on the types of soils present and their engineering properties.

STORAGE RACKS: Include industrial pallet racks, moveable shelf racks, and stacker racks made of cold-formed or hot-rolled structural members. Does not include other types of racks such as drive-in and drive-through racks, cantilever racks, portable racks, or racks made of materials other than steel.

STOREY: The portion of a structure between the tops of two successive finished floor surfaces and, for the topmost storey, from the top of the floor finish to the top of the roof structural element.

STOREY ABOVE GRADE: Any storey having its finished floor surface entirely above grade, except that a storey shall be considered as a storey above grade where the finished floor surface of the storey immediately above is more than 6 ft (1,829 mm) above the grade plane, more than 6 ft (1,829 mm) above the finished ground level for more than 40 percent of the total structure perimeter, or more than 12 ft (3,658 mm) above the finished ground level at any point.

STOREY DRIFT: The horizontal deflection at the top of the storey relative to the bottom of the storey as determined in Section 12.8.6 in ASCE 7-05.

STOREYS DRIFT RATIO: The storey drift, as determined in Section 12.8.6 in ASCE 7-05divided by the storey height.

STOREY SHEAR: The summation of design lateral seismic forces at levels above the storey under consideration.

STRENGTH:

Design Strength: Nominal strength multiplied by a strength reduction factor, ϕ.

Nominal Strength: Strength of a member or cross-section calculated in accordance with the requirements and assumptions of the strength design methods of this standard (or the reference documents) before application of any strength- reduction factors.

Required Strength: Strength of a member, cross-section, or connection required to resist factored loads or related internal moments and forces in such combinations as stipulated by this standard.

STRUCTURAL OBSERVATIONS: The visual observations to determine that the seismic force-resisting system is constructed in general conformance with the construction documents.

STRUCTURE: That which is built or constructed and limited to buildings and non-building structures as defined herein.

SUBDIAPHRAGM: A portion of a diaphragm used to transfer wall anchorage forces to diaphragm cross ties.

SUPPORTS: Those structural members, assemblies of members, or manufactured elements, including braces, frames, legs, lugs, snubbers, hangers, saddles, or struts, which transmit loads between the nonstructural components and the structure.

TESTING AGENCY: A company or corporation that provides testing and/or inspection services.

VENEERS: Facings or ornamentation of brick, concrete, stone, tile, or similar materials attached to a backing.

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 non- bearing, 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.

3.4.1.3 Notation

The unit dimensions used with the items covered by the symbols shall be consistent throughout except where specifically noted. Notation presented in this section applies only to the seismic requirements in this standard as indicated.

A_{ch} = cross-sectional area (in^2 or mm^2) of a structural member measured out-to-out of transverse reinforcement

A_0 = area of the load-carrying foundation (ft^2 or m^2)

A_{sh} = total cross-sectional area of hoop reinforcement (in^2 or mm^2), including supplementary cross-ties, having a spacing of s_h and crossing a section with a core dimension of h_c

A_{vd} = required area of leg (in^2 or mm^2) of diagonal reinforcement

A_x = torsional amplification factor (Section 12.8.4.3 of ASCE 7-05)

a_i = the acceleration at level i obtained from a modal analysis

a_p = the amplification factor related to the response of a system or component as affected by the type of seismic attachment.

b_p = the width of the rectangular glass panel

C_d = deflection amplification factor as given in Table 3.4.8.

C_s = seismic response coefficient determined in Section 3.4.8.1.1. (dimensionless)

C_t = building period coefficient in Section 3.4.8.2.1

C_{vx} = vertical distribution factor as determined in Section 12.8.3 of ASCE 7-05

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

D_{clear} = relative horizontal (drift) displacement, measured over the height of the glass panel under consideration, which causes initial glass-to-frame contact

d_C = Total thickness of cohesive soil layers in the top 100 ft (30 m); see Section 3.4.1.4.8 (ft or m)

d_i = The thickness of any soil or rock layer i (between 0 and 100 ft [30 m]); see Section 3.4.1.4.8 (ft or m)

d_S = The total thickness of cohesionless soil layers in the top 100 ft (30 m); see Section 3.4.1.4.8 (ft or m)

E = effect of horizontal and vertical earthquake- induced forces (see Section 12.4 of ASCE 7-05)

F_a = short-period site coefficient (at 0.2 s-period); see Section 3.4.1.4.3

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 of ASCE 7-05

F_p = the seismic force acting on a component of a structure as determined in Section 13.3.1 of ASCE 7-05)

F_v = long-period site coefficient (at 1.0 s-period); see Section 3.4.1.4.3

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 = $\gamma\, v_s^2 / g$

= the average shear modulus for the soils beneath the foundation at large strain levels (psf or Pa)

G_0 = $\gamma\, v^2_{so} / g$

= the average shear modulus for the soils beneath the foundation at small strain levels (psf or Pa)

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

h_c = core dimension of a component measured to the outside of the special lateral reinforcement (in. or mm)

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

h_p = the height of the rectangular glass panel

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

I = the importance factor in Section 3.4.1.5.1

I_p = the component importance factor

i = the building level referred to by the subscript i ;i= 1

K_p = the stiffness of the component or attachment

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 of ASCE 7-05

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 centre of mass and the centre of rigidity Section 12.8.4.1 of ASCE 7-05

M_{ta} = accidental torsional moment as determined in Section 12.8.4.1 of ASCE 7-05

N = standard penetration resistance, ASTM 1586

N = number of storeys (Section 3.4.8.2.1)

$\bar{N}$ = average field standard penetration resistance for the top 100 ft (30 m); see Section 3.4.1.4.8

$\bar{N}_{ch}$ = average standard penetration resistance for cohesionless soil layers for the top 100 ft (30 m); see Section 3.4.1.4.8

N_i = standard penetration resistance of any soil or rock layer *i* [between 0 and 100 ft (30 m)]; see Section 3.4.1.4.8

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 of ASCE 7-05)

PI = plasticity index, ASTM D4318

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

R = response modification coefficient as given in Table 3.4.8

R_p = component response modification factor

S_S = specified MCE, 5 percent damped, spectral response acceleration parameter at short periods as defined in Section 3.4.1.4.1

S_1 = specified MCE, 5 percent damped, spectral response acceleration parameter at a period of 1 s as defined in Section 3.4.1.4.1

S_{aM} = the site-specific MCE spectral response acceleration at any period. (Chapter 21 of ASCE 7-05)

S_{DS} = design, 5 percent damped, spectral response acceleration parameter at short periods as defined in Section 3.4.1.4.4

S_{D1} = design, 5 percent damped, spectral response acceleration parameter at a period of 1 s as defined in Section 3.4.1.4.4

S_{MS} = the MCE, 5 percent damped, spectral response acceleration at short periods adjusted for site class effects as defined in Section 3.4.1.4.3

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 3.4.1.4.3

s_u = undrained shear strength; see Section 3.4.1.4.8

$\bar{s}_u$ = average undrained shear strength in top 100 ft (30 m); see Sections 3.4.1.4.8, 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 3.4.1.4.8

s_h = spacing of special lateral reinforcement (in. or mm)

T = the fundamental period of the building

T_a = approximate fundamental period of the building as determined in Section 3.4.8.2.1

T_L = long-period transition period as defined in Section 3.4.1.4.5 (Table 3.4.1)

T_p = fundamental period of the component and its attachment

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 of ASCE 7-05

V_x = seismic design shear in storey *x* as determined in Section 12.8.4 or 12.9.4 of ASCE 7-05

v_s = shear wave velocity at small shear strains (equal to 10^{-3} percent strain or less); see Section 3.4.1.4.8(ft/s or m/s)

$\bar{v}_s$ = average shear wave velocity at small shear strains in top 100 ft (30 m); see Sections 3.4.1.4.8

v_{si} = the shear wave velocity of any soil or rock layer i (between 0 and 100 ft (30 m)); see Section 3.4.1.4.8

W = effective seismic weight of the building as defined in Section 3.4.8.3

W_c = gravity load of a component of the building

W_p = component operating weight (lb or N) Chapter 15 of ASCE 7-05

w = moisture content (in percent), ASTM D2216

w_i, w_n, w_x = portion of W that is located at or assigned to Level i, n, or x, respectively Section 12.8.3 of ASCE 7-05

x = level under consideration, 1 designates the first level above the base Section 12.8.3 of ASCE 7-05

z = height in structure of point of attachment of component with respect to the base

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

β_0 = foundation damping factor Section 19.2 of ASCE 7-05

γ = average unit weight of soil (lb/ft^3 or N/m^3)

Δ = design storey drift as determined in Section 3.4.8.6

$\Delta_{fallout}$ = the relative seismic displacement (drift) at which glass fallout from the curtain wall, storefront, or partition occurs Section 12.8.3 of ASCE 7-05

Δ_a = allowable storey drift as specified in Section 12.8.3 of ASCE 7-05

δ_{max} = maximum displacement at Level x, considering torsion, Section 12.8.3 of ASCE 7-05

δ_{avg} = the average of the displacements at the extreme points of the structure at Level x, Section 12.8.3 of ASCE 7-05

δ_x = deflection of Level x at the centre of the mass at and above Level x, Eq. (3.4.22) Section 12.8.3 of ASCE 7-05

δ_{xe} = deflection of Level x at the centre of the mass at and above Level x determined by an elastic analysis, Section 12.8.3 of ASCE 7-05

θ = stability coefficient for P -delta effects as determined in Section 12.8.3 of ASCE 7-05

ρ = a redundancy factor based on the extent of structural redundancy present in a building as defined in Section 12.8.3 of ASCE 7-05

λ = time effect factor Section 12.8.3 of ASCE 7-05

Ω_0 = overstrength factor as defined in Table 3.4.8.

3.4.1.4 Seismic Ground Motion Values

3.4.1.4.1 Specified Acceleration Parameters

The parameters S_S and S_1 shall be determined from the 0.2s and 1.0 s spectral response accelerations in Table 3.4.1. Where S_1 is less than or equal to 0.04 and S_S is less than or equal to 0.15, the structure is permitted to be assigned to Seismic Design Category A and is only required to comply with (Section 11.7 of ASCE 7-05).

3.4.1.4.2 Site Class Definitions

Based on the site soil properties, the site shall be classified as Site Class A, B, C, D, E, or F in accordance with Table 3.4.2. Where the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used unless the authority having jurisdiction or geotechnical data determines Site Class E or F soils are present at the site.

3.4.1.4.3 Site Coefficients and Adjusted Maximum Considered Earthquake (MCE) Spectral Response Acceleration Parameters

The MCE spectral response acceleration for short periods (S_{MS}) and at 1 s (S_{M1}), adjusted for Site Class effects, shall be determined by Eqs. (3.4.1) and (3.4.2), respectively.

$$S_{MS} = F_a S_S \qquad \textit{Eq. (3.4.1)}$$

$$S_{M1} = F_v S_1 \qquad \textit{Eq. (3.4.2)}$$

where,

S_S = the MCE spectral response acceleration at short periods as determined from Table 3.4.1 and Figure 3.4.1

S_1 = the MCE spectral response acceleration at a period of 1s as determined from Table 3.4.1 and Figure 3.4.2

where site coefficients F_a and F_v are defined in Tables 3.4.3 and 3.4.4, respectively. Where the simplified design procedure of Section 3.4.1.6.3(Section 12.14 of ASCE 7-05) is used, the value of F_a shall be determined in accordance with Section 12.14.8.1 of ASCE 7-05, and the values for F_v, S_{MS}, and S_{M1} need not be determined.

TABLE 3.4.1 0.2s (S_s) and 1.0s (S_1) Spectral Maximum Considered Earthquake Ground Motion Accelerations at 2% Probability in 50 Years with 5% Critical Damping, Site Class B

Note: *Long-period transition period T_L is to be taken as 6 sec.*

State/ Division	Town	မြို့	Longi-tude	Lati-tude	S_s	S_1
Ayeyarwady	Bogale	ဘိုကလေး	16.3	95.4	0.34	0.3
Ayeyarwady	Danubyu	ဓနုဖြူ	17.25	95.6	0.6	0.3
Ayeyarwady	Dedaye	ဒေးဒရဲ	16.41	95.89	0.3	0.3
Ayeyarwady	Einme	အိမ်မဲ	16.9	95.18	0.6	0.3
Ayeyarwady	Hinthada	ဟင်္သာတ	17.65	95.46	0.6	0.3
Ayeyarwady	Kyaiklat	ကျိုက်လတ်	16.44	95.73	0.38	0.3
Ayeyarwady	Kyangin	ကြံခင်း	18.34	95.24	0.6	0.3
Ayeyarwady	Labutta	လပွတ္တာ	16.15	94.76	0.6	0.43
Ayeyarwady	Maubin	မအူပင်	16.73	95.65	0.6	0.3
Ayeyarwady	Myanaung	မြန်အောင်	18.29	95.32	0.6	0.3
Ayeyarwady	Myaungmya	မြောင်းမြ	16.6	94.93	0.6	0.3
Ayeyarwady	Ngapudaw	ငပုတော	16.54	94.69	0.6	0.31
Ayeyarwady	Nyaungdon	ညောင်တုန်း	17.05	95.63	0.6	0.3
Ayeyarwady	Pantanaw	ပန်းတနော်	16.98	95.47	0.6	0.3
Ayeyarwady	Pathein	ပုသိမ်	16.78	94.73	0.6	0.3
Ayeyarwady	Pyapon	ဖျာပုံ	16.29	95.68	0.36	0.3
Ayeyarwady	Thabaung	သာပေါင်း	17.05	94.81	0.6	0.3
Ayeyarwady	Wakema	ဝါးခယ်မ	16.6	95.18	0.6	0.3
Bago (East)	Bago	ပဲခူး	17.34	96.48	2.4	1.18
Bago (East)	Nyaunglebin	ညောင်လေးပင်	17.95	96.72	0.87	0.3
Bago (East)	Phyu	ဖြူး	18.48	96.44	2.13	1.02
Bago (East)	Shwegyin	ရွှေကျင်	17.92	96.88	0.61	0.3
Bago (East)	Taungoo	တောင်ငူ	18.94	96.43	1.65	0.6
Bago (East)	Thanatpin	သနပ်ပင်	17.29	96.58	2.03	0.73
Bago (East)	Waw	ဝေါ	17.48	96.68	1.14	0.4
Bago (West)	Gyobingauk	ကြို့ပင်ကောက်	18.23	95.65	0.61	0.3
Bago (West)	Letpadan	လက်ပံတန်း	17.78	95.75	0.63	0.3
Bago (West)	Minhla	မင်းလှ	17.97	95.71	0.61	0.3
Bago (West)	Paukkhaung	ပေါက်ခေါင်း	18.9	95.55	0.8	0.3
Bago (West)	Paungde	ပေါင်းတည်	18.49	95.51	0.6	0.3
Bago (West)	Pyay	ပြည်	18.82	95.22	0.74	0.3
Bago (West)	Shwedaung	ရွှေတောင်	18.71	95.21	0.64	0.3
Bago (West)	Thayarwady	သာယာဝတီ	17.65	95.79	0.65	0.3
Chin	Falam	လမ်း	22.91	93.68	1.02	0.74
Chin	Hakha	ဟားခါး	22.64	93.6	1.01	0.75
Chin	Kanpetlet	ကန်ပက်လက်	21.19	94.06	1.2	0.4
Chin	Matupi	မတူပီ	21.6	93.44	1.16	0.6
Chin	Mindat	မင်းတပ်	21.37	93.97	1.2	0.58
Chin	Paletwa	ပလက်ဝ	21.3	92.85	1.18	1
Chin	Tedim	တီးတိန်	23.37	93.66	1	1.05
Chin	Thantlang	ထန်တလန်	22.7	93.43	0.96	0.72
Chin	Tonzang	တွန်းဇန်	23.6	93.69	1.02	1.2
Kachin	Bhamo	ဗန်းမော်	24.25	97.23	0.68	0.5
Kachin	Chipwi	ချီဖွေ	25.89	98.13	0.69	0.3
Kachin	Hpakant	ဖားကန့်	25.61	96.31	1.5	1.7
Kachin	Machanbaw	မချမ်းဘော	27.28	97.59	0.76	0.97
Kachin	Mogaung	မိုးကောင်း	25.3	96.94	1.8	1.56
Kachin	Mohnyin	မိုးညှင်း	24.78	96.36	1.6	1.52
Kachin	Myitkyina	မြစ်ကြီးနား	25.39	97.39	1.01	0.9
Kachin	Nawngmun	နောင်မွန်း	27.51	97.82	0.6	0.6
Kachin	Puta-O	ပူတာအို	27.3	97.42	0.95	1.18
Kachin	Sumprabum	ဆွမ်ပရာဘွမ်	26.54	97.57	2.1	1.12
Kachin	Tanai	တနိုင်း	26.36	96.72	1.8	1.6
Kayah	Bawlake	ဘောလခဲ	19.17	97.34	0.9	0.3
Kayah	Hpasawng	ဖားဆောင်း	18.87	97.32	0.9	0.3
Kayah	Loikaw	လွိုင်ကော်	19.67	97.21	0.9	0.46
Kayah	Mese	မယ်စဲ	18.67	97.66	1.2	0.3
Kayin	Hpa-An	ဘားအံ	16.88	97.64	0.41	0.3
Kayin	Hpapun	ဖာပွန်	18.06	97.44	0.9	0.36
Kayin	Kawkareik	ကော့ကရိတ်	16.56	98.24	0.31	0.3
Kayin	Myawaddy	မြဝတီ	16.69	98.51	0.32	0.3
Kayin	Thandaunggyi	သံတောင်ကြီး	19.07	96.68	0.6	0.3
Magway	Aunglan	အောင်လံ	19.36	95.22	0.97	0.36
Magway	Chauk	ချောက်	20.89	94.82	1.13	0.34
Magway	Gangaw	ဂန့်ဂေါ	22.17	94.14	1.13	0.76
Magway	Magway	မကွေး	20.14	94.93	1.15	0.4
Magway	Minbu	မင်းဘူး	20.17	94.88	1.17	0.42
Magway	Mindon	မင်းတုန်း	19.35	94.73	1.06	0.47
Magway	Myaing	မြိုင်	21.61	94.85	1.1	0.55
Magway	Natmauk	နတ်မောက်	20.35	95.4	0.87	0.31
Magway	Pakokku	ပခုက္ကူ	21.34	95.08	0.99	0.46
Magway	Salin	စလင်း	20.58	94.66	1.2	0.6
Magway	Saw	ဆော	21.15	94.15	1.2	0.54
Magway	Sinbaungwe	ဆင်ပေါင်ဝဲ	19.72	95.16	1.02	0.52
Magway	Taungdwingyi	တောင်တွင်းကြီး	20	95.55	0.9	0.3
Magway	Thayet	သရက်	19.32	95.18	0.97	0.38
Magway	Tilin	ထီးလင်း	21.7	94.09	1.15	0.63
Magway	Yenangyaung	ရေနံချောင်း	20.46	94.87	1.14	0.6
Mandalay	Kyaukpadaung	ကျောက်ပန်းတောင်း	20.84	95.13	0.99	0.41
Mandalay	Kyaukse	ကျောက်ဆည်	21.61	96.13	1.34	0.6
Mandalay	Mandalay	မန္တလေး	21.99	96.09	1.8	1.31
Mandalay	Meiktila	မိတ္ထီလာ	20.88	95.86	0.95	0.62
Mandalay	Mogoke	မိုးကုတ်	22.92	96.51	0.77	0.3
Mandalay	Myingyan	မြင်းခြံ	21.46	95.39	0.63	0.3
Mandalay	Natogyi	နွားထိုးကြီး	21.42	95.65	0.6	0.31
Mandalay	Nyaung-U	ညောင်ဦး	21.2	94.91	1.07	0.52
Mandalay	Pyawbwe	ပျော်ဘွယ်	20.59	96.05	1.5	1.12
Mandalay	Pyinoolwin	ပြင်ဦးလွင်	22.01	96.46	0.74	0.47
Mandalay	Thabeikkyin	သပိတ်ကျင်း	22.89	95.98	1.62	1.2
Mandalay	Thazi	သာစည်	20.85	96.06	1.8	1.1
Mandalay	Yamethin	ရမည်းသင်း	20.43	96.14	1.5	0.83
Mon	Bilin	ဘီးလင်း	17.22	97.24	0.4	0.3
Mon	Kyaikto	ကျိုက်ထို	17.31	97.02	0.47	0.3
Mon	Mawlamyine	မော်လမြိုင်	16.48	97.63	0.3	0.3
Mon	Thanbyuzayat	သံဖြူဇရပ်	15.97	97.73	1.13	0.3
Mon	Thaton	သထုံ	16.92	97.37	0.36	0.3
Mon	Ye	ရေး	15.25	97.85	0.46	0.3
Nay Pyi Taw	Lewe	လယ်ဝေး	19.63	96.11	1.35	0.76

TABLE 3.4.1 0.2s (S_s) and 1.0s (S_1) Spectral Maximum Considered Earthquake Ground Motion Accelerations at 2% Probability in 50 Years with 5% Critical Damping, Site Class B

Note: *Long-period transition period T_L is to be taken as 6 sec.*

State/ Division	Town	မြို့	Longi-tude	Lati-tude	S_s	S_1	State/ Division	Town	မြို့	Longi-tude	Lati-tude	S_s	S_1
Nay Pyi Taw	Nay Pyi Taw	နေပြည်တော်	19.8	96.19	1.53	0.93	Shan (North)	Kyaukme	ကျောက်မဲ	22.54	97.04	0.6	0.3
Nay Pyi Taw	Tatkon	တပ်ကုန်း	20.13	96.2	1.62	0.62	Shan (North)	Lashio	လားရှိုး	22.95	97.75	0.6	0.3
Rakhine	Ann	အမ်း	19.8	94.05	1.03	0.6	Shan (North)	Laukkaing	လောက်ကိုင်	23.69	98.76	0.6	0.3
Rakhine	Buthidaung	ဘူးသီးတောင်	20.87	92.53	0.6	0.6	Shan (North)	Matman	မက်မန်း	21.95	98.87	0.66	0.3
Rakhine	Gwa	ဂွ	17.59	94.58	0.6	0.3	Shan (North)	Mongmao	မိုင်းမော	22.97	98.97	0.6	0.3
Rakhine	Kyaukpyu	ကျောက်ဖြူ	19.42	93.55	0.9	0.6	Shan (North)	Mongmit	မိုးမိတ်	23.11	96.67	0.66	0.3
Rakhine	Kyauktaw	ကျောက်တော်	20.84	92.97	1.24	2	Shan (North)	Mongyai	မိုင်းရယ်	22.43	98.04	0.6	0.3
Rakhine	Maungdaw	မောင်တော	20.82	92.37	0.6	0.6	Shan (North)	Muse	မူဆယ်	23.99	97.9	0.6	0.3
Rakhine	Minbya	မင်းပြား	20.36	93.27	1.39	0.6	Shan (North)	Namhkan	နမ့်ခမ်း	23.84	97.68	0.6	0.3
Rakhine	Mrauk-U	မြောက်ဦး	20.59	93.19	1.5	2	Shan (North)	Namhsan	နမ့်ဆန်	22.96	97.16	0.6	0.3
Rakhine	Munaung	မာန်အောင်	18.86	93.72	0.9	0.3	Shan (North)	Namtu	နမ္မတူ	23.09	97.4	0.6	0.3
Rakhine	Myebon	မြေပုံ	20.05	93.37	1.2	0.6	Shan (North)	Nawnghkio	နောင်ချို	22.33	96.8	0.6	0.3
Rakhine	Ramree	ရမ်းဗြဲ	19.09	93.86	0.9	0.6	Shan (South)	Hsihseng	ဆီဆိုင်	20.16	97.25	0.84	0.3
Rakhine	Rathedaung	ရသေ့တောင်	20.48	92.76	0.18	0.6	Shan (South)	Kalaw	ကလော	20.62	96.56	0.6	0.3
Rakhine	Sittwe	စစ်တွေ	20.14	92.89	0.6	0.6	Shan (South)	Kunhing	ကွန်ဟိန်း	21.3	98.42	0.61	0.3
Rakhine	Thandwe	သံတွဲ	18.47	94.37	0.6	0.3	Shan (South)	Kyethi	ကျေးသီး	21.93	97.82	0.64	0.3
Rakhine	Toungup	တောင်ကုတ်	18.85	94.24	0.9	0.6	Shan (South)	Laihka	လဲချား	21.27	97.67	0.88	0.3
Sagaing	Banmauk	ဗန်းမောက်	24.4	95.86	1.61	1.53	Shan (South)	Langkho	လင်းခေး	20.34	98	0.6	0.3
Sagaing	Budalin	ဘုတလင်	22.39	95.15	0.92	0.37	Shan (South)	Loilen	လွိုင်လင်	20.93	97.57	0.89	0.54
Sagaing	Hkamti	ခန္တီး	25.99	95.7	1.42	1.8	Shan (South)	Mawkmai	မောက်မယ်	20.23	97.72	0.6	0.3
Sagaing	Homalin	ဟုမ္မလင်း	24.86	94.91	1.5	1.8	Shan (South)	Monghsu	မိုင်းရှူး	21.91	98.36	0.67	0.3
Sagaing	Kale	ကလေး	23.19	94.03	1.18	1.07	Shan (South)	Mongkaing	မိုင်းကိုင်	21.61	97.53	0.9	0.54
Sagaing	Kalewa	ကလေးဝ	23.2	94.3	1.2	1.06	Shan (South)	Mongnai	မိုးနဲ	20.51	97.87	0.6	0.3
Sagaing	Kanbalu	ကန့်ဘလူ	23.2	95.52	0.71	0.59	Shan (South)	Mongpan	မိုင်းပန်	20.32	98.35	0.6	0.3
Sagaing	Kani	ကနီ	22.43	94.85	1.07	0.62	Shan (South)	Nansang	နမ့်စန်	20.89	97.72	0.76	0.3
Sagaing	Katha	ကသာ	24.18	96.33	1.33	1.12	Shan (South)	Nyaungshwe	ညောင်ရွှေ	20.66	96.93	0.9	0.3
Sagaing	Kawlin	ကောလင်း	23.79	95.68	1.09	1.26	Shan (South)	Pekon	ဖယ်ခုံ	19.86	97.01	0.9	0.52
Sagaing	Lahe	လဟယ်	26.33	95.44	1.28	1.8	Shan (South)	Pindaya	ပင်းတယ	20.94	96.66	0.6	0.3
Sagaing	Lay Shi	လေရှီး	25.45	94.96	1.32	1.8	Shan (South)	Pinlaung	ပင်လောင်း	20.12	96.78	0.63	0.3
Sagaing	Mawlaik	မော်လိုက်	23.64	94.41	1.2	1.43	Shan (South)	Taunggyi	တောင်ကြီး	20.77	97.04	0.9	0.41
Sagaing	Mingin	မင်းကင်း	22.88	94.49	1.2	0.68	Shan (South)	Ywangan	ရွာငံ	21.16	96.44	0.6	0.3
Sagaing	Monywa	မုံရွာ	22.12	95.14	0.95	0.69	Tanintharyi	Bokpyin	ဘုတ်ပြင်း	11.27	98.76	0.42	0.3
Sagaing	Myinmu	မြင်းမူ	21.93	95.57	0.6	0.3	Tanintharyi	Dawei	ထားဝယ်	14.08	98.2	1.36	0.9
Sagaing	Nanyun	နန်းယွန်း	26.98	96.17	1.24	1.77	Tanintharyi	Kawthoung	ကော့သောင်း	9.98	98.55	1.2	0.3
Sagaing	Sagaing	စစ်ကိုင်း	21.88	95.96	1.64	1.2	Tanintharyi	Kyunsu	ကျွန်းစု	12.47	98.45	0.3	0.3
Sagaing	Shwebo	ရွှေဘို	22.57	95.7	0.6	0.34	Tanintharyi	Myeik	မြိတ်	12.44	98.61	0.3	0.3
Sagaing	Tamu	တမူး	24.21	94.32	1.21	1.5	Tanintharyi	Palaw	ပုလော	12.98	98.65	0.3	0.3
Sagaing	Tigyaing	ထီးချိုင့်	23.75	96.15	1.64	1.13	Tanintharyi	Tanintharyi	တနင်္သာရီ	12.09	99.01	0.3	0.3
Sagaing	Yinmarbin	ယင်းမာပင်	22.08	94.9	1.06	0.67	Tanintharyi	Yebyu	ရေဖြူ	14.25	98.2	1.41	0.9
Shan (East)	Kengtung	ကျိုင်းတုံ	21.29	99.6	0.6	0.3	Yangon	Cocokyun	ကိုကိုးကျွန်း	14.13	93.37	0.8	0.3
Shan (East)	Monghpyak	မိုင်းဖြတ်	20.88	99.92	0.6	0.3	Yangon	Hlegu	လှည်းကူး	17.1	96.23	0.93	0.34
Shan (East)	Monghsat	မိုင်းဆတ်	20.51	99.25	0.6	0.3	Yangon	Htaukkyant	ထောက်ကြန့်	17.04	96.13	0.87	0.3
Shan (East)	Mongla	မိုင်းလား	21.67	100.02	0.6	0.3	Yangon	Kayan	ခရမ်း	16.91	96.56	1.67	0.76
Shan (East)	Mongping	မိုင်းပျဉ်း	21.35	99.03	0.6	0.3	Yangon	Kungyangon	ကွမ်းခြံကုန်း	16.44	96.01	0.63	0.3
Shan (East)	Mongton	မိုင်းတုံ	20.3	98.9	0.6	0.3	Yangon	Kyauktan	ကျောက်တန်း	16.63	96.32	1.31	0.5
Shan (East)	Mongyang	မိုင်းယန်း	21.84	99.69	0.6	0.3	Yangon	Taikkyi	တိုက်ကြီး	17.31	95.96	0.74	0.3
Shan (East)	Tachileik	တာချီလိတ်	20.45	99.88	0.6	0.3	Yangon	Thongwa	သုံးခွ	16.76	96.52	2.4	1.11
Shan (North)	Hopang	ဟိုပန်	23.43	98.75	0.6	0.3	Yangon	Twantay	တွံတေး	16.71	95.93	0.6	0.3
Shan (North)	Hseni	သိန္နီ	23.31	97.97	0.6	0.3	Yangon	Yangon	ရန်ကုန်	16.78	96.16	0.6	0.3
Shan (North)	Hsipaw	သီပေါ	22.62	97.3	0.6	0.3							

Maximum Considered Earthquake Ground Motion for 0.2 Sec Spectral Response Acceleration at 2% in 50 Years wth 5% Critical Damping for Site Class B (Myo Thant et el., 2016)

FIGURE 3.4.1.1: Maximum Considered Earthquake Ground Motion for 0.2 Sec Spectral Response Acceleration at 2% Probability in 50 Years with 5% Critical Damping, Site Class B

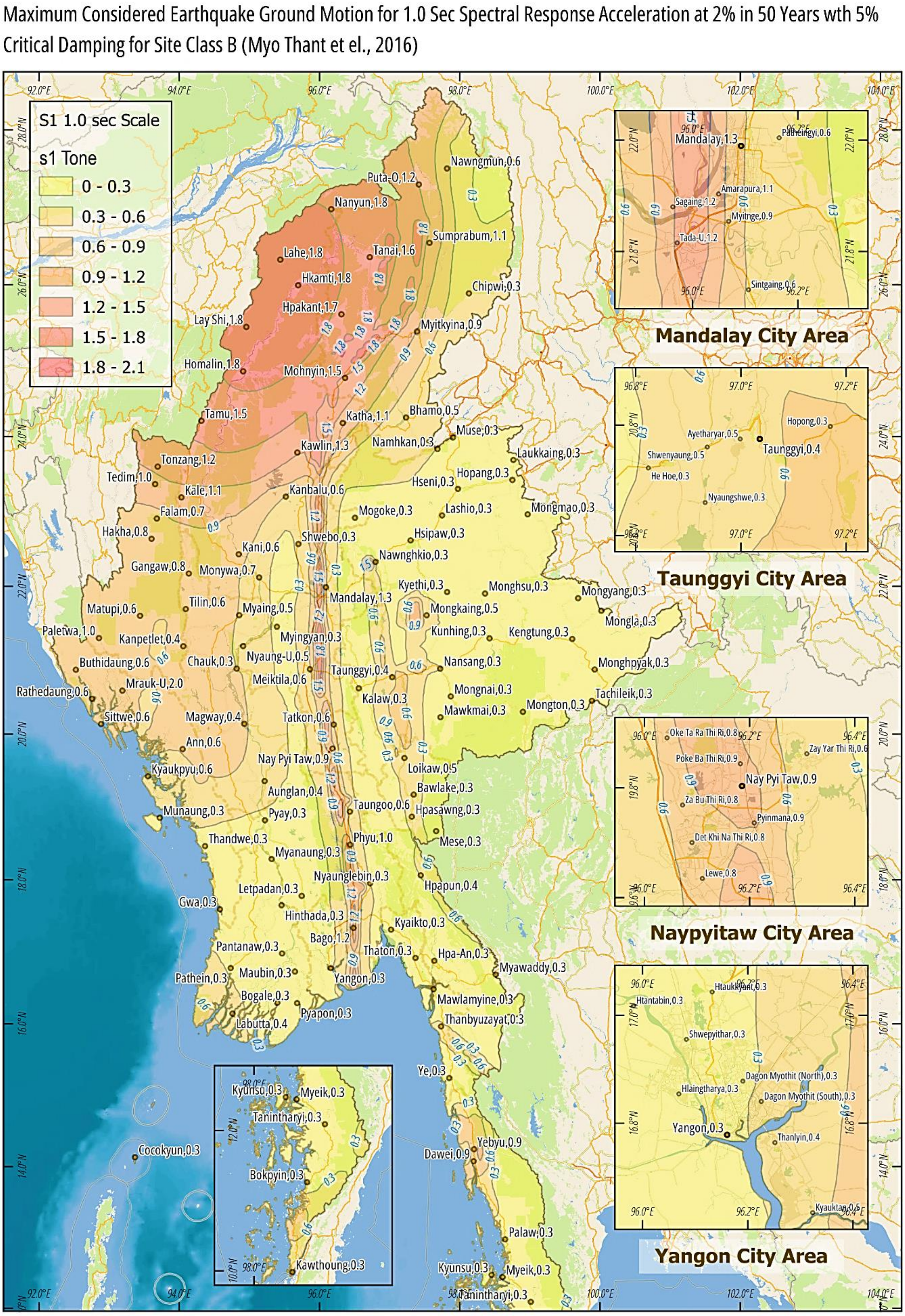


FIGURE 3.4.1.2: Maximum Considered Earthquake Ground Motion for 1 Sec Spectral Response Acceleration at 2% Probability in 50 Years with 5% Critical Damping, Site Class B

TABLE 3.4.2 SITE CLASS DEFINITIONS

<table>
<tr><th rowspan="2">SITE CLASS</th><th rowspan="2">SOIL PROFILE NAME</th><th colspan="3">AVERAGE PROPERTIES IN TOP 100 FEET, SEE SECTION 3.4.1.3.4.2</th></tr>
<tr><th>Soil shear wave velocity $\bar{v}_s$ (ft/sec)</th><th>Standard penetration resistance $\bar{N}$</th><th>Soil undrained shear strength, $\bar{s}_u$(psf)</th></tr>
<tr><td>A</td><td>Hard Rock</td><td>$\bar{v}_s > 5{,}000$</td><td>N/A</td><td>N/A</td></tr>
<tr><td>B</td><td>Rock</td><td>$2{,}500 < \bar{v}_s \leq 5{,}000$</td><td>N/A</td><td>N/A</td></tr>
<tr><td>C</td><td>Very dense soil and soft rock</td><td>$1{,}200 < \bar{v}_s \leq 2{,}500$</td><td>$\bar{N} > 50$</td><td>$\bar{s}_u \geq 2{,}000$</td></tr>
<tr><td>D</td><td>Stiff soil profile</td><td>$600 \leq \bar{v}_s \leq 1{,}200$</td><td>$15 \leq \bar{N} \leq 50$</td><td>$1{,}000 \leq \bar{s}_u \leq 2{,}000$</td></tr>
<tr><td>E</td><td>Soft soil profile</td><td>$\bar{v}_s < 600$</td><td>$\bar{N} < 15$</td><td>$\bar{s}_u < 1{,}000$</td></tr>
<tr><td>E</td><td>-</td><td colspan="3">Any profile with more than 10 feet of soil having the following characteristics:
1. Plasticity index $PI > 20$,
2. Moisture content $w \geq 40\%$, and
3. Undrained shear strength $\bar{s}_u < 500$ psf</td></tr>
<tr><td>F</td><td>-</td><td colspan="3">Any profile containing soils having one or more of the following characteristics:
1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, 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)</td></tr>
<tr><td colspan="5">For SI: 1 foot = 304.8 mm, 1 square foot = 0.0929 m^2, 1 pound per square foot = 0.0479 kPa.
N/A = Not applicable.</td></tr>
</table>

TABLE 3.4.3 SITE COEFFICIENT, F_a

<table>
<tr><th rowspan="2">Site Class</th><th colspan="5">Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at Short Period</th></tr>
<tr><th>$S_S \leq 0.25$</th><th>$S_S = 0.5$</th><th>$S_S = 0.75$</th><th>$S_S = 1.0$</th><th>$S_S \geq 1.25$</th></tr>
<tr><td>A</td><td>0.8</td><td>0.8</td><td>0.8</td><td>0.8</td><td>0.8</td></tr>
<tr><td>B</td><td>1.0</td><td>1.0</td><td>1.0</td><td>1.0</td><td>1.0</td></tr>
<tr><td>C</td><td>1.2</td><td>1.2</td><td>1.1</td><td>1.0</td><td>1.0</td></tr>
<tr><td>D</td><td>1.6</td><td>1.4</td><td>1.2</td><td>1.1</td><td>1.0</td></tr>
<tr><td>E</td><td>2.5</td><td>1.7</td><td>1.2</td><td>0.9</td><td>0.9</td></tr>
<tr><td>F</td><td colspan="5">See Section 11.4.7 (ASCE 7-05)</td></tr>
<tr><td colspan="6">NOTE: Use straight-line interpolation for intermediate values of S_s.</td></tr>
</table>

TABLE 3.4.4 ITE COEFFICIENT, F_v

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
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	See Section 11.4.7 (ASCE 7-05)				
NOTE: Use straight-line interpolation for intermediate values of S_1.					

3.4.1.4.4 Design Spectral Acceleration Parameters

Design earthquake spectral response acceleration parameter at short period, S_{DS}, and at 1 s period, S_{D1} , shall be determined from Eqs. (3.4.3) and (3.4.4), respectively. Where the alternative simplified design procedure of Section 3.4.1.6.3(Section 12.14 of ASCE 7-05) is used, the value of S_{DS} shall be determined in accordance with Section (Section 12.14.8.1 of ASCE 7-05), and the value for S_{D1} need not be determined.

$$S_{DS} = 2/3\ S_{MS} \qquad \text{Eq. (3.4.3)}$$
$$S_{D1} = 2/3\ S_{M1} \qquad \text{Eq. (3.4.4)}$$

3.4.1.4.5 Design Response Spectrum

Where a design response spectrum is required by this standard and site-specific ground motion procedures are not used, the design response spectrum curve shall be developed as indicated in Fig. 3.4.3 and as follows:

1. For periods less than T_0, the design spectral response acceleration, S_a, shall be taken as given by Eq. (3.4.5):

$$Sa = S_{DS}\ (0.4 + 0.6\ T/\ T_0) \qquad \text{Eq. (3.4.5)}$$

2. For periods greater than or equal to T_0 and less than or equal to T_S, the design spectral response acceleration, Sa, shall be taken equal to S_{DS}.

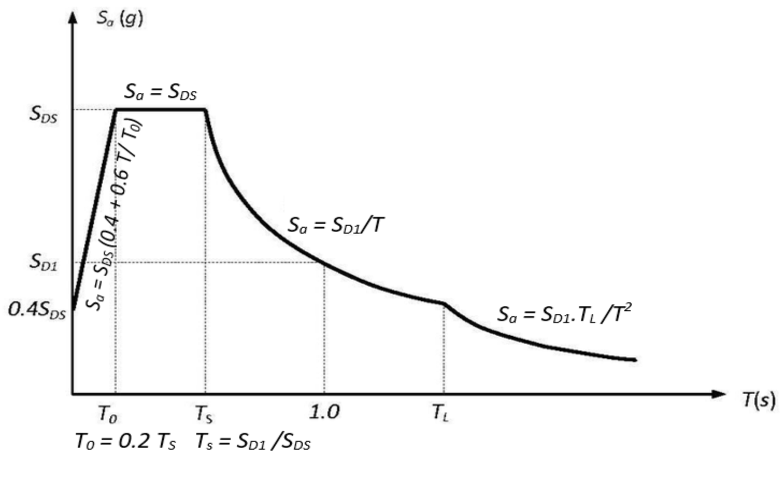


FIGURE 3.4.3 Design response spectrum

3. For periods greater than T_S, and less than or equal to T_L, the design spectral response acceleration, Sa, shall be taken as given by Eq. (3.4.6):

$$S_a = S_{D1}/T \qquad \textit{Eq. (3.4.6)}$$

4. For periods greater than T_L, Sa shall be taken as given by Eq. (3.4.7):

$$S_a = S_{D1}T_L/T^2 \qquad \textit{Eq. (3.4.7)}$$

where,

S_{DS} = the design spectral response acceleration parameter at short periods

S_{D1} = the design spectral response acceleration parameter at 1-s period

T = the fundamental period of the structure, s

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

T_S = S_{D1}/S_{DS} and

T_L = long-period transition period as specified in Table 3.4.1

3.4.1.4.6 MCE Response Spectrum

Where a MCE response spectrum is required, it shall be determined by multiplying the design response spectrum by 1.5.

3.4.1.4.7 Site-Specific Ground Motion Procedures

The site-specific ground motion procedures set forth in Chapter 21 of ASCE 7-05 are permitted to be used to determine ground motions for any structure.

3.4.1.4.8 Site Classification for Seismic Design

Site classification for Site Class C, D or E shall be determined from Table 3.4.5.

The notation presented below apply to the upper 100 feet (30,480 mm) of the site profile. Profiles containing distinctly different soil and/or rock layers shall be subdivided into those layers designated by a number that ranges from 1 to *n* at the bottom where there is a total of *n* distinct layers in the upper 100 feet (30,480 mm). The symbol *i* then refers to any one of the layers between 1 and *n.*

where,

v_{si}= the shear wave velocity in feet per second (m/s).

d_i = the thickness of any layer between 0 and 100 feet (30,480 mm).

where,

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}} \qquad \textit{Eq. (3.4.8)}$$

$\sum_{i=1}^{n} d_i$ = *100 ft (30,480 mm)*

N_i is the Standard Penetration Resistance (ASTM D1586) not to exceed 100 blows/foot (304.8 mm) as directly measured in the field without corrections. When refusal is met for a rock layer, N_i shall be taken as 100 blows/foot (304.8 mm).

$$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \qquad \textit{Eq. (3.4.9)}$$

where,

N_i and d_i in Equation (3.4.9) are for cohesionless soil, cohesive soil and rock layers.

$$\bar{N}_{ch} = \frac{d_s}{\sum_{i=1}^{m} \frac{d_i}{N_i}} \qquad \textit{Eq. (3.4.10)}$$

where,

$$\sum_{i=1}^{m} d_i = d_s$$

Use

d_i and N_i for cohesionless soil layers only in Equation (3.4.10)

d_s= total thickness of cohesionless soil layers in the top 100 feet (30,480 mm)

m = number of cohesionless soil layers in the top 100 feet (30,480 mm)

s_{ui} = the undrained shear strength in psf (kPa), not to exceed 5,000 psf (240 kPa), ASTM D2166 or D2850.

$$\bar{\boldsymbol{s}}_{\boldsymbol{u}} = \frac{d_c}{\sum_{i=1}^{k} \frac{d_i}{s_{ui}}} \qquad \textit{Eq. (3.4.11)}$$

where,

$$\sum_{i=1}^{k} d_i = d_c$$

d_c = the total thickness of cohesive soil layers in the top 100 feet (30,480 mm)

k = the number of cohesive soil layers in the top 100 feet (30,480 mm)

PI = the plasticity index, ASTM D 4318

w = the moisture content in percent, ASTM D 2216

Where a site does not qualify under the criteria for Site Class F and there is a total thickness of soft clay greater than 10 feet (3048 mm) where a soft clay layer is defined by: s_u< 500 psf (24 kPa), $w \geq 40$ percent, and PI > 20, it shall be classified as Site Class E.

The shear wave velocity for rock, Site Class B, shall be either measured on site or estimated by a geotechnical engineer or engineering geologist/seismologist for competent rock with moderate fracturing and weathering. Softer and more highly fractured and weathered rock shall either be measured on site for shear wave velocity or classified as Site Class C.

The hard rock category, Site Class A, shall be supported by shear wave velocity measurements either on site or on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing. Where hard rock conditions are known to be continuous to a depth of 100 feet (30480 mm), surficial shear wave velocity measurements are permitted to be extrapolated to assess $\bar{v}_s$.

The rock categories, Site Classes A and B, shall not be used if there is more than 10 feet (3,048 mm) of soil between the rock surface and the bottom of the spread footing or mat foundation.

TABLE 3.4.5 SITE CLASSIFICATION[a]

SITE CLASS	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
C	1,200 to 2,500 ft/s	>50	> 2,000
D	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E	< 600 ft/s	< 15	< 1,000 psf

For SI: 1 foot per second = 304.8 mm per second, 1 pound per square foot = 0.0479 kN/m^2.

If the $\bar{s}_u$method is used and the $\bar{N}_{ch}$and $\bar{s}_u$ criteria differ, select the category with the softer soils (for example, use Site Class E instead of D).

3.4.1.4.8.1 Steps for Classifying a Site

1. Check for the four categories of Site Class F requiring site-specific evaluation. If the site corresponds to any of these categories, classify the site as Site Class F and conduct a site-specific evaluation.

2. Check for the existence of a total thickness of soft clay > 10 feet (3,048 mm) where a soft clay layer is defined by: $\bar{s}_u$ < 500 psf (24 kPa), w ≥ 40 percent and PI > 20. If these criteria are satisfied, classify the site as Site Class E.

3. Categorize the site using one of the following three methods with $\bar{v}_s$, $\bar{N}$, and $\bar{s}_u$and computed in all cases as specified:

   (i) $\bar{v}_s$ for the top 100 feet (30,480 mm) ($\bar{v}_s$ method)
   (ii) $\bar{N}$ for the top 100 feet (30,480 mm) ($\bar{N}$ method)
   (iii) $\bar{N}_{ch}$ for cohesionless soil layers (PI < 20) in the top 100 feet (30,480 mm) and average, $\bar{s}_u$for cohesive soil layers (PI > 20) in the top 100 feet (30,480 mm) ($\bar{s}_u$ method)

3.4.1.5 Importance Factor and Occupancy Category

3.4.1.5.1 Importance Factor

An importance factor, *I*, shall be assigned to each structure in accordance with Table 3.4.6 based on the Occupancy Category from Table 3.1.2.

TABLE 3.4.6 IMPORTANCE FACTORS (SEISMIC LOAD)

Occupancy Category	I
I , II	1.0
III	1.25
IV	1.5

3.4.1.5.2 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.

3.4.1.6 Seismic Design Category

Structures shall be assigned a Seismic Design Category in accordance with Section 3.4.1.6.1.

3.4.1.6.1 Determination of Seismic Design Category

Occupancy Category I, II, and III structures located where the 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 D and E respectively. Occupancy Category IV structures located where the 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. 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 3.4.1.4.4 or the site-specific procedures of ASCE 7-05. Each building shall be assigned to the more severe Seismic Design Category in accordance with Table 3.4.7, irrespective of the fundamental period of vibration of the structure, T.

TABLE 3.4.7 SEISMIC DESIGN CATEGORY BASED ON SHORT PERIOD AND 1-SECOND PERIOD RESPONSE ACCELERATION PARAMETER

S_{DS}	S_{D1}	Level of Seismicity	I	II	III	IV
$S_{DS} < 0.167g$	$S_{D1} < 0.067g$	Very Low	A	A	A	B
$0.167g \le S_{DS} < 0.33g$	$0.067g \le S_{D1} < 0.133g$	Low	A	A	B	C
$0.33g \le S_{DS} < 0.5g$	$0.133g \le S_{D1} < 0.2$ g	Moderate	A	B	C	D
$0.5g \le S_{DS} < 0.9g$	$0.2g \le S_{D1} < 0.5$ g	High	B	C	D	D
$S_{DS} \ge 0.9g$	$S_{D1} \ge 0.5g$	Severe	C	D	D	D
$S_1 \ge 0.75$ g		Extreme	D	D	E	F

3.4.1.6.2 Alternative Seismic Design Category Determination

Where S_1 is less than 0.75, the seismic design category is permitted to be determined from S_{DS} values from Table 3.4.7 alone when all the following apply:

1. In each of the two orthogonal directions, the approximate fundamental period of the structure, Ta, in each of the two orthogonal directions determined in accordance with Section 12.8.2.1 of ASCE 7-05, is less than 0.8 Ts determined in accordance with Section 11.4.5 of ASCE 7-05.
2. In each of the two orthogonal directions, the fundamental period of the structure used to calculate the story drift is less than T_s.
3. Equation 12.8-2 of ASCE 7-05 is used to determine the seismic response coefficient, C_s.
4. The diaphragms are rigid as defined in Section 12.3.1 in ASCE 7-05 or for diaphragms that are flexible, the distance between vertical elements of the seismic-force-resisting system does not exceed 40 feet (12 192 mm).

3.4.1.6.3 Simplified Design Procedure

Where the alternate simplified design procedure of ASCE 7-05 is used, the seismic design category shall be determined in accordance with ASCE 7-05.

3.4.2 Structural System Selection

3.4.2.1 Selection and Limitations

The basic lateral and vertical seismic force–resisting system shall conform to one of the types indicated in Table 3.4.8 or a combination of systems as permitted in Sections 3.4.2.2 and 3.4.2.3. Each type is subdivided by the types of vertical elements used to resist lateral seismic forces. The structural system used shall be in accordance with the Seismic Design Category and height limitations indicated in Table 3.4.8. The appropriate response modification coefficient, R, system over strength factor, Ω_0, and the deflection amplification factor, C_d, indicated in Table 3.4.8 shall be used in determining the base shear, element design forces, and design storey drift.

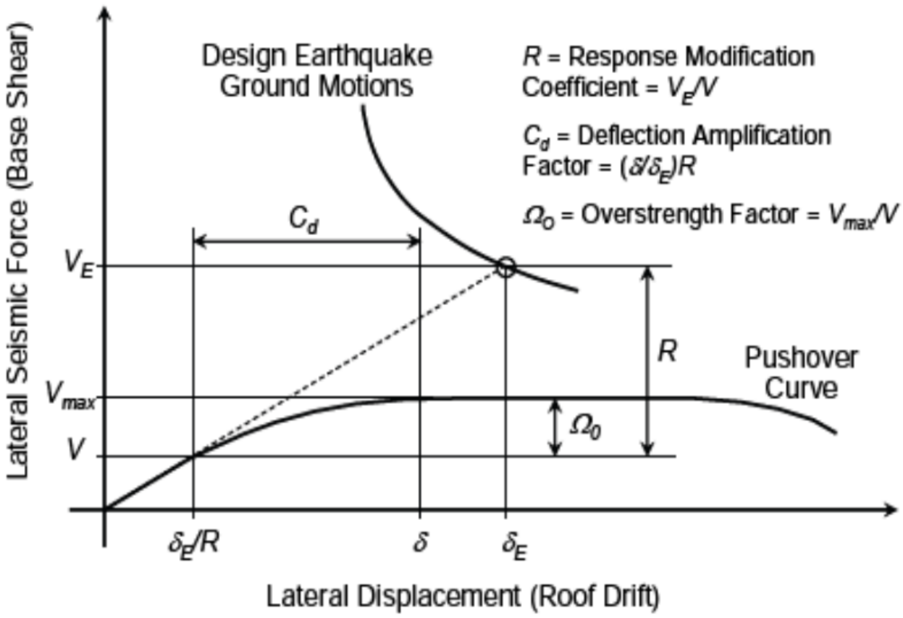


FIGURE 3.4.4 Seismic Response Parameters

TABLE 3.4.8 DESIGN COEFFICIENTS AND FACTORS FOR SEISMIC FORCE-RESISTING SYSTEMS

Seismic Force-Resisting System	ASCE 7-05, Required Detailing	Response Modification Factor, R^a	System Over strength Factor, $\Omega_0{}^g$	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	14.2	5	2½	5	NL	NL	160	160	100
2.Ordinary reinforced concrete shear walls	14.2	4	2½	4	NL	NL	NP	NP	NP
3. Detailed plain concrete shear walls	14.2	2	2½	2	NL	NP	NP	NP	NP
4. Ordinary plain concrete shear walls	14.2	1½	2½	1½	NL	NP	NP	NP	NP
5.Intermediate precast shear walls	14.2	4	2½	4	NL	NL	40[k]	40[k]	40[k]
6. Ordinary precast shear walls	14.2	3	2½	3	NL	NP	NP	NP	NP
7. Special reinforced masonry shear walls	14.4	5	2½	3½	NL	NL	160	160	100
8. Intermediate reinforced masonry shear walls	14.4	3½	2½	2¼	NL	NL	NP	NP	NP
9. Ordinary reinforced masonry shear walls	14.4	2	2½	1¾	NL	160	NP	NP	NP
10. Detailed plain masonry shear walls	14.4	2	2½	1¾	NL	NP	NP	NP	NP
11. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP
12. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
13. Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1, 14.5	6½	3	4	NL	NL	65	65	65
14. Light-framed walls with shear panels of all other materials	14.1, 14.5	2	2½	2	NL	NL	35	NP	NP
15. Light-framed wall systems using flat strap bracing	14.1, 14.5	4	2	3½	NL	NL	65	65	65
B. BUILDING FRAME SYSTEMS									
1. Steel eccentrically braced frames, moment resisting connections at columns away from links	14.1	8	2	4	NL	NL	160	160	100
2. Steel eccentrically braced frames, non-moment-resisting, connections at columns away from links	14.1	7	2	4	NL	NL	160	160	100
3. Special steel concentrically braced frames	14.1	6	2	5	NL	NL	160	160	100
4. Ordinary steel concentrically braced frames	14.1	3¼	2	3¼	NL	NL	35[j]	35[j]	NP[j]
5. Special reinforced concrete shear walls	14.1, 14.5	6	2½	5	NL	NL	160	160	100
6. Ordinary reinforced concrete shear walls	14.1, 14.5	5	2½	4½	NL	NL	NP	NP	NP
7. Detailed plain concrete shear walls	14.1, 14.5	2	2½	2	NL	NP	NP	NP	NP
8. Ordinary plain concrete shear walls	14.1, 14.5	1½	2½	1½	NL	NP	NP	NP	NP
9. Intermediate precast shear walls	14.1, 14.5	5	2½	4½	NL	NL	40[k]	40[k]	40[k]
10. Ordinary precast shear walls	14.1, 14.5	4	2½	4	NL	NP	NP	NP	NP
11. Composite steel and concrete eccentrically braced frames	14.3	8	2	4	NL	NL	160	160	100
12. Composite steel and concrete concentrically braced frames	14.3	5	2	4½	NL	NL	160	160	100
13. Ordinary composite steel and concrete braced frames	14.3	3	2	3	NL	NL	NP	NP	NP
14. Composite steel plate shear walls	14.3	6½	2½	5½	NL	NL	160	160	100
15. Special composite reinforced concrete shear walls with steel elements	14.3	6	2½	5	NL	NL	160	160	100
16. Ordinary composite reinforced concrete shear walls with steel elements	14.3	5	2½	4½	NL	NL	NP	NP	NP
17. Special reinforced masonry shear walls	14.4	5½	2½	4	NL	NL	160	160	100
18.Intermidiate reinforced masonry shear walls	14.4	4	2½	4	NL	NL	NP	NP	NP
19. Ordinary reinforced masonry shear walls	14.4	2	2½	2	NL	160	NP	NP	NP
20. Detailed plain masonry shear walls	14.4	2	2½	2	NL	NP	NP	NP	NP
21. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP
22. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
23. Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1, 14.5	7	2½	4½	NL	NL	65	65	65
24. Light-framed walls with shear panels of all other materials	14.1, 14.5	2½	2½	2½	NL	NL	35	NP	NP
25. Buckling-restrained braced frames, non-moment-resisting beam-column connections	14.1	7	2	5½	NL	NL	160	160	100
26. Buckling-restrained braced frames, moment-resisting beam-column connections	14.1	8	2½	5	NL	NL	160	160	100
27. Special steel plate shear wall	14.1	7	2	6	NL	NL	160	160	100

Seismic Force-Resisting System	ASCE 7-05, Required Detailing	Response Modification Factor, R[a]	System Over strength 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	14.1, 12.2.5.5	8	3	5½	NL	NL	NL	NL	NL
2. Special steel truss moment frames	14.1	7	3	5½	NL	NL	160	100	NP
3. Intermediate steel moment frames	12.2.5.6, 12.2.5.7, 12.2.5.8, 12.2.5.9, and 14.1	4.5	3	4	NL	NL	35[h,i]	NP[h]	NP[i]
4. Ordinary steel moment frames	12.2.5.6, 12.2.5.7, 12.2.5.8, and 14.1	3.5	3	3	NL	NL	NP[h]	NP[h]	NP[i]
5. Special reinforced concrete moment frames	12.2.5.5, 14.2	8	3	5½	NL	NL	NL	NL	NL
6. Intermediate reinforced concrete moment frames	14.2	5	3	4½	NL	NL	NP	NP	NP
7. Ordinary reinforced concrete moment frames	14.2	3	3	2½	NL	NP	NP	NP	NP
8. Special composite steel and concrete moment frames	12.2.5.5, 14.3	8	3	5½	NL	NL	NL	NL	NL
9. Intermediate composite moment frames	14.3	5	3	4½	NL	NL	NP	NP	NP
10. Composite partially restrained moment frames	14.3	6	3	5½	160	160	100	NP	NP
11. Ordinary composite moment frames	14.3	3	3	2½	NL	NP	NP	NP	NP
D. DUAL SYSTEMS WITH SPECIAL MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES	12.2.5.1								
1. Steel eccentrically braced frames	14.1	8	2½	4	NL	NL	NL	NL	NL
2. Special steel concentrically braced frames	14.1	7	2½	5½	NL	NL	NL	NL	NL
3. Special reinforced concrete shear walls	14.2	7	2½	5½	NL	NL	NL	NL	NL
4. Ordinary reinforced concrete shear walls	14.2	6	2½	5	NL	NL	NP	NP	NP
5. Composite steel and concrete eccentrically braced frames	14.3	8	2½	4	NL	NL	NL	NL	NL
6. Composite steel and concrete concentrically braced frames	14.3	6	2½	5	NL	NL	NL	NL	NL
7. Composite steel plate shear walls	14.3	7½	2½	6	NL	NL	NL	NL	NL
8. Special composite reinforced concrete shear walls with steel elements	14.3	7	2½	6	NL	NL	NL	NL	NL
9. Ordinary composite reinforced concrete shear walls with steel elements	14.3	6	2½	5	NL	NL	NP	NP	NP
10. Special reinforced masonry shear walls	14.4	5½	3	5	NL	NL	NL	NL	NL
11. Intermediate reinforced masonry shear walls	14.4	4	3	3½	NL	NL	NP	NP	NP
12. Buckling-restrained braced frame	14.1	8	2½	5	NL	NL	NL	NL	NL
13. Special steel plate shear walls	14.1	8	2½	6½	NL	NL	NL	NL	NL
E. DUAL SYSTEMS WITH INTERMEDIATE MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES	12.2.5.1								
1. Special steel concentrically braced frames[f]	14.1	6	2½	5	NL	NL	35	NP	NP[h,k]
2. Special reinforced concrete shear walls	14.2	6½	2½	5	NL	NL	160	100	100
3. Ordinary reinforced masonry shear walls	14.4	3	3	2½	NL	160	NP	NP	NP
4. Intermediate reinforced masonry shear walls	14.4	3½	3	3	NL	NL	NP	NP	NP
5. Composite steel and concrete concentrically braced frames	14.3	5½	2½	4½	NL	NL	160	100	NP
6. Ordinary composite braced frames	14.3	3½	2½	3	NL	NL	NP	NP	NP
7. Ordinary composite reinforced concrete shear walls with steel elements	14.3	5	3	4½	NL	NL	NP	NP	NP
8. Ordinary reinforced concrete shear walls	14.2	5½	2½	4½	NL	NL	NP	NP	NP
F. SHEAR WALL-FRAME INTERACTIVE SYSTEM WITH ORDINARY REINFORCED CONCRETE MOMENT FRAMES AND ORDINARY REINFORCED CONCRETE SHEAR WALLS	12.2.5.10, 14.2	4½	2½	4	NL	NP	NP	NP	NP

Seismic Force-Resisting System	ASCE 7-05, Required Detailing	Response Modification Factor, R^a	System Over strength 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]
G. CANTILEVERED COLUMN SYSTEMS DETAILED TO CONFORM TO THE REQUIREMENTS FOR:	12.2.5.2								
1. Special steel moment frames	12.2.5.5, 14.1	2½	1¼	2½	35	35	35	35	35
2. Intermediate steel moment frames	14.1	1½	1¼	1½	35	35	35h	NP[h,i]	NP[h,i]
3. Ordinary steel moment frames	14.1	1¼	1¼	1¼	35	35	NP	NP[h,i]	NP[h,i]
4. Special reinforced concrete moment frames	12.2.5.5, 14.2	2½	1¼	2½	35	35	35	35	35
5. Intermediate concrete moment frames	14.2	1½	1¼	1½	35	35	NP	NP	NP
6. Ordinary concrete moment frames	14.2	1	1¼	1	35	NP	NP	NP	NP
7. Timber frames	14.5	1½	1½	1½	35	35	35	NP	NP
H. STEEL SYSTEMS NOT SPECIFICALLY DETAILED FOR SEISMIC RESISTANCE, EXCLUDING CANTILEVER COLUMN SYSTEMS	14.1	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. Deflection amplification factor, Cd, for use in Sections 3.4.8.6, 3.4.8.7, and 3.4.9.2
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 3.4.2.5.4 for a description of building systems limited to buildings with a height of 240 ft (73.2 m) or less.
e. See Section 3.4.2.5.4 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 over strength factor, Ω_0, 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 except cantilever column systems.
h. See Sections 3.4.2.5.6 and 3.4.2.5.7 for limitations for steel OMFs and IMFs in structures assigned to Seismic Design Category D or E.
i. See Sections 3.4.2.5.8 and 3.4.2.5.9 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/m^2) and in penthouse structures.
k. Increase in height to 45 ft (13.7 m) is permitted for single story storage warehouse facilities.

Note: Section numbers in this foot-notes are referring to Table 3.4.8.

Seismic force–resisting systems that are not contained in Table 3.4.8 are permitted if analytical and test data are submitted that establish the dynamic characteristics and demonstrate the lateral force resistance and energy dissipation capacity to be equivalent to the structural systems listed in Table 3.4.8 for equivalent response modification coefficient, R , system over strength coefficient, Ω_0 , and deflection amplification factor, C_d, values.

The selected seismic force-resisting system shall be designed and detailed in accordance with the specific requirements for the system per the applicable reference document and the additional requirements set forth in later sections on material design standards.

3.4.2.2 Combinations of Framing Systems in Different Directions

Different seismic force–resisting systems are permitted to be used to resist seismic forces along each of the two orthogonal axes of the structure. Where different systems are used, the respective R, C_d, and Ω_0 coefficients shall apply to each system, including the limitations on system use contained in Table 3.4.8.

3.4.2.3 Combinations of Framing Systems in the Same Direction

Where different seismic force–resisting systems are used in combination to resist seismic forces in the same direction of structural response, other than those combinations considered as dual systems, the more stringent system limitation contained in Table 3.4.8 shall apply and the design shall comply with the requirements of this section.

3.4.2.3.1 R, C_d, and Ω_0 Values for Vertical Combinations

The value of the response modification coefficient, R, used for design at any storey shall not exceed the lowest value of R that is used in the same direction at any storey above that storey. Likewise, the deflection amplification factor, C_d, and the system over strength factor, Ω_0, used for the design at any storey shall not be less than the largest value of this factor that is used in the same direction at any storey above that storey.

EXCEPTIONS:

1. Rooftop structures not exceeding two storeys 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.

3.4.2.3.2 R, C_d, and Ω_0 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 storeys 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 the same direction.

The deflection amplification factor, C_d, and the system over strength factor, Ω_0, in the direction under consideration at any storey shall not be less than the largest value of this factor for the R factor used in the same direction being considered.

3.4.2.4 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 Section 3.4.2 required by the highest response modification coefficient, R, of the connected framing systems.

3.4.2.5 System Specific Requirements

The structural framing system shall also comply with the following system specific requirements of this section.

3.4.2.5.1 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.

3.4.2.5.2 Cantilever Column Systems

Cantilever column systems are permitted as indicated in Table 3.4.8 and as follows. The axial load on individual cantilever column elements calculated in accordance with the load combinations of Section 2.1.2 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 2.1.3 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 overstrength factor of Section 3.4.4.3.2.

3.4.2.5.3 Inverted Pendulum-Type Structures

Regardless of the structural system selected, inverted pendulums as defined in Section 3.4.1.2, shall comply with this section. 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 3.4.8 and varying uniformly to a moment at the top equal to one-half the calculated bending moment at the base.

3.4.2.5.4 Increased Building Height Limit for Steel Braced Frames and Special Reinforced Concrete Shear Walls

The height limits in Table 3.4.8 are permitted to be increased from160 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 3.4.9 (horizontal structural irregularity Type H1b).
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.

3.4.2.5.5 Special Moment Frames in Structures Assigned to Seismic Design Categories D Through F

For structures assigned to Seismic Design Categories D, E, or F, a special moment frame that is used but not required by Table 3.4.8 shall not be discontinued and supported by a more rigid system with a lower response modification coefficient, R, unless the requirements of Sections 3.4.2.3.2 and 3.4.3.3.4 are met. Where a special moment frame is required by Table 3.4.8, the frame shall be continuous to the foundation.

3.4.2.5.6 Single-Storey Steel Ordinary and Intermediate Moment Frames in Structures Assigned to Seismic Design Category D or E

Single-storey steel ordinary moment frames and intermediate moment frames in structures assigned to Seismic Design Category D or E are permitted up to a height of 65 ft (20 m) where the dead load supported by and tributary to the roof does not exceed 20 psf (0.96 kN/m^2). In addition, the dead loads tributary to the moment frame, of the exterior wall more than 35 ft above the base shall not exceed 20 psf (0.96 kN/m^2).

3.4.2.5.7 Other Steel Ordinary and Intermediate Moment Frames in Structures Assigned to Seismic Design Category D or E

Steel ordinary moment frames in structures assigned to Seismic Design Category D or E not meeting the limitations set forth in Section 3.4.2.5.6 are permitted within light-frame construction up to a height of 35 ft (10.6 m) where neither the roof nor the floor dead load supported by and tributary to the moment frames exceeds 35 psf (1.68 kN/m^2). In addition, the dead load of the exterior walls tributary to the moment frame shall not exceed 20 psf (0.96 kN/m^2). Steel intermediate moment frames in structures assigned to Seismic Design Category D or E not meeting the limitations set forth in Section 3.4.2.5.6 permitted as follows:

1. In Seismic Design Category D, intermediate moment frames are permitted to a height of 35 ft (10.6 m).
2. In Seismic Design Category E, intermediate moment frames are permitted to a height of 35 ft (10.6 m) provided neither the roof nor the floor dead load supported by and tributary to the moment frames exceeds 35 psf (1.68 kN/m^2). In addition, the dead load of the exterior walls tributary to the moment frame shall not exceed 20 psf (0.96 kN/m^2).

3.4.2.5.8 Single-Storey Steel Ordinary and Intermediate Moment Frames in Structures Assigned to Seismic Design Category F

Single-storey steel ordinary moment frames and intermediate moment frames in structures assigned to Seismic Design Category F are permitted up to a height of 65 ft (20 m) where the dead load supported by and tributary to the roof does not exceed 20 psf (0.96 kN/m^2). In addition, the dead loads of the exterior walls tributary to the moment frame shall not exceed 20 psf (0.96 kN/m^2).

3.4.2.5.9 Other Steel Intermediate Moment Frame Limitations in Structures Assigned to Seismic Design Category F

In addition to the limitations for steel intermediate moment frames in structures assigned to Seismic Design Category E as set forth in Section 3.4.2.5.7, steel intermediate moment frames in structures assigned to Seismic Design Category F are permitted in light-frame construction.

3.4.2.5.10 Shear Wall-Frame Interactive Systems

The shear strength of the shear walls of the shear wall-frame interactive system shall be at least 75 percent of the design storey shear at each storey. The frames of the shear wall-frame interactive system shall be capable of resisting at least 25 percent of the design storey shear in every storey.

3.4.3 Diaphragm Flexibility, Configuration Irregularities, and Redundancy

3.4.3.1 Diaphragm Flexibility

The structural analysis shall consider the relative stiffnesses of diaphragms and the vertical elements of the seismic force–resisting system. Unless a diaphragm can be idealized as either flexible or rigid in accordance with Sections 3.4.3.1.1, 3.4.3.1.2, or 3.4.3.1.3, the structural analysis shall explicitly include consideration of the stiffness of the diaphragm (i.e., semirigid modeling assumption).

3.4.3.1.1 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 untopped steel decks in one- and two-family residential buildings of light-frame construction shall also be permitted to be idealized as flexible.

3.4.3.1.2 Alternatives to ASCE 7

The following provisions shall be permitted as alternatives to the relevant provisions of ASCE 7. Diaphragms constructed of wood structural panels or untopped steel decking shall also be permitted to be idealized as flexible, provided all of the following conditions are met:

1. Toppings of concrete or similar materials are not placed over wood structural panel diaphragms except for nonstructural toppings no greater than 1½ inches (38 mm) thick.
2. Each line of vertical elements of the lateral-force-resisting system complies with the allowable storey drift of Table 3.4.8.
3. Vertical elements of the lateral-force-resisting system are light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets.
4. Portions of wood structural panel diaphragms that cantilever beyond the vertical elements of the lateral-force-resisting system are designed in accordance with Section 2305.2.5 of the International Building Code.

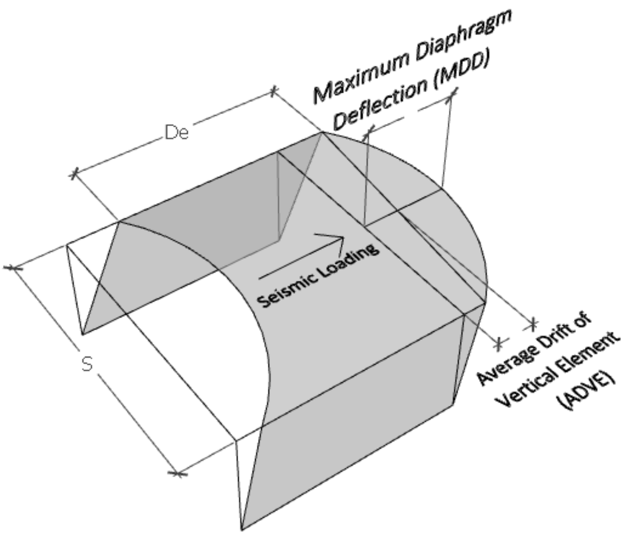


FIGURE 3.4.5 Flexible diaphragm

3.4.3.1.3 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.

3.4.3.1.4 Calculated Flexible Diaphragm Condition

Diaphragms not satisfying the conditions of Sections 3.4.3.1.1 or 3.4.3.1.3 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 storey drift of adjoining vertical elements of the seismic force–resisting system of the associated storey under equivalent tributary lateral load as shown in Fig. 3.4.2. The loadings used for this calculation shall be those prescribed by Section 3.4.8.

3.4.3.2 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.

3.4.3.2.1 Horizontal Irregularity

Structures having one or more of the irregularity types listed in Table 3.4.9 shall be designated as having horizontal structural irregularity.

Such structures assigned to the seismic design categories listed in Table 3.4.9 shall comply with the requirements in the sections referenced in that table.

TABLE 3.4.9 HORIZONTAL STRUCTURAL IRREGULARITIES

Item	Type	Description	SDC					Issue	Reference
			B	C	D	E	F		
H1a	Torsional Irregularity	The maximum storey 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 storey 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.			✓	✓	✓	Force Increase	3.4.3.3.4
					✓	✓	✓	Analysis Limit	3.4.6, Table 3.4.12
			✓	✓	✓	✓	✓	3D Model	3.4.7.3, ASCE7-05 16.2.2
				✓	✓	✓	✓	Torsion Limit	3.4.8.4.3
				✓	✓	✓	✓	Drift Limit	3.4.12.1
H1b	Extreme Torsional Irregularity	The maximum storey 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 storey 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.				✗	✗	Not Permit	3.4.3.3.1
					✓			Force Increase	3.4.3.3.4
					✓			Analysis Limit	3.4.6, Table 3.4.12
			✓	✓	✓			3D Model	3.4.7.3, ASCE7-05 16.2.2
				✓	✓			Torsion Limit	3.4.8.4.3
				✓	✓			Drift Limit	3.4.12.1
H2	Reentrant Corner	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.			✓	✓	✓	Force Increase	3.4.3.3.4
					✓	✓	✓	Drift Limit	3.4.12.1
H3	Diaphragm Discontinuity	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 storey to the next.			✓	✓	✓	Force Increase	3.4.3.3.4
					✓	✓	✓	Drift Limit	3.4.12.1
H4	Out-of-Plane Offsets	There are discontinuities in a lateral force-resistance path, such as out-of-plane offsets of the vertical elements.	✓	✓	✓	✓	✓	Discontinuity	3.4.3.3.3
					✓	✓	✓	Force Increase	3.4.3.3.4
					✓	✓	✓	Analysis Limit	3.4.6, Table 3.4.12
			✓	✓	✓	✓	✓	3D Model	3.4.7.3, ASCE7-05 16.2.2
H5	Nonparallel System	The vertical lateral force-resisting elements are not parallel to or symmetric about the major orthogonal axes of the seismic force–resisting system.		✓	✓	✓	✓	Directional	3.4.5.3
					✓	✓	✓	Analysis Limit	3.4.6, Table 3.4.12
			✓	✓	✓	✓	✓	3D Model	3.4.7.3, ASCE7-05 16.2.2

✓ = Need to be checked

✗ = Not allowed

3.4.3.2.2 Vertical Irregularity

Structures having one or more of the irregularity types listed in Table 3.4.10 shall be designated as having vertical irregularity. Such structures assigned to the Seismic Design Categories listed in Table 3.4.10 shall comply with the requirements in the sections referenced in that table.

TABLE 3.4.10 VERTICAL STRUCTURAL IRREGULARITIES

<table>
<tr><th rowspan="2">Item</th><th rowspan="2">Type</th><th rowspan="2">Description</th><th colspan="5">SDC</th><th rowspan="2">Issue</th><th rowspan="2">Reference</th></tr>
<tr><th>B</th><th>C</th><th>D</th><th>E</th><th>F</th></tr>
<tr><td>V1a</td><td>Soft Story (Stiffness)</td><td>A storey in which the lateral stiffness is less than 70% of that in the storey above or less than 80% of the average stiffness of the three storeys above.</td><td></td><td></td><td>✓</td><td>✓</td><td>✓</td><td>Analysis Limit</td><td>3.4.6, Table 3.4.12</td></tr>
<tr><td rowspan="2">V1b</td><td rowspan="2">Extreme Soft Story (Stiffness)</td><td rowspan="2">A storey in which the lateral stiffness is less than 60% of that in the storey above or less than 70% of the average stiffness of the three storeys above.</td><td></td><td></td><td></td><td rowspan="2">✗</td><td rowspan="2">✗</td><td>Not Permit</td><td>3.4.3.3.1</td></tr>
<tr><td></td><td></td><td>✓</td><td>Analysis Limit</td><td>3.4.6, Table 3.4.12</td></tr>
<tr><td>V2</td><td>Weight Irregularity</td><td>The effective mass of any storey is more than 150% of the effective mass of an adjacent storey. A roof that is lighter than the floor below need not be considered.</td><td></td><td></td><td>✓</td><td>✓</td><td>✓</td><td>Analysis Limit</td><td>3.4.6, Table 3.4.12</td></tr>
<tr><td>V3</td><td>Vertical Geometry</td><td>The horizontal dimension of the seismic force–resisting system in any storey is more than 130% of that in an adjacent storey.</td><td></td><td></td><td>✓</td><td>✓</td><td>✓</td><td>Analysis Limit</td><td>3.4.6, Table 3.4.12</td></tr>
<tr><td rowspan="3">V4</td><td rowspan="3">In-plane Discontinuity</td><td rowspan="3">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 storey below.</td><td>✓</td><td>✓</td><td>✓</td><td>✓</td><td>✓</td><td>Discontinuity</td><td>3.4.3.3.3</td></tr>
<tr><td></td><td></td><td></td><td>✓</td><td>✓</td><td>Force Increase</td><td>3.4.3.3.4</td></tr>
<tr><td></td><td></td><td>✓</td><td>✓</td><td>✓</td><td>Analysis Limit</td><td>3.4.6, Table 3.4.12</td></tr>
<tr><td rowspan="2">V5a</td><td rowspan="2">Weak Story (Strength)</td><td rowspan="2">The storey lateral strength is less than 80% of that in the storey above. The storey lateral strength is the total lateral strength of all seismic-resisting elements sharing the storey shear for the direction under consideration.</td><td></td><td></td><td></td><td rowspan="2">✗</td><td rowspan="2">✗</td><td>Not Permit</td><td>3.4.3.3.1</td></tr>
<tr><td></td><td></td><td>✓</td><td>Analysis Limit</td><td>3.4.6, Table 3.4.12</td></tr>
<tr><td rowspan="3">V5b</td><td rowspan="3">Extreme Weak Story (Strength)</td><td rowspan="3">The storey lateral strength is less than 65% of that in the storey above. The storey strength is the total strength of all seismic-resisting elements sharing the storey shear for the direction under consideration.</td><td></td><td></td><td rowspan="3">✗</td><td rowspan="3">✗</td><td rowspan="3">✗</td><td>Not Permit</td><td>3.4.3.3.1</td></tr>
<tr><td>✓</td><td>✓</td><td>Height Limit</td><td>3.4.3.3.2</td></tr>
<tr><td></td><td></td><td>Analysis Limit</td><td>3.4.6, Table 3.4.12</td></tr>
</table>

✓ = Need to be checked

✗ = Not allowed

EXCEPTIONS:

1. Vertical structural irregularities of Types V1a, V1b, or V2 in Table 3.4.10 do not apply where no storey drift ratio under design lateral seismic force is greater than 130 percent of the storey drift ratio of the next storey above. Torsional effects need not be considered in the calculation of storey drifts. The storey drift ratio relationship for the top two storeys of the structure are not required to be evaluated.
2. Irregularities of Types V1a, V1b, and V2 in Table 3.4.10 are not required to be considered for one-storey buildings in any Seismic Design Category or for two-storey buildings assigned to Seismic Design Categories B, C, or D.

3.4.3.3 Limitations and Additional Requirements for Systems with Structural Irregularities

3.4.3.3.1 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 H1b of Table 3.4.9 or vertical irregularities Type V1b, V5a, or V5b of Table 3.4.10 shall not be permitted. Structures assigned to Seismic Design Category D having vertical irregularity Type V5b of Table 3.4.9 shall not be permitted.

3.4.3.3.2 Extreme Weak Storeys

Structures with a vertical irregularity Type V5b as defined in Table 3.4.10, shall not be over two storeys or 30 ft (9 m) in height.

EXCEPTION:

The limit does not apply where the "weak" storey is capable of resisting a total seismic force equal to Ω_0 times the design force prescribed in Section 3.4.2.9.

3.4.3.3.3 Elements Supporting Discontinuous Walls or Frames

Columns, beams, trusses, or slabs supporting discontinuous walls or frames of structures having horizontal irregularity Type H4 of Table 3.4.9 or vertical irregularity Type V4 of Table 3.4.10 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 3.4.4.3.2. 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.

3.4.3.3.4 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 H1a, H1b, H2, H3, or H4 in Table 3.4.9 or a vertical structural irregularity of Type V4 in Table 3.4.10, the design forces determined from Section 3.4.8.1 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 3.4.4.3.2, in accordance with Section 3.4.10.2.1.

3.4.3.4 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.

3.4.3.4.1 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 3.4.4.3.2 are used.
6. Design of members or connections where the load combinations with overstrength of Section 3.4.4.3.2 are required for design.

7. Diaphragm loads determined using Eq. (3.4.36).
8. Structures with damping systems

3.4.3.4.2 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 storey resisting more than 35 percent of the base shear in the direction of interest shall comply with Table 3.4.11.
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 storey 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 storey height or two times the length of shear wall divided by the storey height for light- framed construction.

TABLE 3.4.11 REQUIREMENTS FOR EACH STOREY RESISTING MORE THAN 35% OF THE BASE SHEAR

Lateral Force-Resisting Element	Requirement
Braced Frames	Removal of an individual brace, or connection thereto, would not result in more than a 33% reduction in storey strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type H1b).
Moment Frames	Loss of moment resistance at the beam-to-column connections at both ends of a single beam would not result in more than a 33% reduction in storey strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type H1b).
Shear Walls or Wall Pier with a height-to-length ratio of greater than 1.0	Removal of a shear wall or wall pier with a height-to-length ratio greater than 1.0 within any storey, or collector connections thereto, would not result in more than a 33% reduction in storey strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type H1b).
Cantilever Columns	Loss of moment resistance at the base connections of any single cantilever column would not result in more than a 33% reduction in storey strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type H1b).
Other	No requirements

3.4.4 Seismic Load Effects and Combinations

3.4.4.1 Applicability

All members of the structure, including those not part of the seismic force–resisting system, shall be designed using the seismic load effects of Section 3.4.4 unless otherwise exempted by this standard. Seismic load effects are the axial, shear, and flexural member forces resulting from application of horizontal and vertical seismic forces as set forth in Section 3.4.4.2. Where specifically required, seismic load effects shall be modified to account for system overstrength, as set forth in Section 3.4.4.3.

3.4.4.2 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 3.2.1.2.2 or load combinations 5 and 6 in Section 3.2.1.3.1, E shall be determined in accordance with Eq. (3.4.12) as follows:

$$E = E_h + E_v \qquad \textit{Eq. (3.4.12)}$$

2. For use in load combination 7 in Section 3.2.1.2.1 or load combination 8 in Section 3.2.1.3.1, E shall be determined in accordance with Eq. (3.4.13) as follows:

$$E = E_h - E_v \qquad \textit{Eq. (3.4.13)}$$

where

E = seismic load effect
E_h = effect of horizontal seismic forces as defined in Section 3.4.4.2.1
E_v = effect of vertical seismic forces as defined in Section 3.4.4.2.2

3.4.4.2.1 Horizontal Seismic Load Effect

The horizontal seismic load effect, E_h , shall be determined in accordance with Eq.(3.4.14) as follows:

$$E_h = \rho\, Q_E \qquad \textit{Eq. (3.4.14)}$$

where

Q_E = effects of horizontal seismic forces from V or F_p . Where required in Sections 3.4.5.3 and 3.4.5.4, 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 3.4.3.4

3.4.4.2.2 Vertical Seismic Load Effect

The vertical seismic load effect, E_v, shall be determined in accordance with Eq. (3.4.15) as follows:

$$E_v = 0.2\, S_{DS}\, D \qquad \textit{Eq. (3.4.15)}$$

where

S_{DS} = design spectral response acceleration parameter at short periods obtained from Section 3.4.1.4.4
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. (3.4.12), (3.4.13), (3.4.16), and (3.4.17) where S_{DS} is equal to or less than 0.125.
2. In Eq. (3.4.14) where determining demands on the soil-structure interface of foundations.

3.4.4.2.3 Seismic Load Combinations

Where the prescribed seismic load effect, E, defined in Section 3.4.4.2 is combined with the effects of other loads as set forth in Section 3.2, the following seismic load combinations for structures not subject to flood or atmospheric ice loads shall be used in lieu of the seismic load combinations in either Section 3.2.1.2.2 or 3.2.1.3.1.

Basic combinations for strength design (see Sections 3.2.1.2.2 and 3.1.1.2 for notation)

5. $(1.2 + 0.2\ S_{DS})\ D + \rho\ Q_E + L$
7. $(0.9 - 0.2\ S_{DS})\ D + \rho\ Q_E + 1.6\ H$

NOTES:

1. The load factor on L in combination 5 is permitted to equal 0.5 for all occupancies in which L_0 in Table 3.2.2 is less than or equal to 100 psf (4.79 kN/m^2), with the exception of garages or areas occupied as places of public assembly.
2. The load factor on H shall be set equal to zero in combination 7 if the structural action due to H counteracts that due to E. Where lateral earth pressure provides resistance to structural actions from other forces, it shall not be included in H but shall be included in the design resistance.

Basic combinations for allowable stress design (see Sections 3.2.1.3.1 and 3.1.1.2 for notation).

5. $(1.0 + 0.14\ S_{DS})\ D + H + F + 0.7\ \rho\ Q_E$
6. $(1.0 + 0.105\ S_{DS})\ D + H + F + 0.525\ \rho\ Q_E + 0.75L + 0.75(L_r \text{ or } R)$
8. $(0.6 - 0.14S_{DS})\ D + 0.7\ \rho\ Q_E + H$

3.4.4.3 Seismic Load Effect Including Overstrength Factor

Where specifically required, conditions requiring overstrength factor applications shall be determined in accordance with the following:

1. For use in load combination 5 in Section 3.2.1.2.2 or load combinations 5 and 6 in Section 3.2.1.3.1, E shall be taken equal to E_m as determined in accordance with Eq. (3.4.16) as follows:

$$E_m = E_{mh} + E_v \qquad \textit{Eq. (3.4.16)}$$

2. For use in load combination 7 in Section 3.2.1.2.2 or load combination 8 in Section 3.2.1.3.1, E shall be taken equal to Em as determined in accordance with Eq. (3.4.17) as follows:

$$E_m = E_{mh} - E_v \qquad \textit{Eq. (3.4.17)}$$

where

E_m = seismic load effect including overstrength factor
E_{mh} = effect of horizontal seismic forces including structural overstrength as defined in Section 3.4.4.3.1
E_v = vertical seismic load effect as defined in Section 3.4.4.2.2

3.4.4.3.1 Horizontal Seismic Load Effect with Overstrength Factor

The horizontal seismic load effect with overstrength factor, E_{mh}, shall be determined in accordance with Eq. (3.4.18) as
follows:

$$E_{mh} = \Omega_o Q_E \quad \text{Eq. (3.4.18)}$$

where

Q_E = effects of horizontal seismic forces from V as specified in Sections 3.4.8.1 . Where required in Sections 3.4.5.3 and 3.4.5.4, such effects shall result from application of horizontal forces simultaneously in two directions at right angles to each other.

Ω_o = overstrength factor

EXCEPTION:

The value of E_{mh} need not exceed the maximum force that can develop in the element as determined by a rational, plastic mechanism analysis or nonlinear response analysis utilizing realistic expected values of material strengths.

3.4.4.3.2 Load Combinations with Overstrength Factor

Where the seismic load effect with overstrength, E_m, defined in Section 3.4.4.3 is combined with the effects of other loads as set forth in Section 2, the following seismic load combination for structures not subject to flood or atmospheric ice loads shall be used in lieu of the seismic load combinations in either Section 3.2.1.2.2 or 3.2.1.3.1:

Basic combinations for strength design with overstrength factor (see Sections 3.2.1.2.2 and 3.1.1.2 for notation)

5. $(1.2 + 0.2S_{DS})\, D + \Omega_o Q_E + L$

7. $(0.9 - 0.2S_{DS})\, D + \Omega_o Q_E + 1.6\, H$

NOTES:

1. The load factor on L in combination 5 is permitted to equal 0.5 for all occupancies in which L_0 in Table 3.2.2 is less than or equal to 100 psf (4.79 kN/m^2), with the exception of garages or areas occupied as places of public assembly.
2. The load factor on H shall be set equal to zero in combination 7 if the structural action due to H counteracts that due to E. Where lateral earth pressure provides resistance to structural actions from other forces, it shall not be included in H but shall be included in the design resistance.

Basic combinations for allowable stress design with overstrength factor (see Sections 2.1.3.1 and 1.1.2 for notation).

5. $(1.0 + 0.14S_{DS})\, D + H + F + 0.7\, \Omega_o Q_E$

6. $(1.0 + 0.105S_{DS})\, D + H + F + 0.525\, \Omega_o Q_E + 0.75L + 0.75(L_r \text{ or } R)$

8. $(0.6 - 0.14S_{DS})\, D + 0.7\, \Omega_o Q_E + H$

3.4.4.3.3 Allowable Stress Increase for Load Combinations with Overstrength

Where allowable stress design methodologies are used with the seismic load effect defined in Section 3.4.4.3 applied in load combinations 5, 6, or 8 of Section 3.2.1.3.1, allowable stresses are permitted to be determined using an allowable stress increase of 1.2. This increase shall not be combined with increases in allowable stresses or load combination reductions otherwise permitted by this standard or the material reference document except that combination with the duration of load increases permitted in AF&PA NDS (Refer to ASCE7-05) is permitted.

3.4.4.4 Minimum Upward Force for Horizontal Cantilevers for Seismic Design Categories D Through F

In structures assigned to Seismic Design Category D, E, or F, horizontal cantilever structural components shall be designed for a minimum net upward force of 0.2 times the dead load in addition to the applicable load combinations of Section 3.4.4.

3.4.5 Direction of Loading

3.4.5.1 Direction of Loading Criteria

The directions of application of seismic forces used in the design shall be those which will produce the most critical load effects. It is permitted to satisfy this requirement using the procedures of Section 3.4.5.2 for Seismic Design Category B, Section 3.4.5.3 for Seismic Design Category C, and Section 3.4.5.4 for Seismic Design Categories D, E, and F.

3.4.5.2 Seismic Design Category B

For structures assigned to Seismic Design Category B, the design seismic forces are permitted to be applied independently in each of two orthogonal directions and orthogonal interaction effects are permitted to be neglected.

3.4.5.3 Seismic Design Category C

Loading applied to structures assigned to Seismic Design Category C shall, as a minimum, conform to the requirements of Section 3.4.5.2 for Seismic Design Category B and the requirements of this section. Structures that have horizontal structural irregularity Type H5 in Table 3.4.9 shall use one of the following procedures:

a. **Orthogonal Combination Procedure**
   The structure shall be analyzed using the equivalent lateral force analysis procedure of Section 3.4.8, the modal response spectrum analysis procedure of Section 3.4.9, or the linear response history procedure of Section 16.1 (ASCE 7-05), as permitted under Section 3.4.6, with the loading applied independently in any two orthogonal directions and the most critical load effect due to direction of application of seismic forces on the structure is permitted to be assumed to be satisfied if components and their foundations are designed for the following combination of prescribed loads: 100 percent of the forces for one direction plus 30 percent of the forces for the perpendicular direction; the combination requiring the maximum component strength shall be used.
b. **Simultaneous Application of Orthogonal Ground Motion**
   The structure shall be analyzed using the linear response history procedure of Section 16.1 (ASCE 7-05) or the nonlinear response history procedure of Section 16.2 (ASCE 7-05), as permitted by Section 3.4.6, with orthogonal pairs of ground motion acceleration histories applied simultaneously.

3.4.5.4 Seismic Design Categories D Through F

Structures assigned to Seismic Design Category D, E, or F shall, as a minimum, conform to the requirements of Section 3.4.5.3. In addition, any column or wall that forms part of two or more intersecting seismic force–resisting systems and is subjected to axial load due to seismic forces acting along either principal plan axis equaling or exceeding 20 percent of the axial design strength of the column or wall shall be designed for the most critical load effect due to application of seismic forces in any direction. Either of the procedures of Section 3.4.5.3 a or b are permitted to be used to satisfy this requirement. Except as required by Section 3.4.7.3, 2-D analyses are permitted for structures with flexible diaphragms.

3.4.6 Analysis Procedure Selection

The structural analysis required by Section 3.4 shall consist of one of the types permitted in Table 3.4.12, based on the structure's Seismic Design Category, structural system, dynamic properties, and regularity, or with the approval of the authority having jurisdiction, an alternative generally accepted procedure is permitted to be used. The analysis procedure selected shall be completed in accordance with the requirements of the corresponding section referenced in Table 3.4.12.

3.4.7 Modeling Criteria

3.4.7.1 Foundation Modeling

For purposes of determining seismic loads, it is permitted to consider the structure to be fixed at the base. Alternatively, where foundation flexibility is considered it shall be in accordance with section 12.13.3 (ASCE 7-05) or Chapter 19 (ASCE 7-05).

TABLE 3.4.12 PERMITTED ANALYTICAL PROCEDURES

Seismic Design Category	Structural Characteristics	Equivalent Lateral Force Analysis (Sec. 3.4.8)	Modal Response Spectrum Analysis (Sec. 3.4.9)	Response History Procedures (Chap. 16 ASCE 7-05)
B, C	All 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 < 3.5Ts and having only horizontal irregularities Type H2, H3, H4, or H5 of Table 3.4.9 or vertical irregularities Type V4, V5a, or V5b of Table 3.4.10	P	P	P
	All other structures	NP	P	P

NOTE: P: Permitted; NP: Not Permitted.

3.4.7.2 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 3.2.3.2.2in 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.

3.4.7.3 Structural Modeling

A mathematical model of the structure shall be constructed for the purpose of determining member forces and structure displacements resulting from applied loads and any imposed displacements or P-Delta effects. The model shall include the stiffness and strength of elements that are significant to the distribution of forces and deformations in the structure and represent the spatial distribution of mass and stiffness throughout the structure.

Structures that have horizontal structural irregularity Type H1a, H1b, H4, or H5 of Table 3.4.9 shall be analyzed using a 3-D representation. Where a 3-D model is used, a minimum of three dynamic degrees of freedom consisting of translation in two orthogonal plan directions and torsional rotation about the vertical axis shall be included at each level of the structure. Where the diaphragms have not been classified as rigid or flexible in accordance with Section 3.4.3.1, the model shall include representation of the diaphragm's stiffness characteristics and such additional dynamic degrees of freedom as are required to account for the participation of the diaphragm in the structure's dynamic response. In addition, the model shall comply with the following:

a. Stiffness properties of concrete and masonry elements shall consider the effects of cracked sections.
b. For steel moment frame systems, the contribution of panel zone deformations to overall storey drift shall be included.

3.4.7.4 Interaction Effects

Moment-resisting frames that are enclosed or adjoined by elements that are more rigid and not considered to be part of the seismic force–resisting system shall be designed so that the action or failure of those elements will not impair the vertical load and seismic force–resisting capability of the frame. The design shall provide for the effect of these rigid elements on the structural system at structural deformations corresponding to the design storey drift (Δ) as determined in Section 12.8.6 (ASCE 7-05). In addition, the effects of these elements shall be considered where determining whether a structure has one or more of the irregularities defined in Section 12.3.2 (ASCE 7-05).

3.4.8 Equivalent Lateral Force Procedure

3.4.8.1 Seismic Base Shear

The seismic base shear, V, in a given direction shall be determined in accordance with the following equation:

$$V = C_s W \qquad \textit{Eq. (3.4.19)}$$

where

C_s= the seismic response coefficient determined in accordance with Section 3.4.8.1.1

W= the effective seismic weight per Section 3.4.7.2.

3.4.8.1.1 Calculation of Seismic Response Coefficient

The seismic response coefficient, C_s, shall be determined in accordance with Eq. (3.4.20).

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} \qquad \textit{Eq. (3.4.20)}$$

where

S_{DS} = the design spectral response acceleration parameter in the short period range as determined from Section 3.4.1.4.4.

R = the response modification factor in Table 3.4.11
I = the occupancy importance factor determined in accordance with Section 3.4.1.5.1
The value of C_s computed in accordance with Eq. (3.4.20) need not exceed the following:

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} \quad for T \leq T_L \qquad Eq. (3.4.21)$$

$$C_s = \frac{S_{D1}T_L}{T^2\left(\frac{R}{I}\right)} \quad for\ T > T_L \qquad Eq. (3.4.22)$$

C_s shall not be less than

$$C_s = 0.044\ S_{DS}I \geq 0.01 \qquad Eq. (3.4.23)$$

In addition, for structures located where $S_1$is equal to or greater than 0.6g, C_s shall not be less than

$$C_s = \frac{0.5S_1}{\left(\frac{R}{I}\right)} \qquad Eq. (3.4.24)$$

where I and R are as defined in Section 3.4.8.1.1 and
S_{D1} = the design spectral response acceleration parameter at a period of 1.0 s, as determined from Section 3.4.1.4.4
T = the fundamental period of the structure (s) determined in Section 3.4.8.2
T_L= long-period transition period (s) determined in Section 3.4.1.4.5
S_1= the mapped maximum considered earthquake spectral response acceleration parameter determined in accordance with Section 3.4.1.4.1.

3.4.8.1.2 Soil Structure Interaction Reduction

A soil structure interaction reduction is permitted where determined using Chapter19 (ASCE 7-05) or other generally accepted procedures approved by the authority having jurisdiction.

3.4.8.1.3 Maximum S_s Value in Determination of C_s

For regular structures five storeys 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.

3.4.8.2 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 3.4.13 and the approximate fundamental period, T_a, in accordance with Section 3.4.8.2.1. 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 3.4.8.2.1, directly.

TABLE 3.4.13 COEFFICIENT FOR UPPER LIMIT ON CALCULATED PERIOD

Design Spectral Response Acceleration Parameter at 1 s, 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

3.4.8.2.1 Approximate Fundamental Period

The approximate fundamental period (T_a), in s, shall be determined from the following equation:

$$T_a = C_t\, h_n^x \qquad \textit{Eq. (3.4.25)}$$

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 3.4.14.

TABLE 3.4.14 VALUES OF APPROXIMATE PERIOD PARAMETERS C_t AND x

Structure Type	C_t	x
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

Note: 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:

[a] Metric equivalent 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 storeys in height in which the seismic force–resisting system consists entirely of concrete or steel moment resisting frames and the storey height is at least 10 ft (3 m):

$$T_a = 0.1N \qquad \textit{Eq. (3.4.26)}$$

where

N = number of storeys.

The approximate fundamental period, T_a , in s for masonry or concrete shear wall structures is permitted to be determined from Eq. (3.4.27) as follows:

$$T_a = \frac{0.0019}{\sqrt{C_w}} h_n \qquad \textit{Eq. (3.4.27)}$$

where h_n is as defined in the preceding text and C_w is calculated from Eq. (3.4.28) as follows:

$$C_w = \frac{100}{A_B}\sum_{i=1}^{x}\left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{\left[1+0.83\left(\frac{h_i}{D_i}\right)^2\right]} \qquad Eq.(3.4.28)$$

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.

3.4.8.3 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:

$$F_x = C_{vx} V \qquad Eq.\ (3.4.29)$$

and

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad Eq.\ (3.4.30)$$

where
C_{vx} = vertical distribution factor
V = total design lateral force or shear at the base of the structure (kip or kN)
w_i , w_x = the portion of the total effective seismic weight of the structure (W) located or assigned to Level i or x
h_i , 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 2.5 s or more, k = 2
- for structures having a period between 0.5 and 2.5s, k shall be 2 or shall be determined by linear interpolation between 1 and 2

3.4.8.4 Horizontal Distribution of Forces

The seismic design storey shear in any storey (V_x) (kip or kN) shall be determined from the following equation:

$$V_x = \sum_{i=x}^{n} F_i \qquad Eq.\ (3.4.31)$$

where F_i = the portion of the seismic base shear (V) (kip or kN) induced at Level i.
The seismic design storey shear (V_x) (kip or kN) shall be distributed to the various vertical elements of the seismic force-resisting system in the storey under consideration based on the relative lateral stiffness of the vertical resisting elements and the diaphragm.

3.4.8.4.1 Inherent Torsion

For diaphragms that are not flexible, the distribution of lateral forces at each level shall consider the effect of the inherent torsional moment, M_t, resulting from eccentricity between the locations of the center of mass and the center of rigidity. For flexible diaphragms, the distribution of forces to the vertical elements shall account for the position and distribution of the masses supported.

3.4.8.4.2 Accidental Torsion

Where diaphragms are not flexible, the design shall include the inherent torsional moment (M_t) (kip or kN) resulting from the location of the structure masses plus the accidental torsional moments (M_{ta}) (kip or kN) caused by assumed displacement of the center of mass each way from its actual location by a distance equal to 5 percent of the dimension of the structure perpendicular to the direction of the applied forces.

Where earthquake forces are applied concurrently in two orthogonal directions, the required 5 percent displacement of the center of mass need not be applied in both of the orthogonal directions at the same time, but shall be applied in the direction that produces the greater effect.

3.4.8.4.3 Amplification of Accidental Torsional Moment

Structures assigned to Seismic Design Category C, D, E, or F, where Type 1a or 1b torsional irregularity exists as defined in Table 3.4.9 shall have the effects accounted for by multiplying M_{ta} at each level by a torsional amplification factor (A_x) as illustrated in Fig. 3.4.6 and determined from the following equation:

$$A_x = \left(\frac{d_{max}}{1.2d_{avg}}\right)^2 \leq 3 \qquad \textit{Eq. (3.4.32)}$$

where

δ_{max} = the maximum displacement at Level x (in. or mm) computed assuming $A_x = 1$

δ_{avg} = the average of the displacements at the extreme points of structure at Level x computed assuming $A_x = 1$

EXCEPTION:

The accidental torsional moment need not be amplified for structures of light-frame construction.

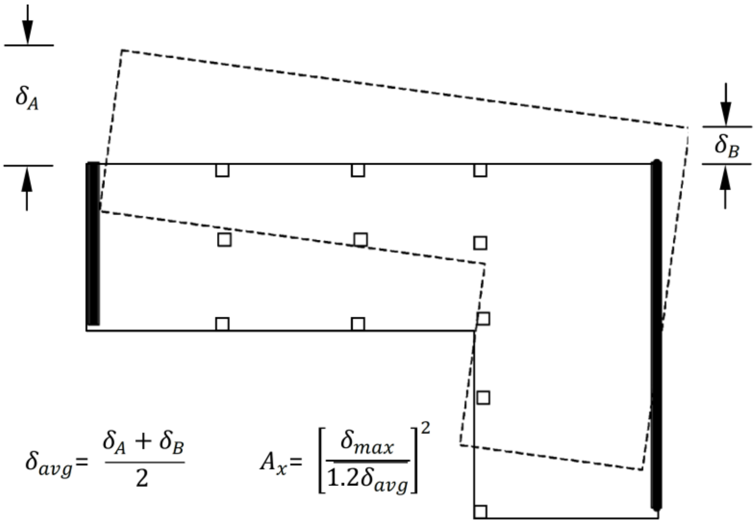


Figure 3.4.6 Torsional Amplification Factor, A_x

3.4.8.5 Overturning

The structure shall be designed to resist overturning effects caused by the seismic forces determined in Section 3.4.8.3.

3.4.8.6 Storey Drift Determination

The design storey drift (Δ) shall be computed as the difference of the deflections at the centers of mass at the top and bottom of the storey under consideration. See Fig.3.4.7. Where allowable stress design is used, Δ shall be computed using the strength level seismic forces specified in Section 3.4.8 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:

$$d_x = \frac{C_d d_{xe}}{I} \qquad \textit{Eq. (3.4.33)}$$

where,

C_d = the deflection amplification factor in Table 3.4.8
δ_{xe} = the deflections determined by an elastic analysis
I = the importance factor determined in accordance with Section 3.4.1.5.1

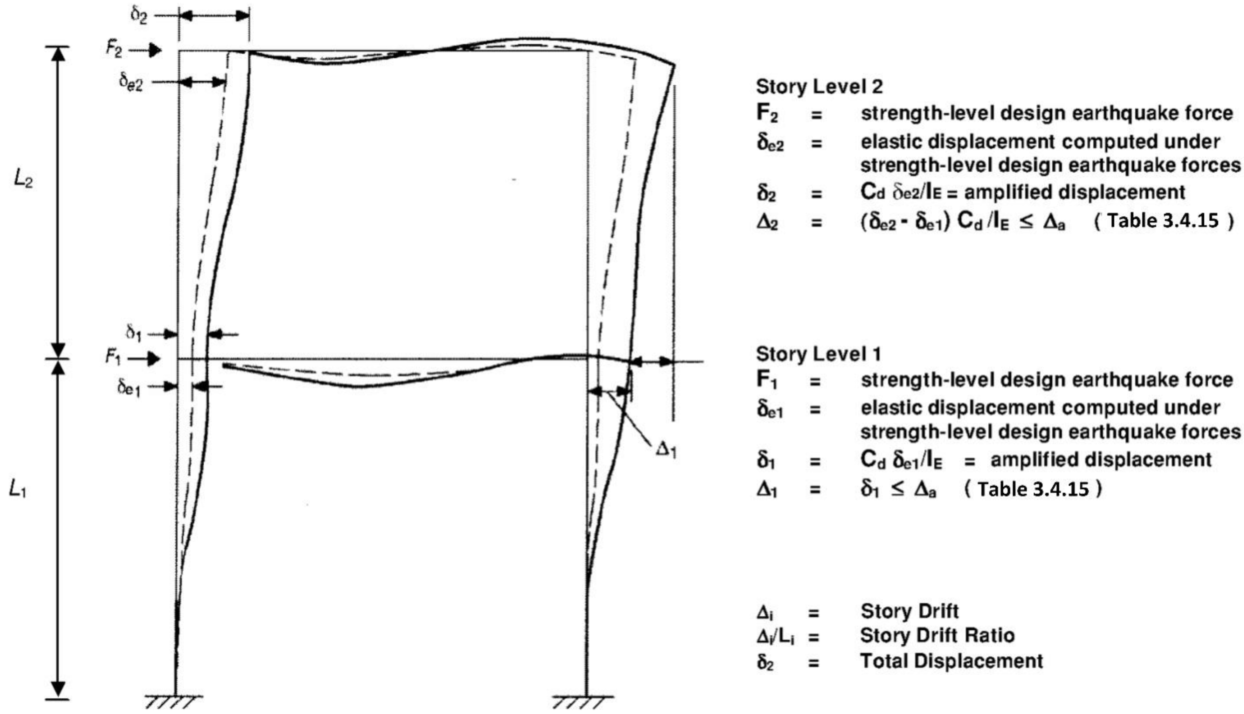


Figure 3.4.7 Storey Drift Determination

3.4.8.6.1 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 Section 3.4.8.

3.4.8.6.2 Period for Computing Drift

For determining compliance with the story drift limits of Section 12.12.1(ASCE 7-05), 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_uT_a) specified in Section 3.4.8.2.

3.4.8.7 P-Delta Effects

P-delta effects on storey shears and moments, the resulting member forces and moments, and the storey 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:

$$q = \frac{P_x D}{V_x h_{sx} C_d} \qquad \text{Eq. (3.4.34)}$$

where,

P_x = the total vertical design load at and above Level x (kip or kN); where computing Px, no individual load factor need exceed 1.0

Δ = the design storey drift as defined in Section 3.4.8.6 occurring simultaneously with Vx (in. or mm)

V_x = the seismic shear force acting between Levels x and x −1 (kip or kN)

h_{sx} = the storey height below Level x (in. or mm)

C_d = the deflection amplification factor in Table 3.4.7

The stability coefficient (θ) shall not exceed θ_{max} determined as follows:

$$q_{max} = \frac{0.5}{b\, C_d} \leq 0.25 \qquad \text{Eq. (3.4.35)}$$

where,

β is the ratio of shear demand to shear capacity for the storey 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−θ).

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. (3.4.35) shall still be satisfied, however, the value of θ computed from Eq. (3.4.34) using the results of the P-delta analysis is permitted to be divided by (1 + θ) before checking Eq. (3.4.35).

3.4.9 Modal Response Spectrum Analysis

3.4.9.1 Number of Modes

An analysis shall be conducted to determine the natural modes of vibration for the structure. The analysis shall include a sufficient number of modes to obtain a combined modal mass participation of at least 90 percent of the actual mass in each of the orthogonal horizontal directions of response considered by the model.

3.4.9.2 Modal Response Parameters

The value for each force- related design parameter of interest, including storey drifts, support forces, and individual member forces for each mode of response shall be computed using the properties of each mode and the response spectra defined in either Section 3.4.1.4.5 divided by the quantity *R/I*. The value for displacement and drift quantities shall be multiplied by the quantity C_d/I.

3.4.9.3 Combined Response Parameters

The value for each parameter of interest calculated for the various modes shall be combined using either the square root of the sum of the squares method (SRSS) or the complete quadratic combination method (CQC), in accordance with ASCE 3.4. The CQC method shall be used for each of the modal values or where closely spaced modes that have significant cross-correlation of translational and torsional response.

3.4.9.4 Scaling Design Values of Combined Response

A base shear *(V)* shall be calculated in each of the two orthogonal horizontal directions using the calculated fundamental period of the structure T in each direction and the procedures of Section 3.4.8, except where the calculated fundamental period exceeds $(C_u)\ (T_a)$, then $(C_u)(T_a)$ shall be used in lieu of T in that direction. Where the combined response for the modal base shear (V_t) is less than 85 percent of the calculated base shear *(V)* using the equivalent lateral force procedure, the forces, but not the drifts, shall be multiplied by $0.85\,\frac{V}{V_t}$:

where

V = the equivalent lateral force procedure base shear, calculated in accordance with this section and Section 3.4.8.

V_t = the base shear from the required modal combination

3.4.9.5 Horizontal Shear Distribution

The distribution of horizontal shear shall be in accordance with the requirements of Section 3.4.8.4 except that amplification of torsion per Section 3.4.8.4.3 is not required where accidental torsional effects are included in the dynamic analysis model.

3.4.9.6 P-Delta Effects

The P-delta effects shall be determined in accordance with Section 3.4.8.7. The base shear used to determine the storey shears and the storey drifts shall be determined in accordance with Section 3.4.8.6.

3.4.9.7 Soil Structure Interaction Reduction

A soil structure interaction reduction is permitted where determined using generally accepted procedures approved by the authority having jurisdiction.

3.4.10 Diaphragms, Chords and Collectors

3.4.10.1 Diaphragm Design

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.

3.4.10.1.1 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. (3.4.36) as follows:

$$F_{PX} = 0.2\, S_{DS}\, I w_{PX} \leq \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{PX} \leq 0.4\, S_{DS}\, I w_{PX} \qquad Eq.\ (3.4.36)$$

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

Where the diaphragm is required to transfer design seismic force from the vertical resisting elements above the diaphragm to other vertical resisting elements below the diaphragm due to offsets in the placement of the elements or to changes in relative lateral stiffness in the vertical elements, these forces shall be added to those determined from Eq. (3.4.36). The redundancy factor, ρ , applies to the design of diaphragms in structures assigned to Seismic Design Category D, E, or F. For inertial forces calculated in accordance with Eq. (3.4.36), the redundancy factor shall equal 1.0. For transfer forces, the redundancy factor, ρ, shall be the same as that used for the structure. For structures having horizontal or vertical structural irregularities of the types indicated in Section 3.4.3.3.1, the requirements of that section shall also apply.

3.4.10.2 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.

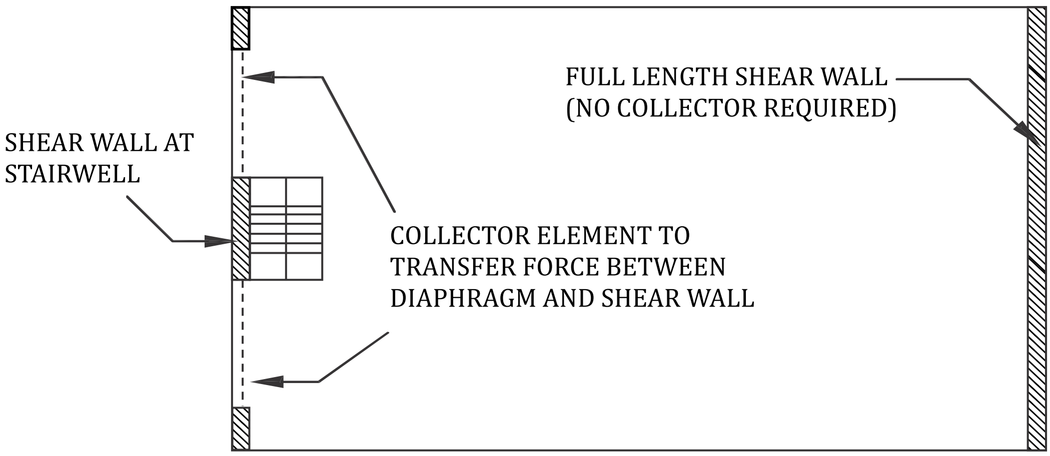


Figure 3.4.8 Collectors

3.4.10.2.1 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. 3.4.8), splices, and their connections to resisting elements shall resist the load combinations with overstrength factor of Section 3.4.4.3.2.

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 3.4.10.1.1.

3.4.11 Structural Walls and Their Anchorage

3.4.11.1 Design for Out-of-Plane Forces

Structural walls and their anchorage shall be designed for a force normal to the surface equal to $0.4S_{DS}I$ times the weight of the structural wall with a minimum force of 10 percent of the weight of the structural wall. Interconnection of structural wall elements and connections to supporting framing systems shall have sufficient ductility, rotational capacity, or sufficient strength to resist shrinkage, thermal changes, and differential foundation settlement when combined with seismic forces.

3.4.11.2 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 3.4.11.1.
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 (3.4.09 kN/m) of wall

Structural walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 ft (1,219 mm).

3.4.11.2.1 Anchorage of Concrete or Masonry Structural Walls to Flexible Diaphragms

In addition to the requirements set forth in Section 3.4.11.2, 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. (3.4.37):

$$F_P = 0.8S_{DS}\,IW_P \qquad \text{Eq. (3.4.37)}$$

where,

F_P = the design force in the individual anchors
S_{DS} = the design spectral response acceleration parameter at short periods per Section 3.4.1.4.4
I = the occupancy importance factor per Section 3.4.1.5.1
W_p = the weight of the wall tributary to the anchor

3.4.11.2.2 Additional Requirements for Diaphragms in Structures Assigned to Seismic Design Categories C through F

3.4.11.2.2.1 Transfer of Anchorage Forces into Diaphragm

Diaphragms shall be provided with continuous ties or struts between diaphragm chords to distribute these anchorage forces into the diaphragms. Diaphragm connections shall be positive, mechanical, or welded. Added chords are permitted to be used to form sub-diaphragms to transmit the anchorage forces to the main continuous cross-ties. The maximum length-to-width ratio of the structural sub-diaphragm shall be 2.5 to 1. Connections and anchorages capable of resisting the prescribed forces shall be provided between the diaphragm and the attached components. Connections shall extend into the diaphragm a sufficient distance to develop the force transferred into the diaphragm.

3.4.11.2.2.2 Steel Elements of Structural Wall Anchorage System

The strength design forces for steel elements of the structural wall anchorage system, with the exception of anchor bolts and reinforcing steel, shall be increased by 1.4 times the forces otherwise required by this section.

3.4.11.2.2.3 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. The diaphragm sheathing shall not be considered effective as providing the ties or struts required by this section.

3.4.11.2.2.4 Metal Deck Diaphragms

In metal deck diaphragms, the metal deck shall not be used as the continuous ties required by this section in the direction perpendicular to the deck span.

3.4.11.2.2.5 Embedded Straps

Diaphragm to structural wall anchorage using embedded straps shall be attached to, or hooked around, the reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel.

3.4.11.2.2.6 Eccentrically Loaded Anchorage System

Where elements of the wall anchorage system are loaded eccentrically or are not perpendicular to the wall, the system shall be designed to resist all components of the forces induced by the eccentricity.

3.4.11.2.2.7 Walls with Pilasters

Where pilasters are present in the wall, the anchorage force at the pilasters shall be calculated considering the additional load transferred from the wall panels to the pilasters. However, the minimum anchorage force at a floor or roof shall not be reduced.3.4.12 Drift and Deformation

3.4.12 Drift and Deformation

3.4.12.1 Storey Drift Limit

The design storey drift (Δ) as determined in Sections 3.4.8.6, 3.4.9.2, or 3.4.3.1 shall not exceed the allowable story drift (Δa) as obtained from Table 3.4.15 for any storey. 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 H1a or H1b of Table 3.4.9, the design storey 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 storey under consideration.

TABLE 3.4.15 ALLOWABLE STOREY DRIFT, Δ_a [a,b]

Structure	Occupancy Category		
	I, II	III	IV
Structures, other than masonry shear wall structures, 4 storeys or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the storey drifts.	$0.025h_{sx}$ [c]	$0.020hsx$	$0.015hsx$
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 storey 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 storey drift shall comply with the requirements of Section 3.4.12.1.1.
c. There shall be no drift limit for single-storey structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the storey drifts. The structure separation requirement of Section 3.4.12.3 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.

3.4.12.1.1 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 storey drift (Δ) shall not exceed Δ_a/ρ for any storey. ρ shall be determined in accordance with Section 3.4.3.4.2.

3.4.12.2 Diaphragm Deflection

The deflection in the plane of the diaphragm, as determined by engineering analysis, shall not exceed the permissible deflection of the attached elements. Permissible deflection shall be that deflection that will permit the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads.

3.4.12.3 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 3.4.8.6.

3.4.12.4 Deformation Compatibility for Seismic Design Categories D through F

For structures assigned to Seismic Design Category D, E, or F, every structural component not included in the seismic force–resisting system in the direction under consideration shall be designed to be adequate for the gravity load effects and the seismic forces resulting from displacement to the design storey drift (Δ) as determined in accordance with Section 3.4.8.6 (see also Section 3.4.12.1).

EXCEPTION:

Reinforced concrete frame members not designed as part of the seismic force–resisting system shall comply with Section 21.11 of ACI 318-08.

Where determining the moments and shears induced in components that are not included in the seismic force–resisting system in the direction under consideration, the stiffening effects of adjoining rigid structural and nonstructural elements shall be considered and a rational value of member and restraint stiffness shall be used.

SECTION 3.5
CONCRETE

Italics are used for text within Sections 3.5.3 through 3.5.4 of this PART to indicate provisions that differ from ACI 318-08.

3.5.1 General

3.5.1.1 Scope

The provisions of this section shall govern the materials, quality control, design and construction of concrete used in structures.

3.5.1.2 Plain and Reinforced Concrete

Structural concrete shall be designed and constructed in accordance with the requirements of this SECTION and ACI 318-08 as amended in Section 3.5.4. Except for the provisions of Sections 1904 and 1910 (IBC-2006), the design and construction of slabs on grade shall not be governed by this SECTION unless they transmit vertical loads or lateral forces from other parts of the structure to the soil.

3.5.1.3 Source and Applicability

The format and subject matter of Sections 3.5.2 through 3.5.4 and 3.5.6 of this section are patterned after, and in general conformity with, the provisions for structural concrete in ACI 318-08.

3.5.1.4 Construction Documents

The design and construction documents for structural concrete construction shall include:

1. The specified compressive strength of concrete at the stated ages or stages of construction for which each concrete element is designed.
2. The specified strength or grade of reinforcement.
3. The size and location of structural elements, reinforcement, and anchors.
4. Provision for dimensional changes resulting from creep, shrinkage and temperature.
5. The magnitude and location of prestressing forces.
6. Anchorage length of reinforcement and location and length of lap splices.
7. Type and location of mechanical and welded splices of reinforcement.
8. Details and location of contraction or isolation joints specified for plain concrete.
9. Minimum concrete compressive strength at time of posttensioning.
10. Stressing sequence for posttensioning tendons.
11. For structures assigned to Seismic Design Category D, E or F, a statement if slab on grade is designed as a structural diaphragm (see Section 21.12.3.4 of ACI 318-08).

3.5.2 Definitions

3.5.2.1 General

The words and terms defined in ACI 318-08 shall, for the purposes of this section and as used elsewhere in this part for concrete construction, have the meanings shown in ACI 318-08.

3.5.3 Specifications for Tests and Materials

3.5.3.1 General

Materials used to produce concrete; concrete itself and testing thereof shall comply with the applicable standards listed in ACI 318-08.

3.5.3.2 Glass Fiber Reinforced Concrete

Glass fiber reinforced concrete (GFRC) and the materials used in such concrete shall be in accordance with the PCI MNL 128 standard.

3.5.4 MODIFICATIONS TO ACI318-08

3.5.4.1 General

The text of ACI 318-08 shall be modified as indicated in Sections 3.5.4.1.1 through 3.5.4.1.10.

3.5.4.1.1 ACI 318-08, Section 2.2

Modify existing definitions and add the following definitions to ACI 318-08, Section 2.2.
DESIGN DISPLACEMENT. Total lateral displacement expected for the design-basis earthquake, *as specified by Section* 12.8.6 *of ASCE 7-05.*
DETAILEDPLAINCONCRETESTRUCTURAL WALL. A wall complying with the requirements of Chapter 22, including 22.6.7.
ORDINARYPRECASTSTRUCTURAL WALL. A precast wall complying with the requirements of Chapters 1 *through* 18.
ORDINARY REINFORCED CONCRETE STRUCTURAL WALL. A *cast-in-place* wall complying with the requirements of Chapters 1 through 18.
ORDINARY STRUCTURAL PLAIN CONCRETE WALL. A wall complying with the requirements of Chapter *22, excluding 22.6.7.*
SPECIAL STRUCTURAL WALL. A cast-in-place or precast wall complying with the requirements of 21 .1.3 through 21.1.7,21.9 and 21.10, as applicable, in addition to the requirements for ordinary reinforced concrete structural walls *or ordinary precast structural walls, as applicable. Where ASCE 7-05 refers to a "special reinforced concrete structural wall," it shall be deemed to mean a "special structural wall. "*
WALL PIER. A wall segment with a horizontal length to- thickness ratio of at least 2.5, *but not exceeding* 6, *whose clear height is at least two times its horizontal length.*

3.5.4.1.2 ACI 318-08, Section 21.1.1

Modify ACI 318-08 Sections 21.1.1.3 and 21.1.1.7 to read as follows:
21.1.1.3 - *Structures assigned to Seismic Design Category A* shall satisfy requirements of Chapters 1 to 19 and *22, Chapter* 21 *does not apply.* Structures assigned to *Seismic Design Category B,* C, D, E or F also shall satisfy 21.1.1.4 through 21.1.1.8, as applicable. *Except for structural elements of plain concrete complying with Section 3.5.4.1.8, structural elements of plain concrete are prohibited in structures assigned to Seismic Design Category* C, *D, E or F.*
21.1.1.7 - Structural systems designated as part of the seismic-force-resisting system shall be restricted to those *permitted by ASCE 7-05*. Except for *Seismic Design Category* A, for which Chapter 21 does not apply, the following provisions shall be satisfied for each structural system designated as part of the seismic-force-resisting system, regardless of the *Seismic Design Category:*

a. Ordinary moment frames shall satisfy 21.2.
b. Ordinary reinforced concrete structural walls *and ordinary precast structural walls* need not satisfy any provisions in Chapter 21.

c. Intermediate moment frames shall satisfy 21.3.
d. Intermediate precast *structural walls* shall satisfy 21.4.
e. Special moment frames shall satisfy 21 .5 through 21.8.
f. Special structural walls shall satisfy 21.9.
g. Special structural walls constructed using pre- I cast concrete shall satisfy 21.10. All special moment frames and special structural walls shall also satisfy 21.1.3 through 21.1.7.

3.5.4.1.3 ACI 318-08, Section 21.4

Modify ACI 318-08, Section 21.4, by renumbering Section 21.4.3 to become 21.4.4 and adding new Sections 21.4.3, 21.4.5 and 21.4.6 to read as follows:
21.4.3 - *Connections that are designed to yield shall be capable of maintaining 80 percent of their design strength at the deformation induced by the design displacement or shall use Type* 2 *mechanical splices.*
21.4.4 - Elements of the connection that are not designed to yield shall develop at least 1.5 *Sy.*
21.4.5 - *Wall piers not designed as part of a moment frame shall have transverse reinforcement designed to resist the shear forces determined from* 21.3.3. *Spacing of transverse reinforcement shall not exceed* 8 *inches (203 mm). Transverse reinforcement shall be extended beyond the pier clear height for at least* 12 *inches (305 mm).*
Exceptions:
1. *Wall piers that satisfy 21.13.*
2. *Wall piers along a wall line within a story where other shear wall segments provide lateral support to the wall piers and such segments have a total stiffness of at least six times the sum of the stiffnesses of all the wall piers.*

21.4.6- Wall segments with a horizontal length-to-thickness ratio less than 2.5 *shall be designed as columns.*

3.5.4.1.4 ACI 318-08, Section 21.9

Modify ACI 318-08, Section 21.9, by adding new Section 21.9.10 to read as follows:
21.9.10 - Wall piers and wall segments.
21.9.10.1 - Wall piers not designed as a part of a special moment frame shall have transverse reinforcement designed to satisfy the requirements in 21.9.10.2.
Exceptions:
1. *Wall piers that satisfy 21.13.*
2. *Wall piers along a wall line within a story where other shear wall segments provide lateral support to the wall piers and such segments have a total stiffness of at least six times the sum of the stiffnesses of all the wall piers.*

21.9.10.2 - Transverse reinforcement with seismic hooks at both ends shall be designed to resist the shear forces determined from 21.6.5.1. *Spacing of transverse reinforcement shall not exceed* 6 *inches* (152 *mm). Transverse reinforcement shall be extended beyond the pier clear height for at least* 12 *inches (305 mm).*
21.9.10.3 - Wall segments with a horizontal length-to thickness ratio less than 2.5 *shall be designed as columns.*

3.5.4.1.5 ACI 318-08, Section 21.10

Modify ACI 318-08, Section 21.10.2, to read as follows:
21.10.2 - Special structural walls constructed using precast concrete shall satisfy all the requirements of 21.9 for cast-in-place special structural walls in addition to Sections 21.4.2 *through 21.4.4.*

3.5.4.1.6 ACI 318-08, Section 21.12.1.

Modify ACI 318-08, Section 21.12.1.1, to read as follows:
21.12.1.1 - Foundations resisting earthquake-induced forces or transferring earthquake-induced forces between a structure and ground shall comply with the requirements of Section 21.12 and other applicable provisions of ACI 318-08 *unless modified by Chapter* 18 *of the International Building Code.*

3.5.4.1.7 ACI 318-08, Section 22.6

Modify ACI 318-08, Section 22.6, by adding new Section 22.6.7 to read as follows:
22.6.7 - *Detailed plain concrete structural walls.*
22.6.7.1 - *Detailed plain concrete structural walls are walls conforming to the requirements of ordinary structural plain concrete walls and* 22.6.7.2.
22.6.7.2 - *Reinforcement shall be provided as follows:*

a. *Vertical reinforcement of at least 0.20 square inch* (129 *mm^2) in cross-sectional area shall be provided continuously from support to support at each corner, at each side of each opening and at the ends of walls. The continuous vertical bar required beside an opening is permitted to substitute for one of the two No.5 bars required by* 22.6.6.5.
b. *Horizontal reinforcement at least 0.20 square inch* (129 *mm^2) in cross-sectional area shall be provided:*
    1. *Continuously at structurally connected roof and floor levels and at the top of walls;*
    2. *At the bottom of load-bearing walls or in the top of foundations where doweled to the wall; and*
    3. *At a maximum spacing of 120 inches (3048 mm). Reinforcement at the top and bottom of openings, where used in determining the maximum spacing specified in Item* 3 *above, shall be continuous in the wall.*

3.5.4.1.8 ACI 318-08, Section 22.10

Delete ACI 318-08, Section 22.10, and replace with the following:
22.10 - Plain concrete in structures assigned to Seismic Design Category C, *D, E or F*
22.10.1 - Structures assigned to Seismic Design Category C, *D, E or F shall not have elements of structural*
plain concrete, except as follows:

a. *Structural plain concrete basement, foundation or other walls below the base are permitted in detached one- and two-family dwellings three stories or less in height constructed with stud-bearing walls. In dwellings assigned to Seismic Design Category D or E, the height of the wall shall not exceed* 8 *feet* (2438 *mm), the thickness shall not be less than* 710 *inches (190 mm), and the wall shall retain no more than* 4 *feet (1219 mm) of unbalanced fill. Walls shall have reinforcement in accordance with 22.6.6.5.*
b. *Isolated footings of plain concrete supporting pedestals or columns are permitted, provided the projection of the footing beyond the face of the supported member does not exceed the footing thickness.*

    Exception:
    In detached one- and two-family dwellings three stories or less in height, the projection of the footing beyond the face of the supported member is permitted to exceed the footing thickness.

a. *Plain concrete footings supporting walls are permitted, provided the footings have at least two continuous longitudinal reinforcing bars. Bars shall not be* *smaller than No.* 4 *and shall have a total area of not less than 0.002 times the gross cross-sectional area of the footing. For footings that exceed* 8inches *(203mm) in thickness, a minimum of one bar shall be provided at the top and bottom of the footing. Continuity of reinforcement shall be provided at corners and intersections.*

Exceptions:

1. *In detached one- and two-family dwellings three stories or less in height and constructed with stud-bearing walls, plain concrete footings without longitudinal reinforcement supporting walls are permitted.*
2. *For foundation systems consisting of a plain concrete footing and a plain concrete stem wall, a minimum of one bar shall be provided at the top of the stem wall and at the bottom of the footing.*
3. *Where a slab on ground is cast monolithically with the footing, one No.5 bar is permitted to be located at either the top of the slab or bottom of the footing.*

3.5.4.1.9 ACI 318-08, Section D.3.3

Modify ACI 318-08, Sections D.3.3.4 and D.3.3.5 to read as follows:

D.3.3.4 - Anchors shall be designed to be governed by the steel strength of a ductile steel element as determined in accordance with D. 5.1 and D. 6.1, unless either D.3.3.5 or D.3.3.6 is satisfied.

Exceptions:

1. *Anchors in concrete designed to support nonstructural components in accordance with ASCE 7-05 Section* 13.4.2 *need not satisfy Section D.* 3. 3. 4.
2. *Anchors designed to resist wall out-of-plane forces with design strengths equal to or greater than the force determined in accordance with ASCE 7-05 Equation* 12.11-1 *or 12.14-10 need not satisfy* Section D.3.3.4.

D.3.3.5 - Instead of D.3.3.4, the attachment that the anchor is connecting to the structure shall be designed so that the attachment will undergo ductile yielding at a force level corresponding to anchor forces no greater than the design strength of anchors specified in D.3.3.3.

Exceptions:

1. *Anchors in concrete designed to support nonstructural components in accordance with ASCE 7-05 Section* 13.4.2 *need not satisfy Section D.* 3. 3. 5.
2. *Anchors designed to resist wall out-of-plane forces with design strengths equal to or greater than the force determined in accordance with ASCE 7-05 Equation* 12.11-1 *or 12.14-10 need not satisfy Section D.* 3. 3.5.

3.5.4.1.10 ACI 318-08, Section D.4.2.2

Delete ACI 318-08, Section D.4.2.2, and replace with the following:

D.4.2.2 - The concrete breakout strength requirements for anchors in tension shall be considered satisfied by the design procedure of D.5.2 provided Equation D-8 is not used for anchor embedment exceeding 25*inches. The concrete breakout strength requirements for anchors in shear with diameters not exceeding* 2 *inches shall be considered satisfied by the design procedure of D.6.2. For anchors in shear with diameters exceeding*2 *inches, shear anchor reinforcement shall be provided in accordance with the procedures of D.6.2.9.*

3.5.5 Structural Plain Concrete

3.5.5.1 Scope

The design and construction of structural plain concrete, both cast-in-place and precast, shall comply with the minimum requirements of Section 3.5.5 and Chapter 22 of ACI 318-08, as modified in Section 3.5.4.

3.5.5.1.1 Special Structures

For special structures, such as arches, underground utility structures, gravity walls and shielding walls, the provisions of this section shall govern where applicable.

3.5.5.2 Limitations

The use of structural plain concrete shall be limited to:

1. Members that are continuously supported by soil, such as walls and footings, or by other structural members capable of providing continuous vertical support.
2. Members for which arch action provides compression under all conditions of loading.
3. Walls and pedestals.

The use of structural plain concrete columns and structural plain concrete footings on piles is not permitted. See Section 3.5.4.1.19 for additional limitations on the use of structural plain concrete.

3.5.5.3 Joints

Contraction or isolation joints shall be provided to divide structural plain concrete members into flexurally discontinuous elements in accordance with ACI 318-08, Section 22.3.

3.5.5.4 Design

Structural plain concrete walls, footings and pedestals shall be designed for adequate strength in accordance with ACI 318-08, Sections 22.4 through 22.8.

EXCEPTION:

For Group R-3 occupancies and buildings of other occupancies less than two storeys in height of light-frame construction, the required edge thickness of ACI 318-08 is permitted to be reduced to 6 inches (152 mm), provided that the footing does not extend more than 4 inches (102 mm) on either side of the supported wall.

3.5.5.5 Precast Members

The design, fabrication, transportation and erection of precast, structural plain concrete elements shall be in accordance with ACI 318-08, Section 22.9.

3.5.5.6 Walls

In addition to the requirements of this section, structural plain concrete walls shall comply with the applicable requirements of ACI 318-08, Chapter 22.

3.5.5.6.1 Basement Walls

The thickness of exterior basement walls and foundation walls shall be not less than 7½ inches (191 mm). Structural plain concrete exterior basement walls shall be exempt from the requirements for special exposure conditions of Section 1904.2.2 (IBC 2006).

3.5.5.6.2 Other Walls

Except as provided for in Section 3.5.5.6.1, the thickness of bearing walls shall be not less than 1/24 the unsupported height or length, whichever is shorter, but not less than 5½ inches (140 mm).

3.5.5.6.3 Openings in Walls

Not less than two No. 5 bars shall be provided around window and door openings. Such bars shall extend at least 24 inches (610 mm) beyond the corners of openings.

3.5.6 Minimum Slab Provisions

3.5.6.1 General

The thickness of concrete floor slabs supported directly on the ground shall not be less than 3½ inches (89 mm). A 6-mil (0.006 inch; 0.15 mm) polyethylene vapour retarder with joints lapped not less than 6 inches (152 mm) shall be placed between the base course or subgrade and the concrete floor slab, or other approved equivalent methods or materials shall be used to retard vapour transmission through the floor slab.

EXCEPTION:

A vapour retarder is not required:

1. For detached structures accessory to occupancies in Group R-3 (permanent residential group), such as garages, utility buildings or other unheated facilities.
2. For unheated storage rooms having an area of less than 70 square feet (6.5 m^2) and carports attached to occupancies in Group R-3.
3. For buildings of other occupancies where migration of moisture through the slab from below will not be detrimental to the intended occupancy of the building.
4. For driveways, walks, patios and other flatwork which will not be enclosed at a later date.
5. Where approved based on local site conditions.

3.5.7 Anchorage to Concrete — Allowable Stress Design

3.5.7.1 Scope

The provisions of this section shall govern the allowable stress design of headed bolts and headed stud anchors cast in normal-weight concrete for purposes of transmitting structural loads from one connected element to the other. These provisions do not apply to anchors installed in hardened concrete or where load combinations include earthquake loads or effects. The bearing area of headed anchors shall be not less than one and one-half times the shank area. Where strength design is used, or where load combinations include earthquake loads or effects, the design strength of anchors shall be determined in accordance with Section 3.5.8. Bolts shall conform to ASTM A 307 or an approved equivalent.

3.5.7.2 Allowable Service Load

The allowable service load for headed anchors in shear or tension shall be as indicated in Table 3.5.1. Where anchors are subject to combined shear and tension, the following relationship shall be satisfied:

$$(P_s/P_t)^{\frac{5}{3}} + (V_s/V_t)^{\frac{5}{3}} \leq 1 \qquad \textit{Eq. (3.5.1)}$$

where:

P_s = Applied tension service load, pounds (N).
P_t = Allowable tension service load from Table 3.5.1, pounds (N).
V_s = Applied shear service load, pounds (N).
V_t = Allowable shear service load from Table 3.5.1, pounds (N).

TABLE 3.5.1 ALLOWABLE SERVICE LOAD ON EMBEDDED BOLTS (Pounds)

BOLT DIAMETER (inches)	MINIMUM EMBEDMENT (inches)	EDGE DISTANCE (inches)	SPACING (inches)	MINIMUM CONCRETE STRENGTH (psi)					
				f'c = 2,500		f'c = 3,000		f'c = 4,000	
				Tension	Shear	Tension	Shear	Tension	Shear
1/4	$2\frac{1}{2}$	$1\frac{1}{2}$	3	200	500	200	500	200	500
3/8	3	$2\frac{1}{4}$	$4\frac{1}{2}$	500	1,100	500	1,100	500	1,100
1/2	4	3	6	950	1,250	950	1,250	950	1,250
	4	5	6	1,450	1,600	1,500	1,650	1,550	1,750
5/8	$4\frac{1}{2}$	$3\frac{3}{4}$	$7\frac{1}{2}$	1,500	2,750	1,500	2,750	1,500	2,750
	$4\frac{1}{2}$	$6\frac{1}{4}$	$7\frac{1}{2}$	2,125	2,950	2,200	3,000	2,400	3,050
3/4	5	$4\frac{1}{2}$	9	2,250	3,250	2,250	3,560	2,250	3,560
	5	$7\frac{1}{2}$	9	2,825	4,275	2,950	4,300	3,200	4,400
7/8	6	$5\frac{1}{4}$	$10\frac{1}{2}$	2,550	3,700	2,550	4,050	2,550	4,050
1	7	6	12	3,050	4,125	3,250	4,500	3,650	5,300
11/8	8	$6\frac{3}{4}$	$13\frac{1}{2}$	3,400	4,750	3,400	4,750	3,400	4,750
11/4	9	$7\frac{1}{2}$	15	4,000	5,800	4,000	5,800	4,000	5,800

For SI: 1 inch = 25.4 mm, 1 pound per square inch = 0.00689 MPa, 1 pound = 4.45 N.

3.5.7.3 Required Edge Distance and Spacing

The allowable service loads in tension and shear specified in Table 3.5.1 are for the edge distance and spacing specified. The edge distance and spacing are permitted to be reduced to 50 percent of the values specified with an equal reduction in allowable service load. Where edge distance and spacing are reduced less than 50 percent, the allowable service load shall be determined by linear interpolation.

3.5.7.4 Increase for Special Inspection

Where special inspection is provided for the installation of anchors, a 100-percent increase in the allowable tension values of Table 3.5.1 is permitted. No increase in shear value is permitted.

3.5.8 Anchorage to Concrete- Strength Design

3.5.8.1 Scope

The provisions of this section shall govern the strength design of anchors installed in concrete for purposes of transmitting structural loads from one connected element to the other. Headed bolts, headed studs and hooked (J- or L-) bolts cast in concrete and expansion anchors and undercut anchors installed in hardened concrete shall be designed in accordance with Appendix D of ACI 318-08 as modified by Section 3.5.4.1.20, provided they are within the scope of Appendix D of ACI Code.

EXCEPTION:

Where the basic concrete breakout strength in tension of a single anchor, N_b, is determined in accordance with Equation (D-7), the concrete breakout strength requirements of Section D.4.2.2 of ACI Code shall be considered satisfied by the design procedures of Sections D.5.2 and D.6.2 of ACI Code for anchors exceeding 2 inches (51 mm) in diameter or 25 inches (635 mm) tensile embedment depth.

The strength design of anchors that are not within the scope of Appendix D of ACI 318-08, and as amended above, shall be in accordance with an approved procedure.

3.5.9 Shotcrete

3.5.9.1 General

Shotcrete is mortar or concrete that is pneumatically projected at high velocity onto a surface. Except as specified in this section, shotcrete shall conform to the requirements of this section for plain or reinforced concrete.

3.5.9.2 Proportions and Materials

Shotcrete proportions shall be selected that allow suitable placement procedures using the delivery equipment selected and shall result in finished in-place hardened shotcrete meeting the strength requirements of this code.

3.5.9.3 Aggregate

Coarse aggregate, if used, shall not exceed 3/4 inch (19.1 mm).

3.5.9.4 Reinforcement

Reinforcement used in shotcrete construction shall comply with the provisions of Sections 3.5.9.4.1 through 3.5.9.4.4.

3.5.9.4.1 Size

The maximum size of reinforcement shall be No. 5 bars unless it is demonstrated by preconstruction tests that adequate encasement of larger bars will be achieved.

3.5.9.4.2 Clearance

When No. 5 or smaller bars are used, there shall be a minimum clearance between parallel reinforcement bars of 2½ inches (64 mm). When bars larger than No. 5 are permitted, there shall be a minimum clearance between parallel bars equal to six diameters of the bars used. When two curtains of steel are provided, the curtain nearer the nozzle shall have a minimum spacing equal to 12 bar diameters and the remaining curtain shall have a minimum spacing of six bar diameters.

EXCEPTION:

Subject to the approval of the building official, required clearances shall be reduced where it is demonstrated by preconstruction tests that adequate encasement of the bars used in the design will be achieved.

3.5.9.4.3 Splices

Lap splices of reinforcing bars shall utilize the noncontact lap splice method with a minimum clearance of 2 inches (51 mm) between bars. The use of contact lap splices necessary for support of the reinforcing is permitted when approved by the building official, based on satisfactory preconstruction tests that show that adequate encasement of the bars will be achieved, and provided that the splice is oriented so that a plane through the centre of the spliced bars is perpendicular to the surface of the shotcrete.

3.5.9.4.4 Spirally Tied Columns

Shotcrete shall not be applied to spirally tied columns.

3.5.9.5 Preconstruction Tests

When required by the building official, a test panel shall be shot, cured, cored or sawn, examined and tested prior to commencement of the project. The sample panel shall be representative of the project and simulate job conditions as closely as possible. The panel thickness and reinforcing shall reproduce the thickest and most congested area specified in the structural design. It shall be shot at the same angle, using the same nozzleman and with the same concrete mix design that will be used on the project. The equipment used in preconstruction testing shall be the same equipment used in the work requiring such testing, unless substitute equipment is approved by the building official.

3.5.9.6 Rebound

Any rebound or accumulated loose aggregate shall be removed from the surfaces to be covered prior to placing the initial or any succeeding layers of shotcrete. Rebound shall not be used as aggregate.

3.5.9.7 Joints

Except where permitted herein, unfinished work shall not be allowed to stand for more than 30 minutes unless edges are sloped to a thin edge. For structural elements that will be under compression and for construction joints shown on the approved construction documents, square joints are permitted. Before placing additional material adjacent to previously applied work, sloping and square edges shall be cleaned and wetted.

3.5.9.8 Damage

In-place shotcrete that exhibits sags, sloughs, segregation, honeycombing, sand pockets or other obvious defects shall be removed and replaced. Shotcrete above sags and sloughs shall be removed and replaced while still plastic.

3.5.9.9 Curing

During the curing periods specified herein, shotcrete shall be maintained above 40°F (4°C) and in moist condition.

3.5.9.9.1 Initial Curing

Shotcrete shall be kept continuously moist for 24 hours after shotcreting is complete or shall be sealed with an approved curing compound.

3.5.9.9.2 Final Curing

Final curing shall continue for seven days after shotcreting, or for three days if high- early-strength cement is used, or until the specified strength is obtained. Final curing shall consist of the initial curing process or the shotcrete shall be covered with an approved moisture-retaining cover.

3.5.9.9.3 Natural Curing

Natural curing shall not be used in lieu of that specified in this section unless the relative humidity remains at or above 85 percent, and is authorized by the registered design professional and approved by the building official.

3.5.9.10 Strength Tests

Strength tests for shotcrete shall be made by an approved agency on specimens that are representative of the work and which have been water soaked for at least 24 hours prior to testing. When the maximum-size aggregate is larger than 3/8 inch (9.5 mm), specimens shall consist of not less than three 3-inch-diameter (76 mm) cores or 3-inch (76 mm) cubes. When the maximum-size aggregate is 3/8 inch (9.5 mm) or smaller, specimens shall consist of not less than 2-inch-diameter (51 mm) cores or 2-inch (51 mm) cubes.

3.5.9.10.1 Sampling

Specimens shall be taken from the in-place work or from test panels, and shall be taken at least once each shift, but not less than one for each 50 cubic yards (38.2 m^3) of shotcrete.

3.5.9.10.2 Panel Criteria

When the maximum-size aggregate is larger than 3/8 inch (9.5 mm), the test panels shall have minimum dimensions of 18 inches by 18 inches (457 mm by 457 mm). When the maximum size aggregate is 3/8 inch (9.5 mm) or smaller, the test panels shall have minimum dimensions of 12 inches by 12 inches (305 mm by 305 mm). Panels shall be shot in the same position as the work, during the course of the work and by the nozzlemen doing the work. The conditions under which the panels are cured shall be the same as the work.

3.5.9.10.3 Acceptance Criteria

The average compressive strength of three cores from the in-place work or a single test panel shall equal or exceed 0.85 f_c' with no single core less than 0.75 f_c' . The average compressive strength of three cubes taken from the in-place work or a single test panel shall equal or exceed f_c' with no individual cube less than 0.88 f_c' . To check accuracy, locations represented by erratic core or cube strengths shall be retested.

3.5.10 Concrete- Filled Pipe Columns

3.5.10.1 General

Concrete-filled pipe columns shall be manufactured from standard, extra-strong or double-extra-strong steel pipe or tubing that is filled with concrete so placed and manipulated as to secure maximum density and to ensure complete filling of the pipe without voids.

3.5.10.2 Design

The safe supporting capacity of concrete-filled pipe columns shall be computed in accordance with the approved rules or as determined by a test.

3.5.10.3 Connections

Caps, base plates and connections shall be of approved types and shall be positively attached to the shell and anchored to the concrete core. Welding of brackets without mechanical anchorage shall be prohibited. Where the pipe is slotted to accommodate webs of brackets or other connections, the integrity of the shell shall be restored by welding to ensure hooping action of the composite section.

3.5.10.4 Reinforcement

To increase the safe load-supporting capacity of concrete-filled pipe columns, the steel reinforcement shall be in the form of rods, structural shapes or pipe embedded in the concrete core with sufficient clearance to ensure the composite action of the section, but not nearer than

1 inch (25 mm) to the exterior steel shell. Structural shapes used as reinforcement shall be milled to ensure bearing on cap and base plates.

3.5.10.5 Fire-Resistance-Rating Protection

Pipe columns shall be of such size or so protected as to develop the required fire-resistance ratings specified in this Code. Where an outer steel shell is used to enclose the fire-resistant covering, the shell shall not be included in the calculations for strength of the column section. The minimum diameter of pipe columns shall be 4 inches (102 mm) except that in structures of Type V construction not exceeding three storeys or 40 feet (12192 mm) in height, pipe columns used in the basement and as secondary steel members shall have a minimum diameter of 3 inches (76 mm).

3.5.10.6 Approvals

Details of column connections and splices shall be shop fabricated by approved methods and shall be approved only after tests in accordance with the approved rules. Shop-fabricated concrete-filled pipe columns shall be inspected by the building official or by an approved representative of the manufacturer at the plant.

SECTION 3.6
STEEL

3.6.1 General

3.6.1.1 Scope

The provisions of this section govern the quality, design, fabrication and erection of steel used structurally in buildings.

3.6.2 Definitions

The following words and terms shall, for the purposes of this chapter and as used elsewhere in this code, have the meaning shown herein

STEEL CONSTRUCTION, COLD-FORMED. That type of construction made up entirely or in part of steel structural members cold formed to shape from sheet or strip steel such as roof deck, floor and wall panels, studs, floor joists, roof joists and other structural elements.

STEEL JOIST. Any steel structural member of a building or structure made of hot-rolled or cold-formed solid or open-web sections, or riveted or welded bars, strip or sheet steel members, or slotted and expanded, or otherwise deformed rolled sections.

STEEL MEMBER, STRUCTURAL. Any steel structural member of a building or structure consisting of a rolled steel structural shape other than cold-formed steel, or steel joist members

3.6.3 Identification and Protection of Steel for Structural Purposes

3.6.3.1 Identification

Steel furnished for structural load-carrying purposes shall be properly identified for conformity to the ordered grade in accordance with the specified ASTM standard or other specification and the provisions of this section. Steel that is not readily identifiable as to grade from marking and test records shall be tested to determine conformity to such standards.

3.6.3.2 Protection

Painting of structural steel shall comply with the requirements contained in AISC 360. Individual structural members and assembled panels of cold-formed steel construction, except where fabricated of approved corrosion-resistant steel or of steel having a corrosion-resistant or other approved coating, shall be protected against corrosion with an approved coat of paint, enamel or other approved protection.

3.6.4 Connections

3.6.4.1 Welding

The details of design, workmanship and technique for welding, inspection of welding and qualification of welding operators shall conform to the requirements of the specifications listed in Sections 3.6.5, 3.6.6, 3.6.7, 3.6.9 and 3.6.10. Special inspection of welding shall be provided where required by the authority having jurisdiction.

3.6.4.2 Bolting

The design, installation and inspection of bolts shall be in accordance with the requirements of the specifications listed in Sections 3.6.5, 3.6.6, 3.6.7, 3.6.9 and 3.6.10. Special inspection of the installation of high-strength bolts shall be provided where required by the authority having jurisdiction.

3.6.4.2.1 Anchor rods

Anchor rods shall be set accurately to the pattern and dimensions called for on the plans. The protrusion of the threaded ends through the connected material shall be sufficient to fully engage the threads of the nuts, but shall not be greater than the length of the threads on the bolts.

3.6.5 Structural Steel–Design

3.6.5.1 General

The design of structural steel for buildings and structures shall be in accordance with AISC 360-05. Where required, the seismic design of steel structures shall be in accordance with the additional provisions of Section 3.6.5.2.

3.6.5.2 Seismic Requirements for Steel Structures

The design of structural steel structures to resist seismic forces shall be in accordance with the provisions of Section 3.6.5.2.1 or 3.6.5.2.2 for the appropriate Seismic Design Category.

3.6.5.2.1 Seismic Design Category A, B or C

Structural steel structures assigned to Seismic Design Category A, B or C shall be of any construction permitted in Section 3.6.5. An R factor as set forth in Section 12.2.1 of ASCE 7-05 for the appropriate steel system is permitted where the structure is designed and detailed in accordance with the provisions of AISC 341, Part I. Systems not detailed in accordance with the above shall use the R factor in Section 12.2.1 of ASCE 7-05 designated for “structural steel systems not specifically detailed for seismic resistance.”

3.6.5.2.2 Seismic Design Category D, E or F

Structural steel structures assigned to Seismic Design Category D, E or F shall be designed and detailed in accordance with AISC 341, Part I.

3.6.5.3 Seismic Requirements for Composite Construction

The design, construction and quality of composite steel and concrete components that resist seismic forces shall conform to the requirements of the AISC 360-05 and ACI 318-08. An R factor as set forth in Section 12.2.1 of ASCE 7-05 for the appropriate composite steel and concrete system is permitted where the structure is designed and detailed in accordance with the provisions of AISC 341, Part II. In Seismic Design Category B or above, the design of such systems shall conform to the requirements of AISC 341, Part II.

3.6.5.3.1 Seismic Design Categories D, E and F

Composite structures are permitted in Seismic Design Categories D, E and F, subject to the limitations in Section 12.2.1 of ASCE 7-05, where substantiating evidence is provided to demonstrate that the proposed system will perform as intended by AISC 341, Part II. The substantiating evidence shall be subject to building official approval. Where composite elements or connections are required to sustain inelastic deformations, the substantiating evidence shall be based on cyclic testing.

3.6.6 Steel Joists

3.6.6.1 General

The design, manufacture and use of open web steel joists and joist girders shall be in accordance with one of the following Steel Joist Institute (SJI) specifications:

1. SJI K-1.1
2. SJI LH/DLH-1.1
3. SJI JG-1.1

Where required, the seismic design of buildings shall be in accordance with the additional provisions of Section 6.5.2 or 6.10.5.

3.6.6.2 Design

The registered design professional shall indicate on the construction documents the steel joist and/or steel joist girder designations from the specifications listed in Section 3.6.6.1 and shall indicate the requirements for joist and joist girder design, layout, end supports, anchorage, non-SJI standard bridging, bridging termination connections and bearing connection design to resist uplift and lateral loads. These documents shall indicate special requirements as follows:

1. Special loads including:
   - 1.1. Concentrated loads;
   - 1.2. Non-uniform loads;
   - 1.3. Net uplift loads;
   - 1.4. Axial loads;
   - 1.5. End moments; and
   - 1.6. Connection forces.
2. Special considerations including:
   - 2.1 Profiles for nonstandard joist and joist girder configurations (standard joist and joist girder configurations are as indicated in the SJI catalog);
   - 2.2 Oversized or other nonstandard web openings; and
   - 2.3 Extended ends.
3. Deflection criteria for live and total loads for non-SJI standard joists.

3.6.6.3 Calculations

The steel joist and joist girder manufacturer shall design the steel joists and/or steel joist girders in accordance with the current SJI specifications and load tables to support the load requirements of Section 3.6.6.2. The registered design professional may require submission of the steel joist and joist girder calculations as prepared by a registered design professional responsible for the product design. If requested by the registered design professional, the steel joist manufacturer shall submit design calculations with a cover letter bearing the seal and signature of the joist manufacturer's registered design professional. In addition to standard calculations under this seal and signature, submittal of the following shall be included:

1. Non-SJI standard bridging details (e.g. for cantilevered conditions, net uplift, etc.).
2. Connection details for**:**
   - 2.1 Non-SJI standard connections (e.g. flush-framed or framed connections);
   - 2.2 Field splices; and
   - 2.3 Joist headers.

3.6.6.4 Steel Joist Drawings

Steel joist placement plans shall be provided to show the steel joist products as specified on the construction documents and are to be utilized for field installation in accordance with specific project requirements as stated in Section 3.6.6.2. Steel placement plans shall include, at a minimum, the following:

1. Listing of all applicable loads as stated in Section 3.6.6.2 and used in the design of the steel joists and joist girders as specified in the construction documents.
2. Profiles for nonstandard joist and joist girder configurations (standard joist and joist girder configurations are as indicated in the SJI catalog).
3. Connection requirements for:

   3.1. Joist supports;
   3.2. Joist girder supports;
   3.3. Field splices; and
   3.4. Bridging attachments.

4. Deflection criteria for live and total loads for non-SJI standard joists.
5. Size, location and connections for all bridging.
6. Joist headers.

Steel joist placement plans do not require the seal and signature of the joist manufacturer's registered design professional.

3.6.6.5 Certification

At completion of fabrication, the steel joist manufacturer shall submit a certificate of compliance in accordance with Section 1704.2.2 stating that work was performed in accordance with approved construction documents and with SJI standard specifications.

3.6.7 Steel Cable Structures

3.6.7.1 General

The design, fabrication and erection including related connections, and protective coatings of steel cables for buildings shall be in accordance with ASCE 19.

3.6.7.2 Seismic Requirements for Steel Cable

The design strength of steel cables shall be determined by the provisions of ASCE 19 except as modified by these provisions.

1. A load factor of 1.1 shall be applied to the prestress force included in T3 and T4 as defined in Section 3.12.
2. In Section 3.2.1, Item (c) shall be replaced with "1.5 T3" and Item (d) shall be replaced with "1.5 T4."

3.6.8 Steel Storage Racks

3.6.8.1 Storage Racks

The design, testing and utilization of industrial steel storage racks shall be in accordance with the RMI Specification for the Design, Testing and Utilization of Industrial Steel Storage Racks. Racks in the scope of this specification include industrial pallet racks, movable shelf racks and stacker racks and does not apply to other types of racks, such as drive-in and drive-through racks, cantilever racks, portable racks or rack buildings. Where required, the seismic design of storage racks shall be in accordance with the provisions of Section 15.5.3 of ASCE 7.

3.6.9 Cold-Formed Steel

3.6.9.1 General

The design of cold-formed carbon and low-alloy steel structural members shall be in accordance with AISI-NAS. The design of cold-formed stainless-steel structural members shall be in accordance with ASCE 8. Cold-formed steel light-framed construction shall comply with Section 3.6.10.

3.6.9.2 Composite Slabs on Steel Decks

Composite slabs of concrete and steel deck shall be designed and constructed in accordance with ASCE 3.

3.6.10 Cold-Formed Steel, Light-Framed Construction

3.6.10.1 General

The design, installation and construction of cold-formed carbon or low-alloy steel, structural and nonstructural steel framing shall be in accordance with AISI-General and AISI-NAS.

3.6.10.2 Headers

The design and installation of cold-formed steel box headers, back-to-back headers and single and double L-headers used in single-span conditions for load-carrying purposes shall be in accordance with AISI-Header, subject to the limitations therein.

3.6.10.3 Trusses

The design, quality assurance, installation and testing of cold-formed steel trusses shall be in accordance with AISI-Truss, subject to the limitations therein.

3.6.10.4 Wall Stud Design

The design and installation of cold-formed steel studs for structural and nonstructural walls shall be in accordance with AISI-WSD.

3.6.10.5 Lateral Design

The design of light-framed cold-formed steel walls and diaphragms to resist wind and seismic loads shall be in accordance with AISI-Lateral.

3.6.10.6 Prescriptive Framing

Detached one- and two-family dwellings and townhouses, up to two storeys in height, shall be permitted to be constructed in accordance with AISI-PM, subject to the limitations therein.

Appendix A

Special Provision for Ground Floor Columns in High Seismic Areas

General/Intention

The Mandalay Earthquake, with a magnitude of 7.7 on March 28, 2025, caused notable damage to numerous ground-floor columns. The failure pattern involved the formation of consecutive plastic hinges at both ends of the ground floor columns, leading to a complete structural collapse.

Therefore, before the detailed studies and recommendations arrive, the ground floor column redundancy factor will be applied to strengthen the ground floor columns in high seismic areas. This provision will be reviewed and revised as comprehensive recommendations are coming out after the detailed investigations.

Applicability

This special provision applies to all ground floor columns regardless of the story height in the highly seismic areas where S_{DS} greater 0.5g and S_{D1} greater than0.2g.

Redundancy Factor for Ground Floor Columns

A redundancy factor of 1.3 must be applied to all ground floor columns in highly seismic areas, regardless of the story height, unless load combinations with an overstrength factor as specified in Section 3.4.4.3.2 are used.

Seismic Detailing Requirement

Intermediate seismic detailing is required as a minimum for all ground floor columns and the connecting first-floor beams, walls, and frames.

Appendix B

Special Provisions for Low-Risk, Low-Rise Buildings

General/Intention

Low-risk, low-rise buildings constitute a significant portion of Myanmar's building stock, accounting for over 95% of the total. Due to economic factors, it is challenging to fully implement the national building code for these types of buildings. Additionally, since Myanmar is situated in a highly seismic region, seismic safety remains a major concern. Instead of completely ignoring the building code in the construction of low-rise buildings, it is practical to reduce the seismic force by limiting the probability of exceedance in certain years / under some conditions.

Applicability

This special provision applies to regular structures that are five storeys or less in height, belong to occupancy category II, and have a period, T, of 0.5 seconds or less with a shorter functional lifespan.

Reduction of Seismic Force

The Seismic Load Effect in article 3.4.4.2 can be reduced as follows:

- For $S_s < 1.5g$, E can be reduced by a factor of 0.88.
- For $S_s \geq 1.5g$, E can be reduced by a factor of 0.86.

Seismic Detailing Requirement

Although the seismic forces are reduced, the intermediate seismic detailing is the minimum requirement for all those building categories.

Appendix:
Wind: Method 2 (ASCE 7-05)

Wind: Method 2 (ASCE 7-05)

Main Wind Force Res. Sys. / Comp and Clad. – Method 2		All Heights
Figure 6-5	Internal Pressure Coefficient, GC_{pi}	Walls & Roofs
Enclosed, Partially Enclosed, and Open Buildings		

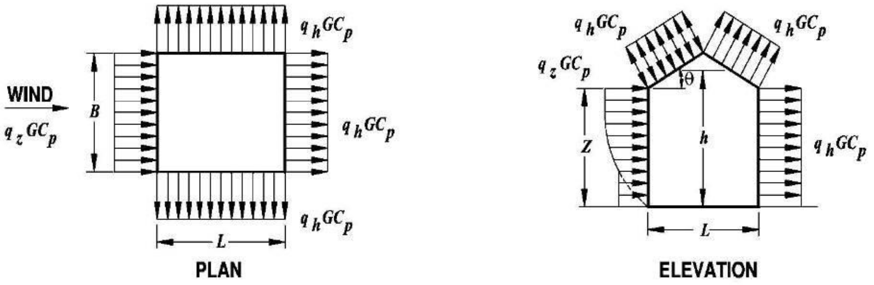
Enclosure Classification	GC_{pi}
Open Buildings	0.00
Partially Enclosed Buildings	+0.55 -0.55
Enclosed Buildings	+0.18 -0.18

Notes:

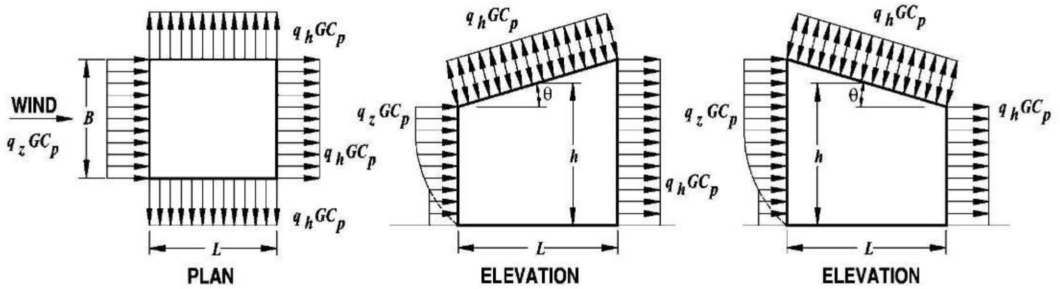
1. Plus and minus signs signify pressures acting toward and away from the internal surfaces, respectively.
2. Values of GC_{pi} shall be used with q_z or q_h as specified in 6.5.12.
3. Two cases shall be considered to determine the critical load requirements for the appropriate condition:

   (i) a positive value of GC_{pi} applied to all internal surfaces
   (ii) a negative value of GC_{pi} applied to all internal surfaces

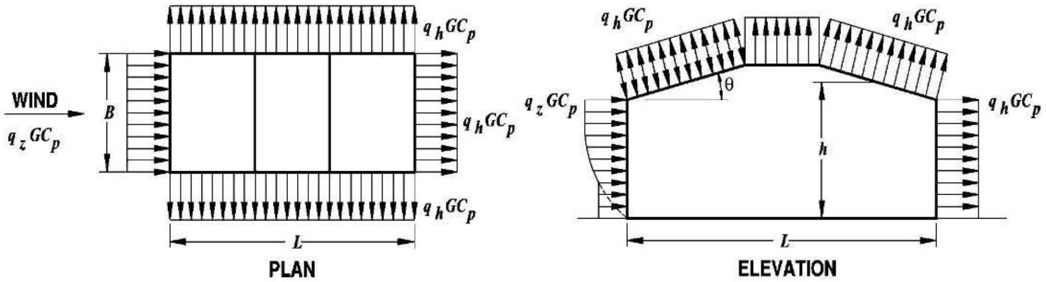
Main Wind Force Resisting System – Method 2		**All Heights**
Figure 6-6	**External Pressure Coefficients, C_p**	**Walls & Roofs**
Enclosed, Partially Enclosed Buildings		



GABLE, HIP ROOF



MONOSLOPE ROOF (NOTE 4)



MANSARD ROOF (NOTE 8)

Main Wind Force Resisting System – Method 2		All Heights
Figure 6-6 (con't)	External Pressure Coefficients, C_p	Walls & Roofs
Enclosed, Partially Enclosed Buildings		

Wall Pressure Coefficients, C_p

Surface	L/B	C_p	Use With
Windward Wall	All values	0.8	q_z
Leeward Wall	0-1	-0.5	q_h
	2	-0.3	
	≥4	-0.2	
Side Wall	All values	-0.7	q_h

Roof Pressure Coefficients, C_p, for use with q_h

Wind Direction	Windward									Leeward		
	Angle, θ (degrees)									Angle, θ (degrees)		
	h/L	10	15	20	25	30	35	45	≥60#	10	15	≥20
Normal to ridge for θ ≥ 10°	≤0.25	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.3	-0.2 0.3	0.0* 0.4	0.4	0.01 θ	-0.3	-0.5	-0.6
	0.5	-0.9 -0.18	-0.7 -0.18	-0.4 0.0*	-0.3 0.2	-0.2 0.2	-0.2 0.3	0.0* 0.4	0.01 θ	-0.5	-0.5	-0.6
	≥1.0	-1.3** -0.18	-1.0 -0.18	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.2	0.0* 0.3	0.01 θ	-0.7	-0.6	-0.6

Wind Direction	h/L	Horiz distance from windward edge	C_p
Normal to ridge for θ < 10 and Parallel to ridge for all θ	≤ 0.5	0 to h/2	-0.9, -0.18
		h/2 to h	-0.9, -0.18
		h to 2 h	-0.5, -0.18
		> 2h	-0.3, -0.18
	≥ 1.0	0 to h/2	-1.3**, -0.18
		> h/2	-0.7, -0.18

*Value is provided for interpolation purposes.

**Value can be reduced linearly with area over which it is applicable as follows

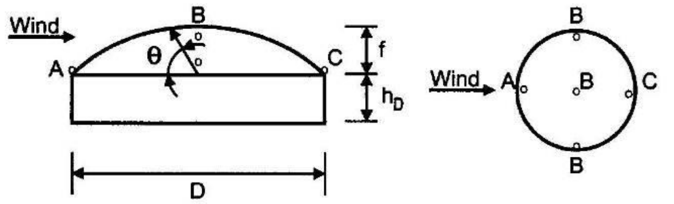
Area (sq ft)	Reduction Factor
≤ 100 (9.3 sq m)	1.0
200 (23.2 sq m)	0.9
≥ 1000 (92.9 sq m)	0.8

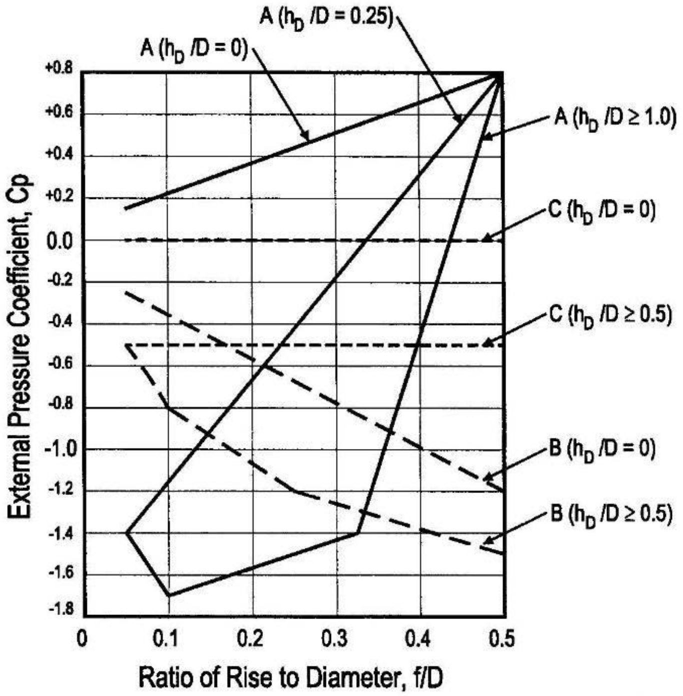
Notes:

1. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
2. Linear interpolation is permitted for values of *L/B*, *h/L* and θ other than shown. Interpolation shall only be carried out between values of the same sign. Where no value of the same sign is given, assume 0.0 for interpolation purposes.
3. Where two values of C_p are listed, this indicates that the windward roof slope is subjected to either positive or negative pressures and the roof structure shall be designed for both conditions. Interpolation for intermediate ratios of h/L in this case shall only be carried out between C_p values of like sign.
4. For monoslope roofs, entire roof surface is either a windward or leeward surface.
5. For flexible buildings use appropriate G_f as determined by Section 6.5.8.
6. Refer to Figure 6-7 for domes and Figure 6-8 for arched roofs.
7. Notation:
   B: Horizontal dimension of building, in feet (meter), measured normal to wind direction.
   L: Horizontal dimension of building, in feet (meter), measured parallel to wind direction.
   h: Mean roof height in feet (meters), except that eave height shall be used for θ ≤ 10 degrees.
   z: Height above ground, in feet (meters).
   G: Gust effect factor.
   q_z, q_h: Velocity pressure, in pounds per square foot (N/m^2), evaluated at respective height.
   θ: Angle of plane of roof from horizontal, in degrees.
8. For mansard roofs, the top horizontal surface and leeward inclined surface shall be treated as leeward surfaces from the table.
9. Except for MWFRS's at the roof consisting of moment resisting frames, the total horizontal shear shall not be less than that determined by neglecting wind forces on roof surfaces.

#For roof slopes greater than 80°, use C_p = 0.8

Main Wind Force Resisting System – Method 2		All Heights
Figure 6-7	**External Pressure Coefficients, C_p**	**Domed Roofs**
Enclosed, Partially Enclosed Buildings and Structures		





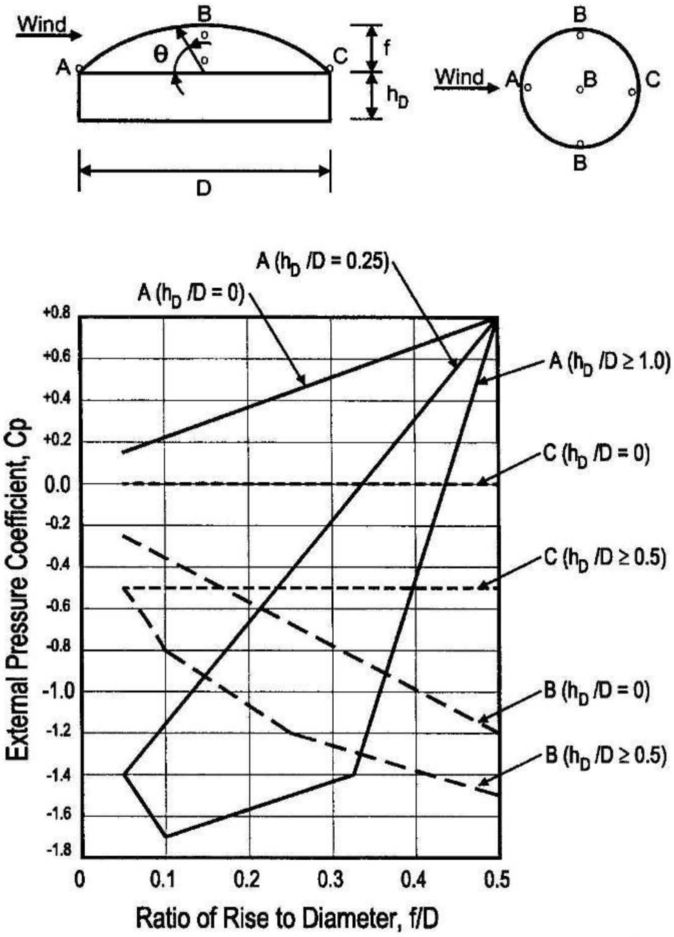
External Pressure Coefficients for Domes with a Circular Base.

(Adapted from Eurocode, 1995)

Notes:

1. Two load cases shall be considered:
   - Case A. C_p values between A and B and between B and C shall be determined by linear interpolation along arcs on the dome parallel to the wind direction;
   - Case B. C_p shall be the constant value of A for $\theta \leq 25$ degrees, and shall be determined by linear interpolation from 25 degrees to B and from B to C.
2. Values denote C_p to be used with $q_{(h_D+f)}$ where $h_D + f$ is the height at the top of the dome.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. C_p is constant on the dome surface for arcs of circles perpendicular to the wind direction; for example, the arc passing through B-B-B and all arcs parallel to B-B-B.
5. For values of h_D/D between those listed on the graph curves, linear interpolation shall be permitted.
6. $\theta = 0$ degrees on dome springline, $\theta = 90$ degrees at dome center top point. f is measured from springline to top.
7. The total horizontal shear shall not be less than that determined by neglecting wind forces on roof surfaces.
8. For f/D values less than 0.05, use Figure 6-6.

Main Wind Force Resisting System – Method 2		All Heights
Figure 6-7	**External Pressure Coefficients, C_p**	**Domed Roofs**
Enclosed, Partially Enclosed Buildings and Structures		



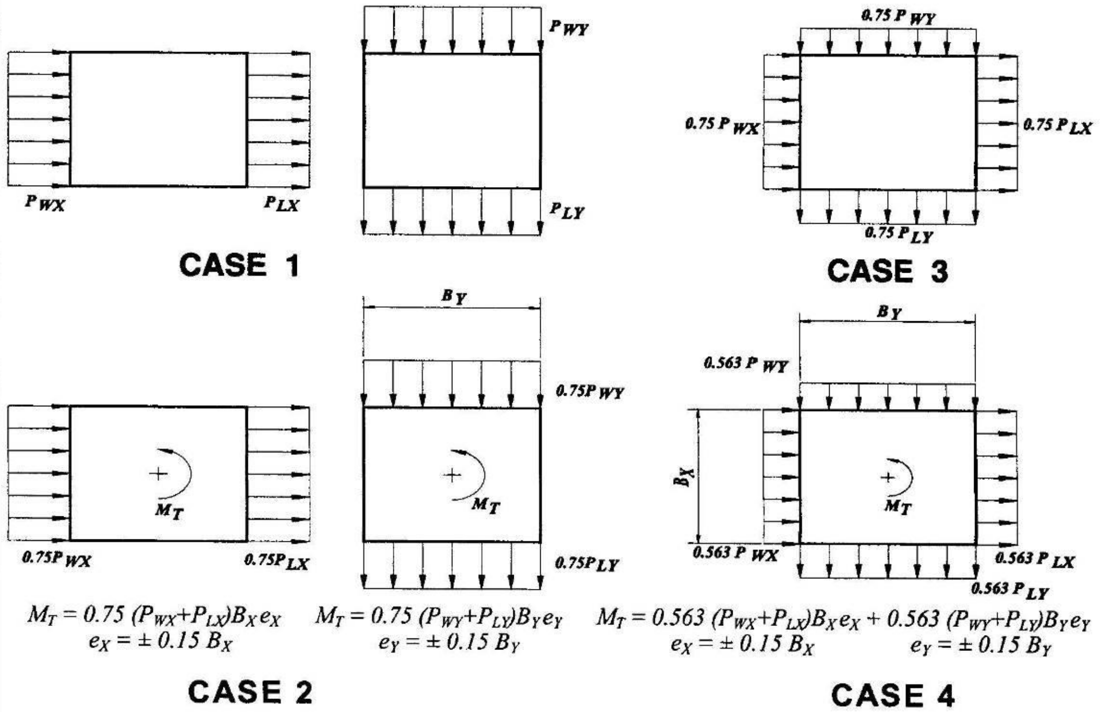
External Pressure Coefficients for Domes with a Circular Base.

(Adapted from Eurocode, 1995)

Notes:

1. Two load cases shall be considered:
   Case A. C_p values between A and B and between B and C shall be determined by linear interpolation along arcs on the dome parallel to the wind direction;
   Case B. C_p shall be the constant value of A for $\theta \leq 25$ degrees, and shall be determined by linear interpolation from 25 degrees to B and from B to C.
2. Values denote C_p to be used with $q_{(h_D+f)}$ where $h_D + f$ is the height at the top of the dome.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. C_p is constant on the dome surface for arcs of circles perpendicular to the wind direction; for example, the arc passing through B-B-B and all arcs parallel to B-B-B.
5. For values of h_D/D between those listed on the graph curves, linear interpolation shall be permitted.
6. $\theta = 0$ degrees on dome springline, $\theta = 90$ degrees at dome center top point. f is measured from springline to top.
7. The total horizontal shear shall not be less than that determined by neglecting wind forces on roof surfaces.
8. For f/D values less than 0.05, use Figure 6-6.

Main Wind Force Resisting System–Method 2		All Heights
Figure 6-9	**Design Wind Load Cases**	



Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

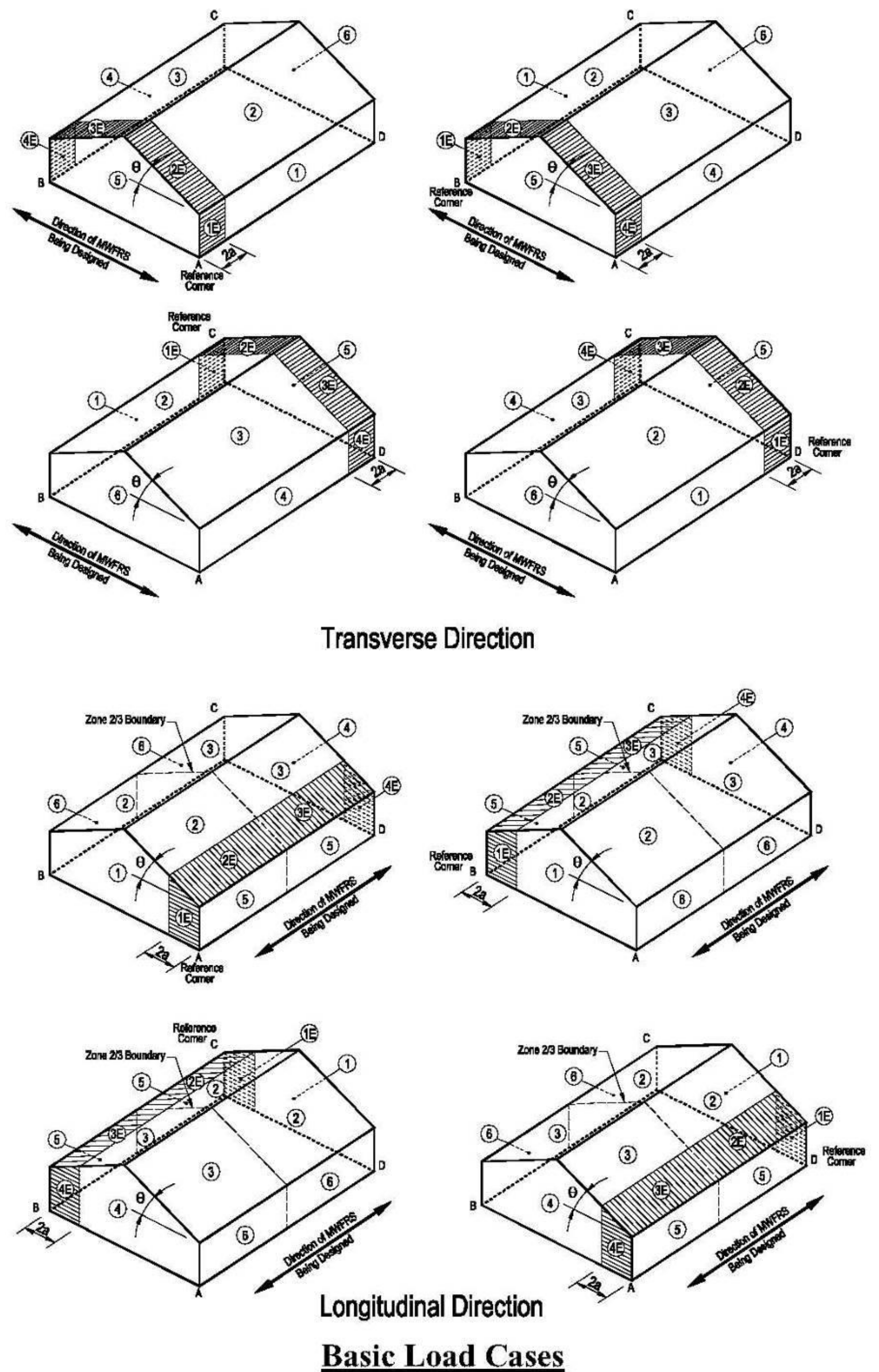
Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 6.5.12.2.1 and 6.5.12.2.3 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:

   P_{WX}, P_{WY}: Windward face design pressure acting in the x, y principal axis, respectively.
   P_{LX}, P_{LY}: Leeward face design pressure acting in the x, y principal axis, respectively.
   e (e_X, e_Y) : Eccentricity for the x, y principal axis of the structure, respectively.
   M_T: Torsional moment per unit height acting about a vertical axis of the building.

Main Wind Force Resisting System – Method 2		$h \leq 60$ ft.
Figure 6-10	External Pressure Coefficients, GC_{pf}	Low-rise Walls & Roofs
Enclosed, Partially Enclosed Buildings		



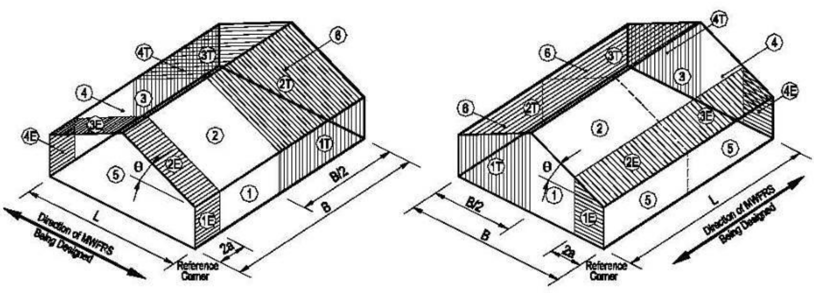
Basic Load Cases

Main Wind Force Resisting System – Method 2		$h \leq 60$ ft.
Figure 6-10 (cont'd)	External Pressure Coefficients, GC_{pf}	**Low-rise Walls & Roofs**
Enclosed, Partially Enclosed Buildings		

Roof Angle θ (degrees)	Building Surface									
	1	2	3	4	5	6	1E	2E	3E	4E
0-5	0.40	-0.69	-0.37	-0.29	-0.45	-0.45	0.61	-1.07	-0.53	-0.43
20	0.53	-0.69	-0.48	-0.43	-0.45	-0.45	0.80	-1.07	-0.69	-0.64
30-45	0.56	0.21	-0.43	-0.37	-0.45	-0.45	0.69	0.27	-0.53	-0.48
90	0.56	0.56	-0.37	-0.37	-0.45	-0.45	0.69	0.69	-0.48	-0.48

Notes:

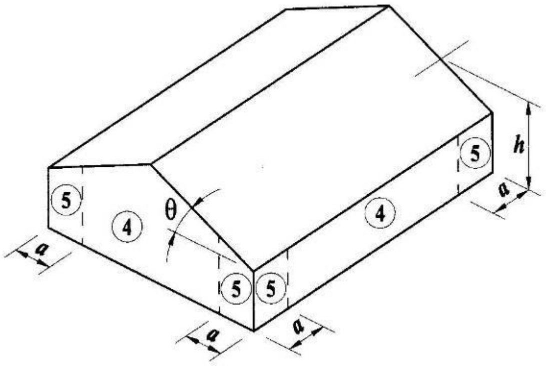
1. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
2. For values of θ other than those shown, linear interpolation is permitted.
3. The building must be designed for all wind directions using the 8 loading patterns shown. The load patterns are applied to each building corner in turn as the Reference Corner.
4. Combinations of external and internal pressures (see Figure 6-5) shall be evaluated as required to obtain the most severe loadings.
5. For the torsional load cases shown below, the pressures in zones designated with a "T" (1T, 2T, 3T, 4T) shall be 25% of the full design wind pressures (zones 1, 2, 3, 4).
   Exception: One story buildings with h less than or equal to 30 ft (9.1m), buildings two stories or less framed with light frame construction, and buildings two stories or less designed with flexible diaphragms need not be designed for the torsional load cases.
   Torsional loading shall apply to all eight basic load patterns using the figures below applied at each reference corner.
6. Except for moment-resisting frames, the total horizontal shear shall not be less than that determined by neglecting wind forces on roof surfaces.
7. For the design of the MWFRS providing lateral resistance in a direction parallel to a ridge line or for flat roofs, use θ = 0° and locate the zone 2/3 boundary at the mid-length of the building.
8. The roof pressure coefficient GC_{pf}, when negative in Zone 2 or 2E, shall be applied in Zone 2/2E for a distance from the edge of roof equal to 0.5 times the horizontal dimension of the building parallel to the direction of the MWFRS being designed or 2.5 times the eave height, h_e, at the windward wall, whichever is less; the remainder of Zone 2/2E extending to the ridge line shall use the pressure coefficient GC_{pf} for Zone 3/3E.
9. Notation:
   a: 10 percent of least horizontal dimension or 0.4h, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m).
   h: Mean roof height, in feet (meters), except that eave height shall be used for θ ≤ 10°.
   θ: Angle of plane of roof from horizontal, in degrees.

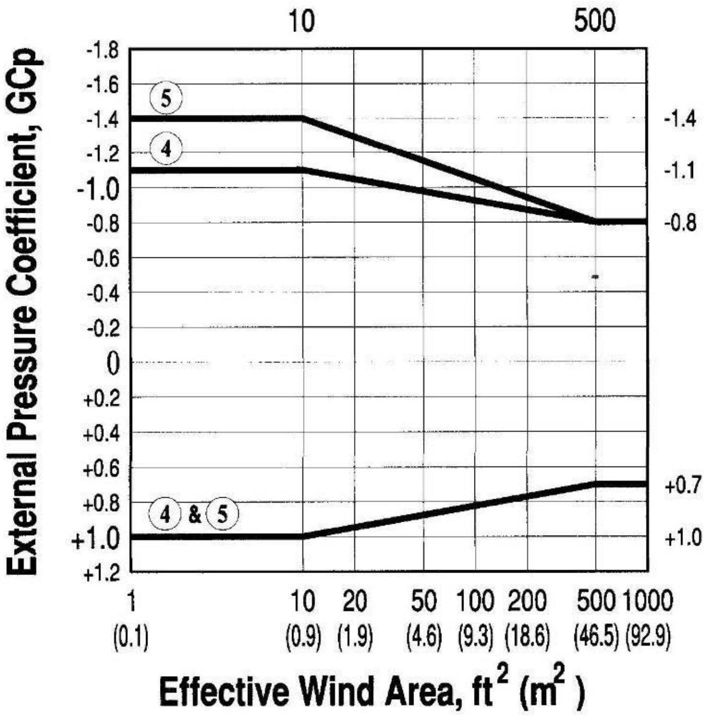


Transverse Direction Longitudinal Direction

Torsional Load Cases

Components and Cladding – Method 2		h ≤ 60 ft.
Figure 6-11A	External Pressure Coefficients, GC_p	Walls
Enclosed, Partially Enclosed Buildings		

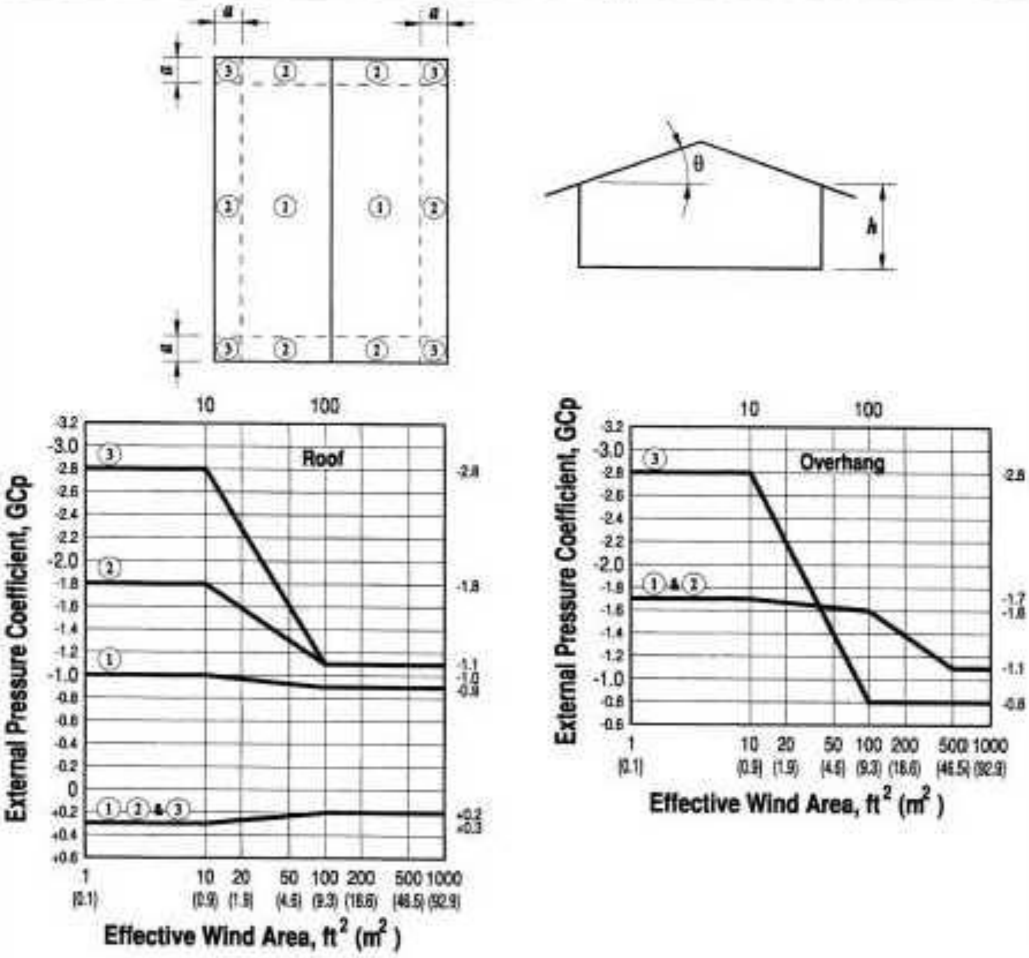




Notes:

1. Vertical scale denotes GC_p to be used with q_h.
2. Horizontal scale denotes effective wind area, in square feet (square meters).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of GC_p for walls shall be reduced by 10% when $\theta \leq 10°$.
6. Notation:
   - a: 10 percent of least horizontal dimension or 0.4h, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m).
   - h: Mean roof height, in feet (meters), except that eave height shall be used for $\theta \leq 10°$.
   - θ: Angle of plane of roof from horizontal, in degrees.

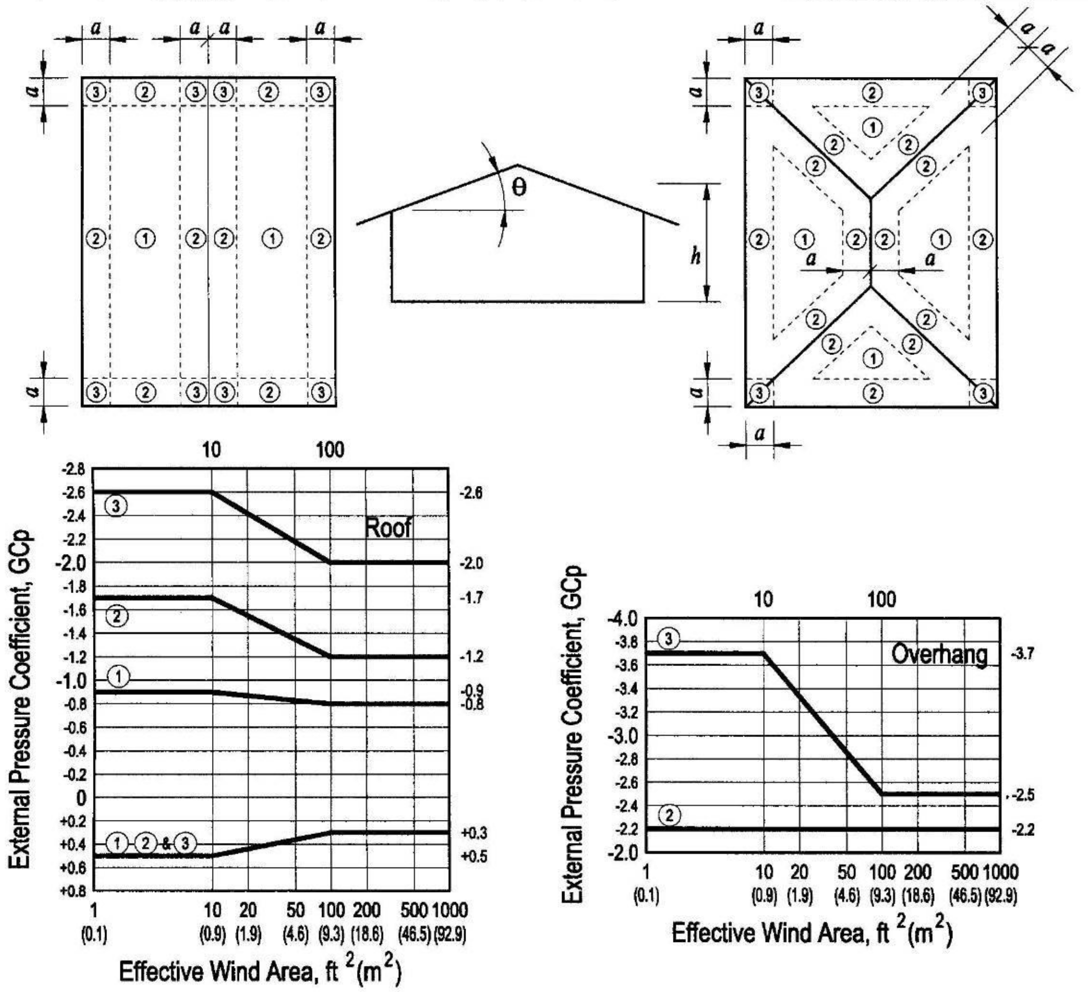
Components and Cladding – Method 2		h ≤ 60 ft.
Figure 6-11B	External Pressure Coefficients, GC_p	**Gable Roofs θ ≤ 7°**
Enclosed, Partially Enclosed Buildings		



Notes:

1. Vertical scale denotes GC_p to be used with q_h.
2. Horizontal scale denotes effective wind area, in square feet (square meters).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. If a parapet equal to or higher than 3 ft (0.9m) is provided around the perimeter of the roof with θ ≤ 7°, the negative values of GC_p in Zone 3 shall be equal to those for Zone 2 and positive values of GC_p in Zones 2 and 4 shall be set equal to those for wall Zones 4 and 5 respectively n Figure 6-11A.
6. Values of GC_p for roof overhangs include pressure contributions from both upper and lower surfaces.
7. Notation:
   - *a*: 10 percent of least horizontal dimension or 0.4h, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m).
   - *h*: Eave height shall be used for θ ≤ 10°.
   - θ: Angle of plane of roof from horizontal, in degrees.

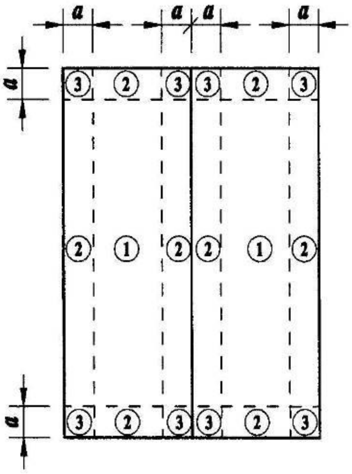
Components and Cladding – Method 2		$h \leq 60$ ft.
Figure 6-11C	**External Pressure Coefficients, GC_p**	**Gable/Hip Roofs $7° < \theta \leq 27°$**
Enclosed, Partially Enclosed Buildings		

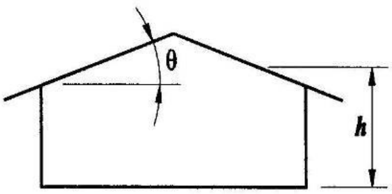


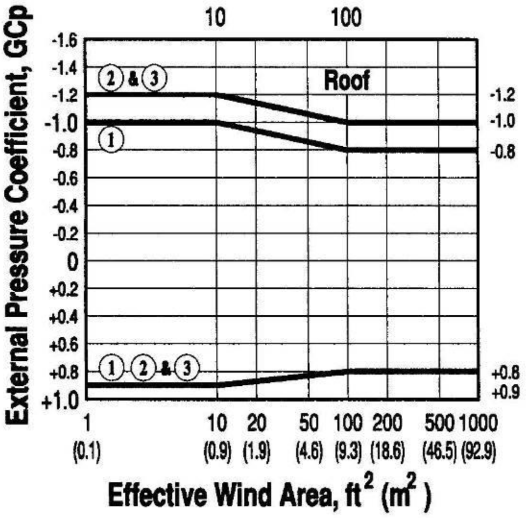
Notes:

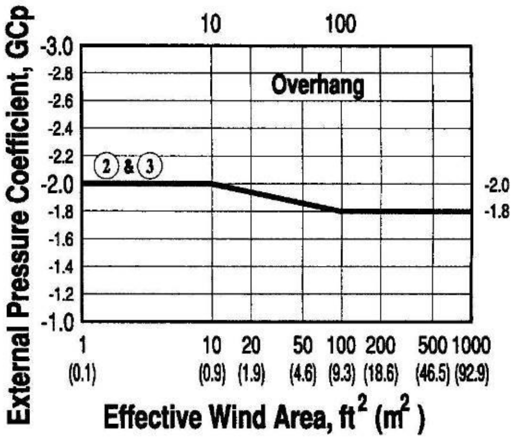
1. Vertical scale denotes GC_p to be used with q_h.
2. Horizontal scale denotes effective wind area, in square feet (square meters).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of GC_p for roof overhangs include pressure contributions from both upper and lower surfaces.
6. For hip roofs with $7° < \theta \leq 27°$, edge/ridge strips and pressure coefficients for ridges of gabled roofs shall apply on each hip.
7. For hip roofs with $\theta \leq 25°$, Zone 3 shall be treated as Zone 2.
8. Notation:
   - a: 10 percent of least horizontal dimension or 0.4h, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m).
   - h: Mean roof height, in feet (meters), except that eave height shall be used for $\theta \leq 10°$.
   - θ: Angle of plane of roof from horizontal, in degrees.

Components and Cladding – Method 2		h ≤ 60 ft.
Figure 6-11D	**External Pressure Coefficients GC_p**	**Gable Roofs 27°< θ ≤ 45°**
Enclosed, Partially Enclosed Buildings		





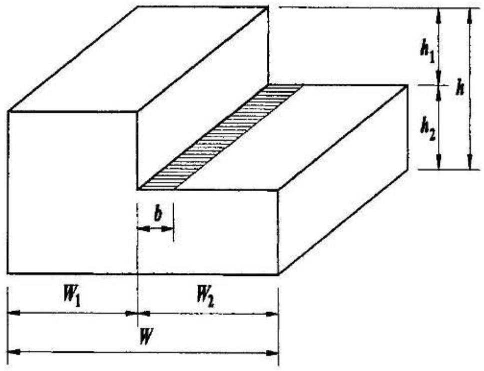




Notes:

1. Vertical scale denotes GC_p to be used with q_h.
2. Horizontal scale denotes effective wind area, in square feet (square meters).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of GC_p for roof overhangs include pressure contributions from both upper and lower surfaces.
6. Notation:
   - *a*: 10 percent of least horizontal dimension or 0.4h, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m).
   - *h*: Mean roof height, in feet (meters).
   - θ: Angle of plane of roof from horizontal, in degrees.

Components and Cladding – Method 2		h ≤ 60 ft.
Figure 6-12	**External Pressure Coefficients, GC_p**	**Stepped Roofs**
Enclosed, Partially Enclosed Buildings		



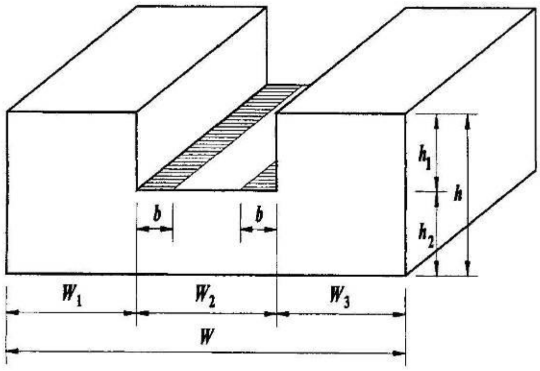
$h_1 \geq 10\,\text{ft.}\,(3\,\text{m})$

$b = 1.5\,h_1$

$b < 100\,\text{ft.}\,(30.5\,\text{m})$

$\dfrac{h_i}{h} = 0.3 \text{ to } 0.7$

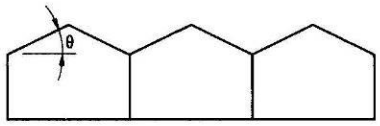
$\dfrac{W_i}{W} = 0.25 \text{ to } 0.75$



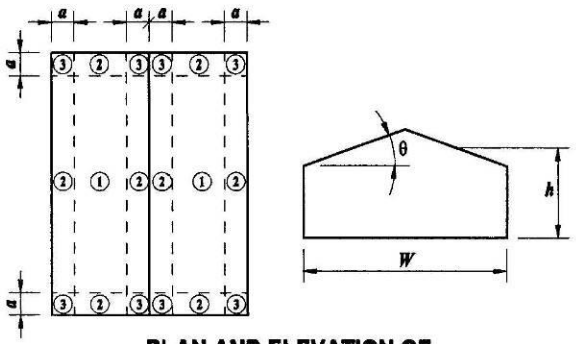
Notes:

1. On the lower level of flat, stepped roofs shown in Fig. 6-12, the zone designations and pressure coefficients shown in Fig. 6-11B shall apply, except that at the roof-upper wall intersection(s), Zone 3 shall be treated as Zone 2 and Zone 2 shall be treated as Zone 1. Positive values of GC_p equal to those for walls in Fig. 6-11A shall apply on the cross-hatched areas shown in Fig. 6-12.
2. Notation:
   - b: $1.5h_1$ in Fig. 6-12, but not greater than 100 ft (30.5 m).
   - h: Mean roof height, in feet (meters).
   - h_i: h_1 or h_2 in Fig. 6-12; $h = h_1 + h_2$; $h_1 \geq 10$ ft (3.1 m); $h_i/h = 0.3$ to 0.7.
   - W: Building width in Fig. 6-12.
   - W_i: W_1 or W_2 or W_3 in Fig. 6-12. $W = W_1 + W_2$ or $W_1 + W_2 + W_3$; $W_i/W = 0.25$ to 0.75.
   - θ: Angle of plane of roof from horizontal, in degrees.

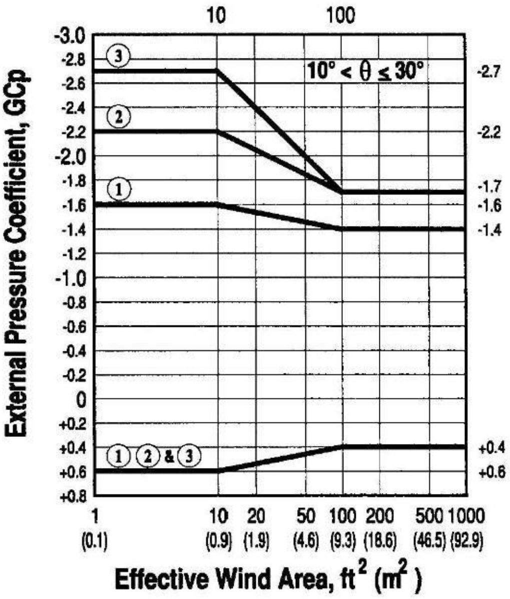
Components and Cladding – Method 2		h ≤ 60 ft.
Figure 6-13	**External Pressure Coefficients, GC_p**	**Multispan Gable Roofs**
Enclosed, Partially Enclosed Buildings		

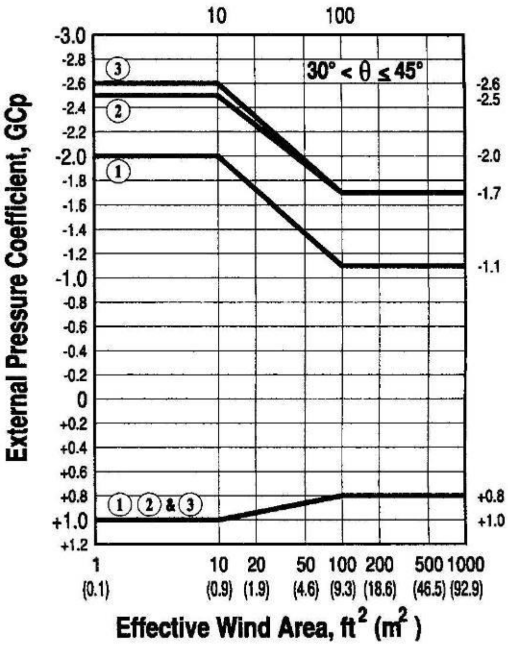


ELEVATION OF BUILDING
(2 or More Spans)



PLAN AND ELEVATION OF A SINGLE SPAN MODULE

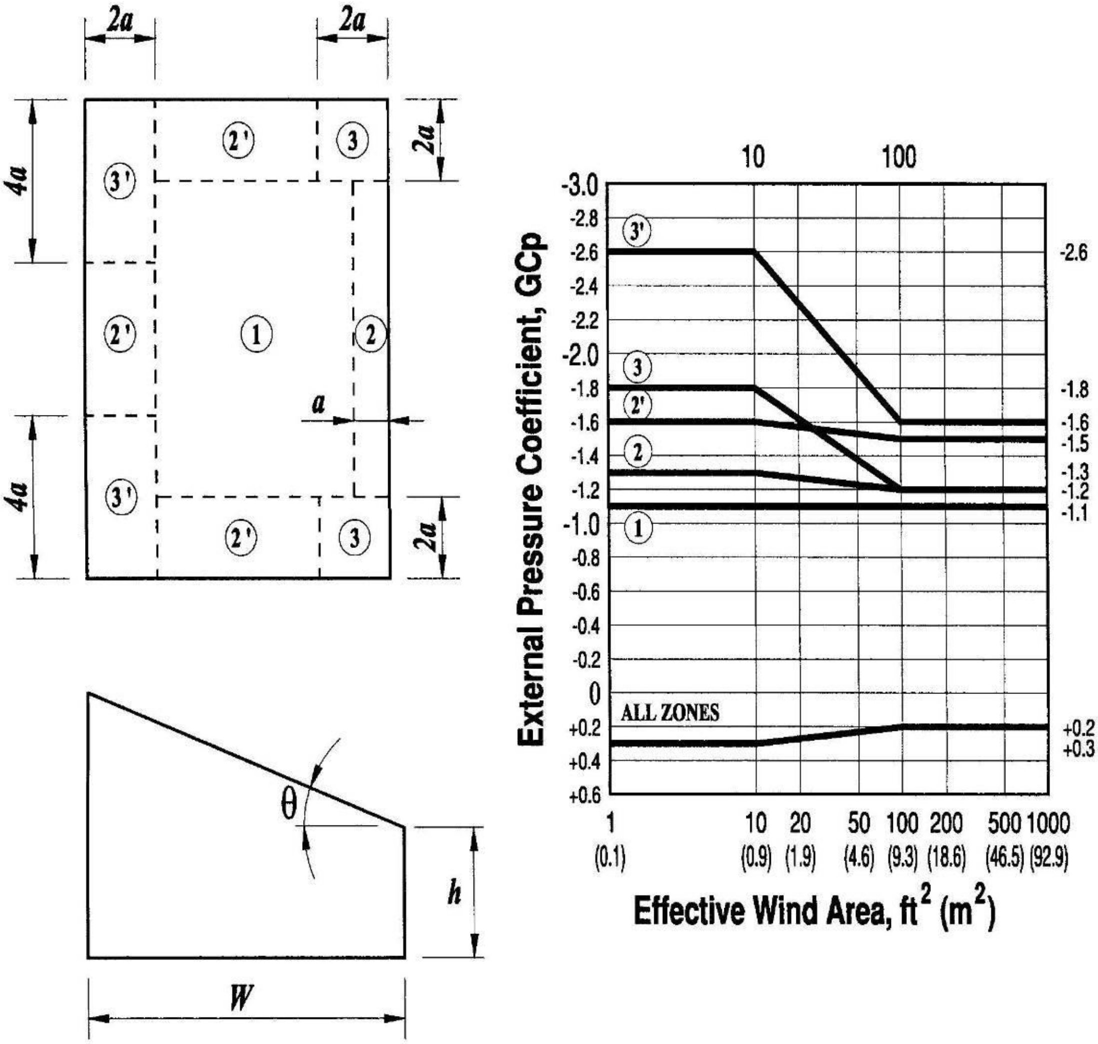




Notes:

1. Vertical scale denotes GC_p to be used with q_h.
2. Horizontal scale denotes effective wind area A, in square feet (square meters).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. For θ ≤ 10°, values of GC_p from Fig. 6-11 shall be used.
6. Notation:
   - *a*: 10 percent of least horizontal dimension of a single-span module or 0.4*h*, whichever is smaller, but not less than either 4 percent of least horizontal dimension of a single-span module or 3 ft (0.9 m).
   - *h*: Mean roof height, in feet (meters), except that eave height shall be used for θ ≤ 10°.
   - *W*: Building module width, in feet (meters).
   - θ: Angle of plane of roof from horizontal, in degrees.

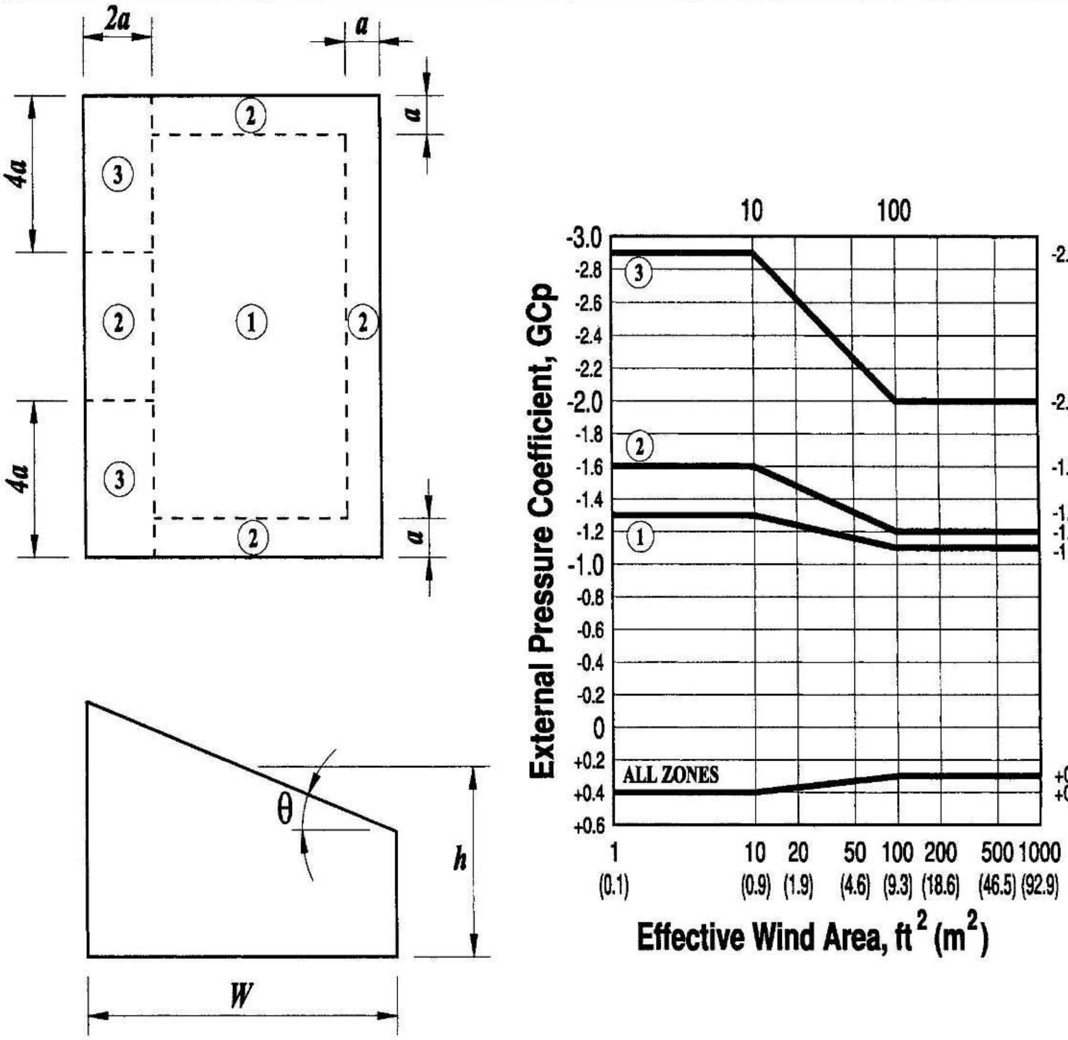
Components and Cladding – Method 2		h ≤ 60 ft.
Figure 6-14A	**External Pressure Coefficients, GC_p**	**Monoslope Roofs**
Enclosed, Partially Enclosed Buildings		**$3° < \theta \leq 10°$**



Notes:

1. Vertical scale denotes GC_p to be used with q_h.
2. Horizontal scale denotes effective wind area A, in square feet (square meters).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. For $\theta \leq 3°$, values of GC_p from Fig. 6-11B shall be used.
6. Notation:
   - *a*: 10 percent of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4 percent of least horizontal dimension or 3 ft (0.9 m).
   - *h*: Eave height shall be used for $\theta \leq 10°$.
   - *W*: Building width, in feet (meters).
   - θ: Angle of plane of roof from horizontal, in degrees.

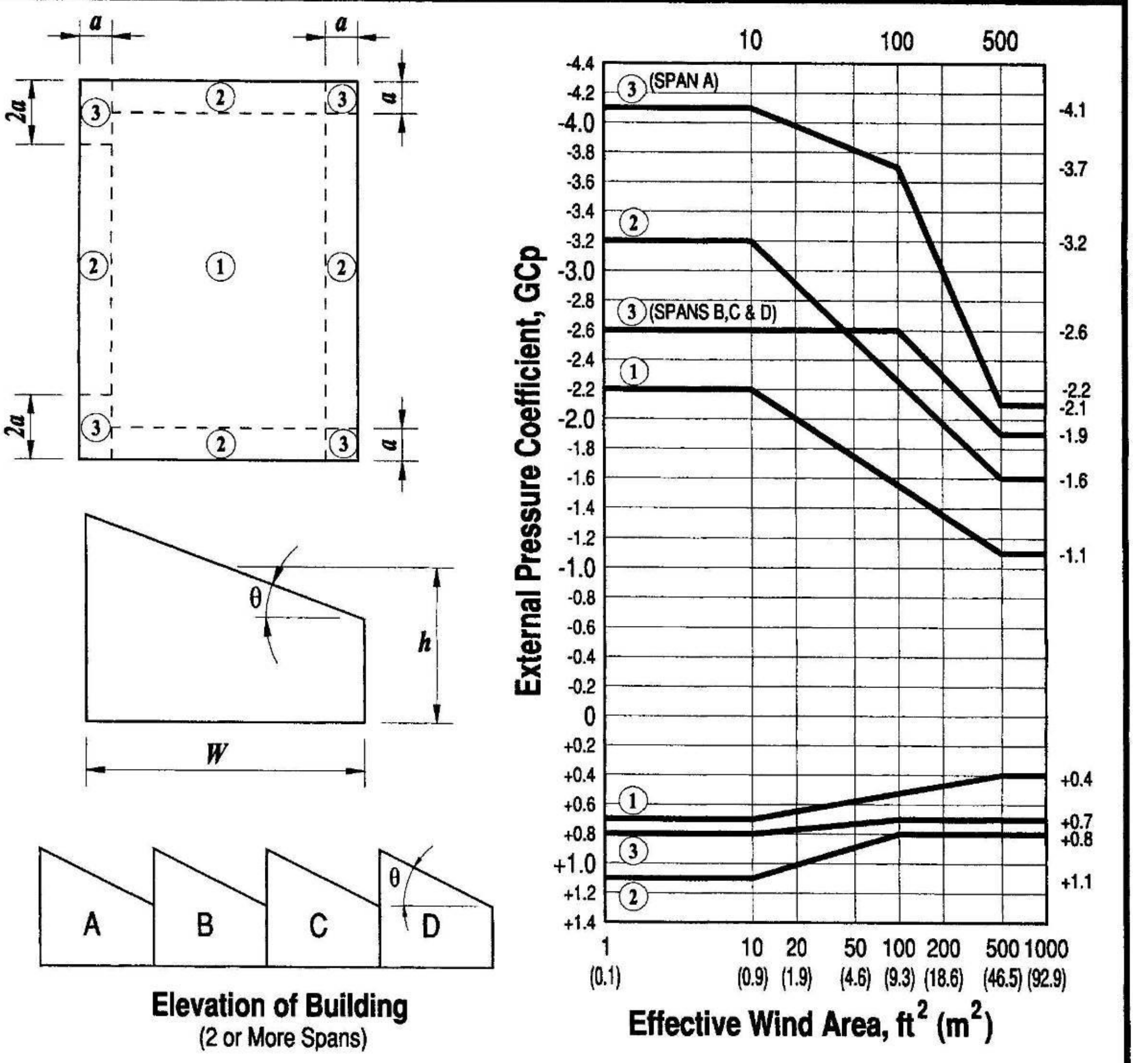
Components and Cladding – Method 2		$h \le 60$ ft.
Figure 6-14B	**External Pressure Coefficients, GC_p**	**Monoslope Roofs**
Enclosed, Partially Enclosed Buildings		**10°< θ ≤ 30°**



Notes:

1. Vertical scale denotes GC_p to be used with q_h.
2. Horizontal scale denotes effective wind area A, in square feet (square meters).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Notation:
   - a: 10 percent of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4 percent of least horizontal dimension or 3 ft (0.9 m).
   - h: Mean roof height, in feet (meters).
   - W: Building width, in feet (meters).
   - θ: Angle of plane of roof from horizontal, in degrees.

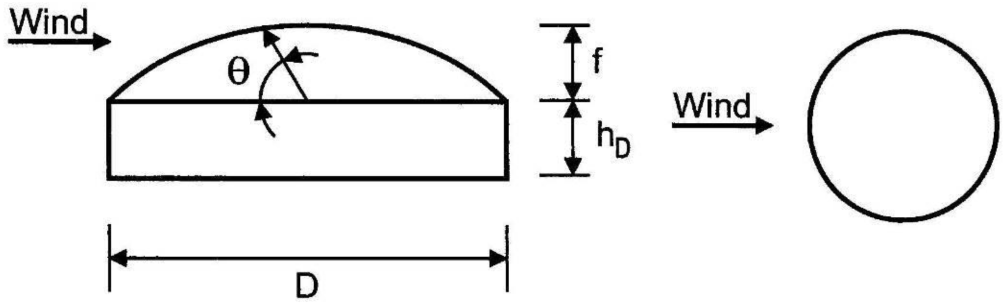
Components and Cladding – Method 2		h ≤ 60 ft.
Figure 6-15	External Pressure Coefficients, GC_p	Sawtooth Roofs
Enclosed, Partially Enclosed Buildings		



Notes:

1. Vertical scale denotes GC_p to be used with q_h.
2. Horizontal scale denotes effective wind area A, in square feet (square meters).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. For θ ≤ 10°, values of GC_p from Fig. 6-11 shall be used.
6. Notation:
   - a: 10 percent of least horizontal dimension or 0.4h, whichever is smaller, but not less than either 4 percent of least horizontal dimension or 3 ft (0.9 m).
   - h: Mean roof height, in feet (meters), except that eave height shall be used for θ ≤ 10°.
   - W: Building width, in feet (meters).
   - θ: Angle of plane of roof from horizontal, in degrees.

Components and Cladding – Method 2		All Heights
Figure 6-16	External Pressure Coefficients, GC_p	Domed Roofs
Enclosed, Partially Enclosed Buildings and Structures		

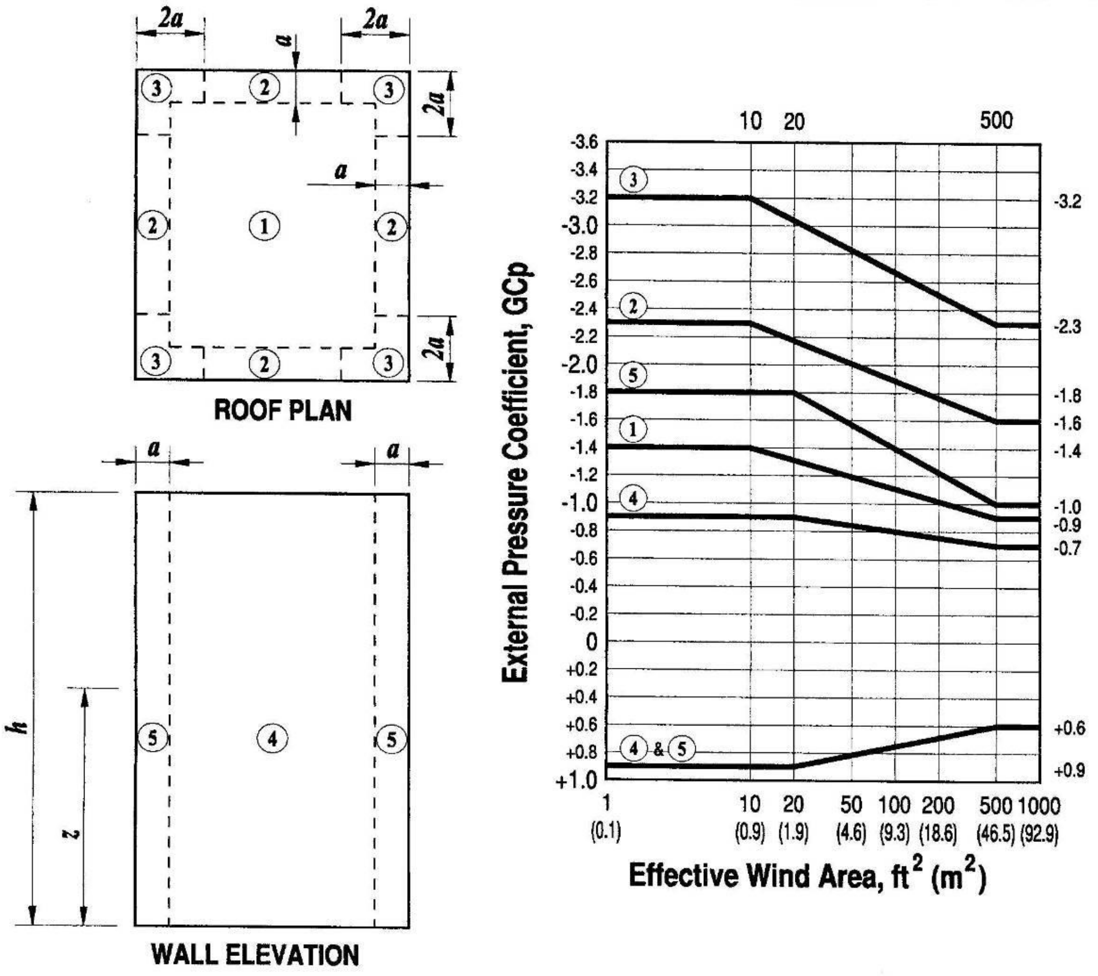


External Pressure Coefficients for Domes with a Circular Base			
	Negative Pressures	Positive Pressures	Positive Pressures
θ, degrees	0 – 90	0 – 60	61 – 90
GC_p	-0.9	+0.9	+0.5

Notes:

1. Values denote GC_p to be used with $q_{(h_D+f)}$ where $h_D + f$ is the height at the top of the dome.
2. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
3. Each component shall be designed for the maximum positive and negative pressures.
4. Values apply to $0 \le h_D/D \le 0.5$, $0.2 \le f/D \le 0.5$.
5. $\theta = 0$ degrees on dome springline, $\theta = 90$ degrees at dome center top point. f is measured from springline to top.

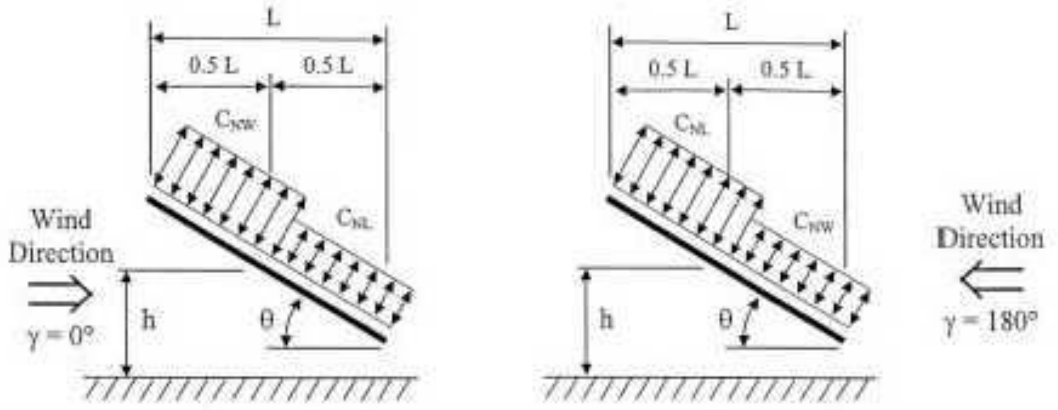
Components and Cladding – Method 2		h > 60 ft.
Figure 6-17	**External Pressure Coefficients, GCp**	**Walls & Roofs**
Enclosed, Partially Enclosed Buildings		



Notes:

1. Vertical scale denotes GC_p to be used with appropriate q_z or q_h.
2. Horizontal scale denotes effective wind area A, in square feet (square meters).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Use q_z with positive values of GC_p and q_h with negative values of GC_p.
5. Each component shall be designed for maximum positive and negative pressures.
6. Coefficients are for roofs with angle $\theta \leq 10°$. For other roof angles and geometry, use GC_p values from Fig. 6-11 and attendant q_h based on exposure defined in 6.5.6.
7. If a parapet equal to or higher than 3 ft (0.9m) is provided around the perimeter of the roof with $\theta \leq 10°$, Zone 3 shall be treated as Zone 2.
8. Notation:
   - a: 10 percent of least horizontal dimension, but not less than 3 ft (0.9 m).
   - h: Mean roof height, in feet (meters), except that eave height shall be used for $\theta \leq 10°$.
   - z: height above ground, in feet (meters).
   - θ: Angle of plane of roof from horizontal, in degrees.

Main Wind Force Resisting System		$0.25 \leq h/L \leq 1.0$
Figure 6-18A	**Net Pressure Coefficient, C_N**	**Monoslope Free Roofs** $q \leq 45°, \gamma = 0°, 180°$
Open Buildings		

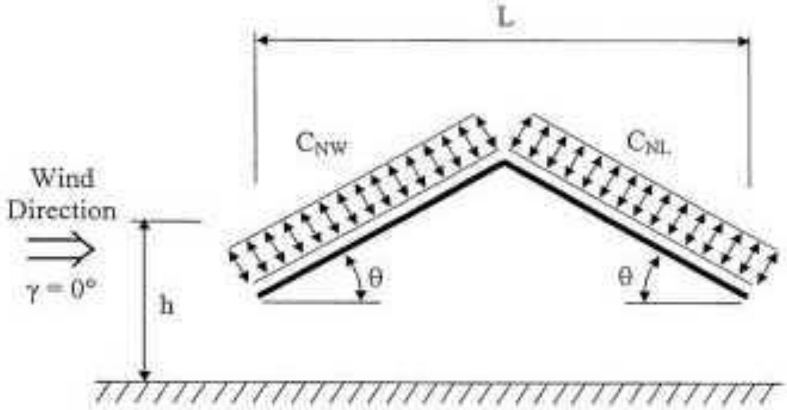


Roof Angle θ	Load Case	Wind Direction, $\gamma = 0°$				Wind Direction, $\gamma = 180°$			
		Clear Wind Flow		Obstructed Wind Flow		Clear Wind Flow		Obstructed Wind Flow	
		C_{NW}	C_{NL}	C_{NW}	C_{NL}	C_{NW}	C_{NL}	C_{NW}	C_{NL}
0°	A	1.2	0.3	-0.5	-1.2	1.2	0.3	-0.5	-1.2
	B	-1.1	-0.1	-1.1	-0.6	-1.1	-0.1	-1.1	-0.6
7.5°	A	-0.6	-1	-1	-1.5	0.9	1.5	-0.2	-1.2
	B	-1.4	0	-1.7	-0.8	1.6	0.3	0.8	-0.3
15°	A	-0.9	-1.3	-1.1	-1.5	1.3	1.6	0.4	-1.1
	B	-1.9	0	-2.1	-0.6	1.8	0.6	1.2	-0.3
22.5°	A	-1.5	-1.6	-1.5	-1.7	1.7	1.8	0.5	-1
	B	-2.4	-0.3	-2.3	-0.9	2.2	0.7	1.3	0
30°	A	-1.8	-1.8	-1.5	-1.8	2.1	2.1	0.6	-1
	B	-2.5	-0.5	-2.3	-1.1	2.6	1	1.6	0.1
37.5°	A	-1.8	-1.8	-1.5	-1.8	2.1	2.2	0.7	-0.9
	B	-2.4	-0.6	-2.2	-1.1	2.7	1.1	1.9	0.3
45°	A	-1.6	-1.8	-1.3	-1.8	2.2	2.5	0.8	-0.9
	B	-2.3	-0.7	-1.9	-1.2	2.6	1.4	2.1	0.4

Notes:

1. C_{NW} and C_{NL} denote net pressures (contributions from top and bottom surfaces) for windward and leeward half of roof surfaces, respectively.
2. Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50%. Obstructed wind flow denotes objects below roof inhibiting wind flow (>50% blockage).
3. For values of θ between 7.5° and 45°, linear interpolation is permitted. For values of θ less than 7.5°, use load coefficients for 0°.
4. Plus and minus signs signify pressures acting towards and away from the top roof surface, respectively.
5. All load cases shown for each roof angle shall be investigated.
6. Notation:
   - L : horizontal dimension of roof, measured in the along wind direction, ft. (m)
   - h : mean roof height, ft. (m)
   - γ : direction of wind, degrees
   - θ : angle of plane of roof from horizontal, degrees

Main Wind Force Resisting System		$0.25 \le h/L \le 1.0$
Figure 6-18B	Net Pressure Coefficient, C_N	**Pitched Free Roofs** $\theta \le 45°, \gamma = 0°, 180°$
Open Buildings		

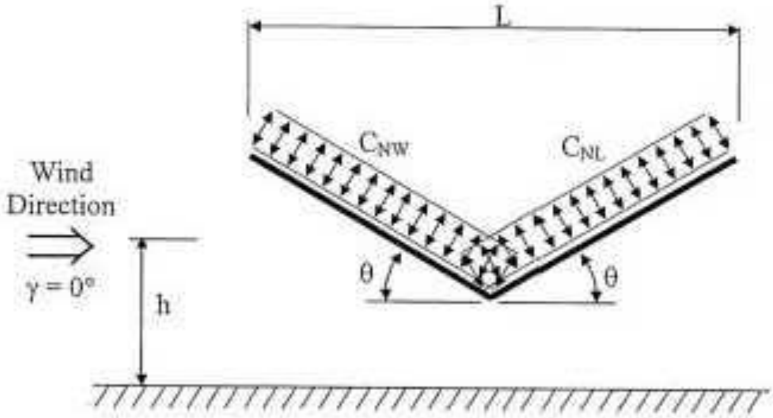


Roof Angle, θ	Load Case	Wind Direction, $\gamma = 0°, 180°$			
		Clear Wind Flow		Obstructed Wind Flow	
		C_{NW}	C_{NL}	C_{NW}	C_{NL}
7.5°	A	1.1	-0.3	-1.6	-1
	B	0.2	-1.2	-0.9	-1.7
15°	A	1.1	-0.4	-1.2	-1
	B	0.1	-1.1	-0.6	-1.6
22.5°	A	1.1	0.1	-1.2	-1.2
	B	-0.1	-0.8	-0.8	-1.7
30°	A	1.3	0.3	-0.7	-0.7
	B	-0.1	-0.9	-0.2	-1.1
37.5°	A	1.3	0.6	-0.6	-0.6
	B	-0.2	-0.6	-0.3	-0.9
45°	A	1.1	0.9	-0.5	-0.5
	B	-0.3	-0.5	-0.3	-0.7

Notes:

1. C_{NW} and C_{NL} denote net pressures (contributions from top and bottom surfaces) for windward and leeward half of roof surfaces, respectively.
2. Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50%. Obstructed wind flow denotes objects below roof inhibiting wind flow (>50% blockage).
3. For values of θ between 7.5° and 45°, linear interpolation is permitted. For values of θ less than 7.5°, use monoslope roof load coefficients.
4. Plus and minus signs signify pressures acting towards and away from the top roof surface, respectively.
5. All load cases shown for each roof angle shall be investigated.
6. Notation:
   - L : horizontal dimension of roof, measured in the along wind direction, ft. (m)
   - h : mean roof height, ft. (m)
   - γ : direction of wind, degrees
   - θ : angle of plane of roof from horizontal, degrees

Main Wind Force Resisting System		$0.25 \leq h/L \leq 1.0$
Figure 6-18C	Net Pressure Coefficient, C_N	Troughed Free Roofs $\theta \leq 45°, \gamma = 0°, 180°$
Open Buildings		

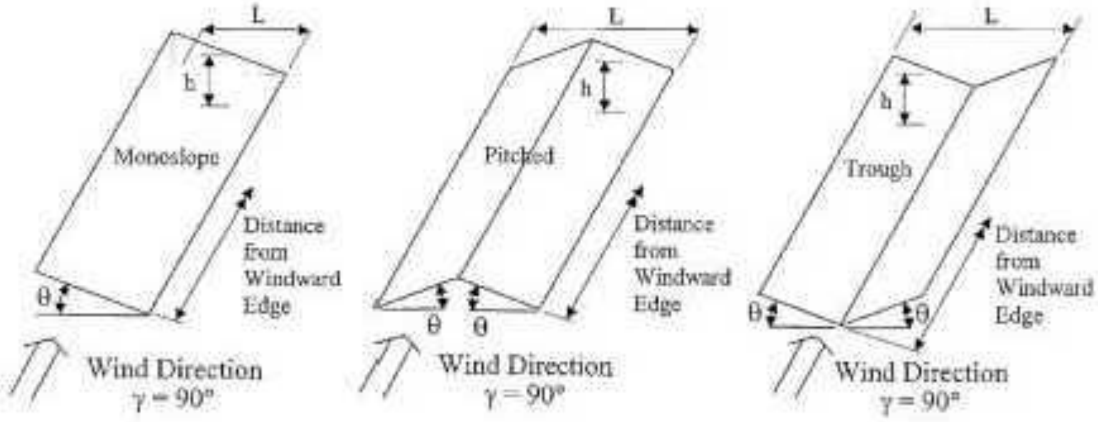


Roof Angle θ	Load Case	Wind Direction, $\gamma = 0°, 180°$			
		Clear Wind Flow		Obstructed Wind Flow	
		C_{NW}	C_{NL}	C_{NW}	C_{NL}
7.5°	A	1.1	0.3	1.6	0.5
	B	02	1.2	0.9	0.8
15°	A	1.1	0.4	1.2	0.5
	B	0.1	1.1	0.6	0.8
22.5°	A	1.1	0.1	1.2	0.6
	B	0.1	0.8	0.8	0.8
30°	A	13	0.3	1.4	0.4
	B	0.1	0.9	0.2	0.5
37.5°	A	13	0.6	1.4	0.3
	B	02	0.6	0.3	0.4
45°	A	1.1	0.9	1.2	0.3
	B	03	0.5	0.3	0.4

Notes:

1. C_{NW} and C_{NL} denote net pressures (contributions from top and bottom surfaces) for windward and leeward half of roof surfaces, respectively.
2. Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50%. Obstructed wind flow denotes objects below roof inhibiting wind flow (>50% blockage).
3. For values of θ between 7.5° and 45°, linear interpolation is permitted. For values of θ less than 7.5°, use monoslope roof load coefficients.
4. Plus and minus signs signify pressures acting towards and away from the top roof surface, respectively.
5. All load cases shown for each roof angle shall be investigated.
6. Notation:
   - L : horizontal dimension of roof, measured in the along wind direction, ft. (m)
   - h : mean roof height, ft. (m)
   - γ : direction of wind, degrees
   - θ : angle of plane of roof from horizontal, degrees

Main Wind Force Resisting System		$0.25 \le h/L \le 1.0$
Figure 6-18D	Net Pressure Coefficient, C_N	**Troughed Free Roofs**
Open Buildings		$\theta \le 45°, \lambda = 0°, 180°$

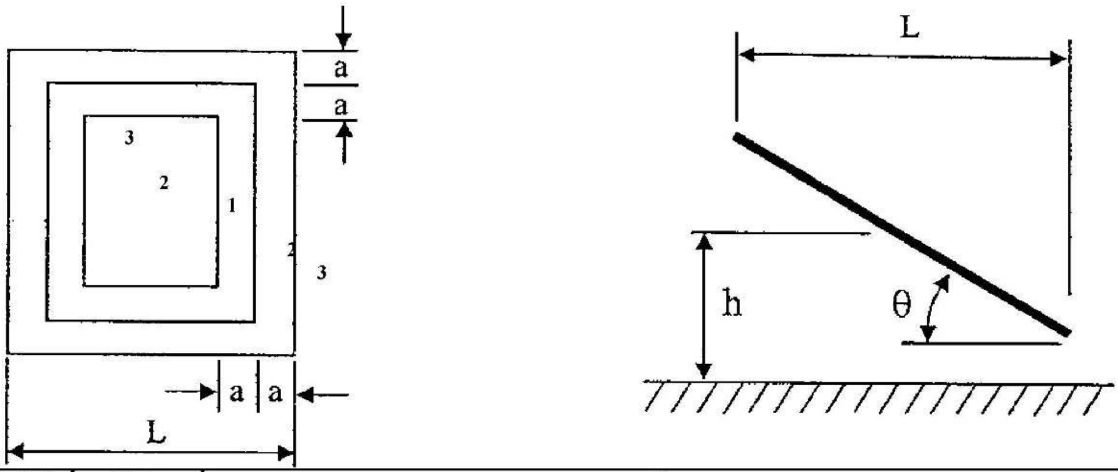


Horizontal Distance from Windward Edge	Roof Angle θ	Load Case	Clear Wind Flow	Obstructed Wind Flow
			C_N	C_N
$\le h$	All Shapes	A	0.8	1.2
	$\theta \le 45°$	B	0.8	0.5
$> h, \le 2h$	All Shapes	A	0.6	0.9
	$\theta \le 45°$	B	0.5	0.5
$> 2h$	All Shapes	A	0.3	0.6
	$\theta \le 45°$	B	0.3	0.3

Notes:

1. C_N denotes net pressures (contributions from top and bottom surfaces).
2. Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50%. Obstructed wind flow denotes objects below roof inhibiting wind flow (> 50% blockage).
3. Plus and minus signs signify pressures acting towards and away from the top roof surface, respectively.
4. All load cases shown for each roof angle shall be investigated.
5. For monoslope roofs with theta less than 5 degrees, Cn values shown apply also for cases where gamma = 0 degrees and 0.05 less than or equal to h/L less than or equal to 0.25. See Figure 6-18A for other h/L values.
6. Notation:
   - L : horizontal dimension of roof, measured in the along wind direction, ft. (m)
   - h : mean roof height, ft. (m)
   - γ : direction of wind, degrees
   - θ : angle of plane of roof from horizontal, degrees

Components and Cladding		0.25 ≤ h/L ≤ 1.0
Figure 6-19A	Net Pressure Coefficient, C_N	**Monoslope Free Roofs**
Open Buildings		$\theta \leq 45°$

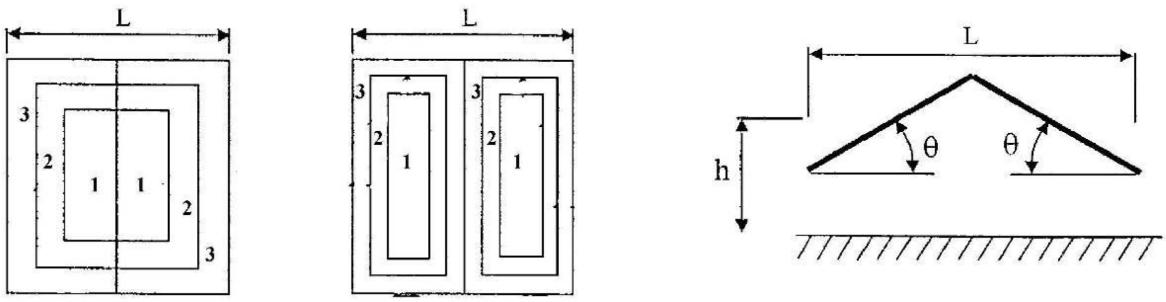


Roof Angle θ	Effective Wind Area	C_N											
		Clear Wind Flow						Obstructed Wind Flow					
		Zone 3		Zone 2		Zone 1		Zone 3		Zone 2		Zone 1	
0°	$\leq a^2$	2.4	-3.3	1.8	-1.7	1.2	-1.1	1	-3.6	0.8	-1.8	0.5	-1.2
	$> a^2, \leq 4.0a^2$	1.8	-1.7	1.8	-1.7	1.2	-1.1	0.8	-1.8	0.8	-1.8	0.5	-1.2
	$> 4.0a^2$	1.2	-1.1	1.2	-1.1	1.2	-1.1	0.5	-1.2	0.5	-1.2	0.5	-1.2
7.5°	$\leq a^2$	3.2	-4.2	2.4	-2.1	1.6	-1.4	1.6	-5.1	1.2	-2.6	0.8	-1.7
	$> a^2, \leq 4.0a^2$	2.4	-2.1	2.4	-2.1	1.6	-1.4	1.2	-2.6	1.2	-2.6	0.8	-1.7
	$> 4.0a^2$	1.6	-1.4	1.6	-1.4	1.6	-1.4	0.8	-1.7	0.8	-1.7	0.8	-1.7
15°	$\leq a^2$	3.6	-3.8	2.7	-2.9	1.8	-1.9	2.4	-4.2	1.8	-3.2	1.2	-2.1
	$>a^2, \leq 4.0a^2$	2.7	-2.9	2.7	-2.9	1.8	-1.9	1.8	-3.2	1.8	-3.2	1.2	-2.1
	$> 4.0a^2$	1.8	-1.9	1.8	-1.9	1.8	-1.9	1.2	-2.1	1.2	-2.1	1.2	-2.1
30°	$\leq a^2$	5.2	-5	3.9	-3.8	2.6	-2.5	3.2	-4.6	2.4	-3.5	1.6	-2.3
	$> a^2, \leq 4.0a^2$	3.9	-3.8	3.9	-3.8	2.6	-2.5	2.4	-3.5	2.4	-3.5	1.6	-2.3
	$> 4.0a^2$	2.6	-2.5	2.6	-2.5	2.6	-2.5	1.6	-2.3	1.6	-2.3	1.6	-2.3
45°	$\leq a^2$	5.2	-4.6	3.9	-3.5	2.6	-2.3	4.2	-3.8	3.2	-2.9	2.1	-1.9
	$> a^2, \leq 4.0a^2$	3.9	-3.5	3.9	-3.5	2.6	-2.3	3.2	-2.9	3.2	-2.9	2.1	-1.9
	$> 4.0a^2$	2.6	-2.3	2.6	-2.3	2.6	-2.3	2.1	-1.9	2.1	-1.9	2.1	-1.9

Notes:

1. C_N denotes net pressures (contributions from top and bottom surfaces).
2. Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50%. Obstructed wind flow denotes objects below roof inhibiting wind flow (>50% blockage).
3. For values of θ other than those shown, linear interpolation is permitted.
4. Plus and minus signs signify pressures acting towards and away from the top roof surface, respectively.
5. Components and cladding elements shall be designed for positive and negative pressure coefficients shown.
6. Notation:
   - a : 10% of least horizontal dimension or 0.4h, whichever is smaller but not less than 4% of least horizontal dimension or 3 ft. (0.9 m)
   - h : mean roof height, ft. (m)
   - L : horizontal dimension of building, measured in along wind direction, ft. (m)
   - θ : angle of plane of roof from horizontal, degrees

Components and Cladding		0.25 ≤ h/L ≤ 1.0
Figure 6-19B	**Net Pressure Coefficient, C_N**	**Monoslope Free Roofs** $\theta \le 45°$
Open Buildings		



θ < 10°

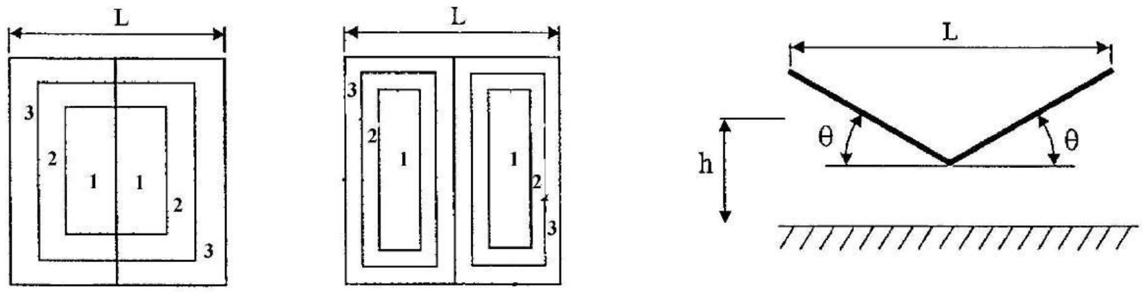
θ ≥ 10°

Roof Angle θ	Effective Wind Area	C_N											
		Clear Wind Flow					O	bstructed Wind Flow					
		Zone 3		Zone 2		Zone 1		Zone 3		Zone 2		Zone 1	
0°	≤ a^2	2.4	-3.3	1.8	-1.7	1.2	-1.1	1	-3.6	0.8	-1.8	0.5	-1.2
	> a^2, ≤ 4.0a^2	1.8	-1.7	1.8	-1.7	1.2	-1.1	0.8	-1.8	0.8	-1.8	0.5	-1.2
	> 4.0a^2	1.2	-1.1	1.2	-1.1	1.2	-1.1	0.5	-1.2	0.5	-1.2	0.5	-1.2
7.5°	≤ a^2	2.2	-3.6	1.7	-1.8	1.1	-1.2	1	-5.1	0.8	-2.6	0.5	-1.7
	> a^2, ≤ 4.0a^2	1.7	-1.8	1.7	-1.8	1.1	-1.2	0.8	-2.6	0.8	-2.6	0.5	-1.7
	> 4.0a^2	1.1	-1.2	1.1	-1.2	1.1	-1.2	0.5	-1.7	0.5	-1.7	0.5	-1.7
15°	≤ a^2	2.2	-2.2	1.7	-1.7	1.1	-1.1	1	-3.2	0.8	-2.4	0.5	-1.6
	>a^2, ≤ 4.0a^2	1.7	-1.7	1.7	-1.7	1.1	-1.1	0.8	-2.4	0.8	-2.4	0.5	-1.6
	> 4.0a^2	1.1	-1.1	1.1	-1.1	1.1	-1.1	0.5	-1.6	0.5	-1.6	0.5	-1.6
30°	≤ a^2	2.6	-1.8	2	-1.4	1.3	-0.9	1	-2.4	0.8	-1.8	0.5	-1.2
	> a^2, ≤ 4.0a^2	2	-1.4	2	-1.4	1.3	-0.9	0.8	-1.8	0.8	-1.8	0.5	-1.2
	> 4.0a^2	1.3	-0.9	1.3	-0.9	1.3	-0.9	0.5	-1.2	0.5	-1.2	0.5	-1.2
45°	≤ a^2	2.2	-1.6	1.7	-1.2	1.1	-0.8	1	-2.4	0.8	-1.8	0.5	-1.2
	> a^2, ≤ 4.0a^2	1.7	-1.2	1.7	-1.2	1.1	-0.8	0.8	-1.8	0.8	-1.8	0.5	-1.2
	> 4.0a^2	1.1	-0.8	1.1	-0.8	1.1	-0.8	0.5	-1.2	0.5	-1.2	0.5	-1.2

Notes:

1. C_N denotes net pressures (contributions from top and bottom surfaces).
2. Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50%. Obstructed wind flow denotes objects below roof inhibiting wind flow (>50% blockage).
3. For values of θ other than those shown, linear interpolation is permitted.
4. Plus and minus signs signify pressures acting towards and away from the top roof surface, respectively.
5. Components and cladding elements shall be designed for positive and negative pressure coefficients shown.
6. Notation:
   - a : 10% of least horizontal dimension or 0.4h, whichever is smaller but not less than 4% of least horizontal dimension or 3 ft. (0.9 m)
   - h : mean roof height, ft. (m)
   - L : horizontal dimension of building, measured in along wind direction, ft. (m)
   - θ : angle of plane of roof from horizontal, degrees

Components and Cladding		**0.25 ≤ h/L ≤ 1.0**
Figure 6-19C	**Net Pressure Coefficient, C_N**	**Troughed Free Roofs**
Open Buildings		**$\theta \leq 45°$**



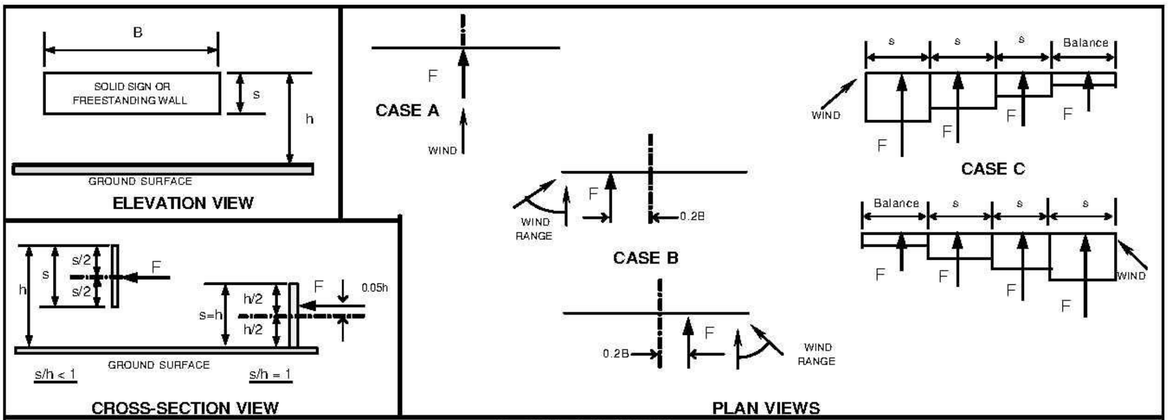
$\theta < 10°$ $\theta \geq 10°$

Roof Angle θ	Effective Wind Area	C_N											
		Clear Wind Flow						Obstructed Wind Flow					
		Zone 3		Zone 2		Zone 1		Zone 3		Zone 2		Zone 1	
0°	$\leq a^2$	2.4	-3.3	1.8	-1.7	1.2	-1.1	1	-3.6	0.8	-1.8	0.5	-1.2
	$> a^2, \leq 4.0a^2$	1.8	-1.7	1.8	-1.7	1.2	-1.1	0.8	-1.8	0.8	-1.8	0.5	-1.2
	$> 4.0a^2$	1.2	-1.1	1.2	-1.1	1.2	-1.1	0.5	-1.2	0.5	-1.2	0.5	-1.2
7.5°	$\leq a^2$	2.4	-3.3	1.8	-1.7	1.2	-1.1	1	-4.8	0.8	-2.4	0.5	-1.6
	$> a^2, \leq 4.0a^2$	1.8	-1.7	1.8	-1.7	1.2	-1.1	0.8	-2.4	0.8	-2.4	0.5	-1.6
	$> 4.0a^2$	1.2	-1.1	1.2	-1.1	1.2	-1.1	0.5	-1.6	0.5	-1.6	0.5	-1.6
15°	$\leq a^2$	2.2	-2.2	1.7	-1.7	1.1	-1.1	1	-2.4	0.8	-1.8	0.5	-1.2
	$> a^2, \leq 4.0a^2$	1.7	-1.7	1.7	-1.7	1.1	-1.1	0.8	-1.8	0.8	-1.8	0.5	-1.2
	$> 4.0a^2$	1.1	-1.1	1.1	-1.1	1.1	-1.1	0.5	-1.2	0.5	-1.2	0.5	-1.2
30°	$\leq a^2$	1.8	-2.6	1.4	-2	0.9	-1.3	1	-2.8	0.8	-2.1	0.5	-1.4
	$> a^2, \leq 4.0a^2$	1.4	-2	1.4	-2	0.9	-1.3	0.8	-2.1	0.8	-2.1	0.5	-1.4
	$> 4.0a^2$	0.9	-1.3	1.9	-1.3	0.9	-1.3	0.5	-1.4	0.5	-1.4	0.5	-1.4
45°	$\leq a^2$	1.6	-2.2	1.2	-1.7	0.8	-1.1	1	-2.4	0.8	-1.8	0.5	-1.2
	$>a^2, \leq 4.0a^2$	1.2	-1.7	1.2	-1.7	0.8	-1.1	0.8	-1.8	0.8	-1.8	0.5	-1.2
	$> 4.0a^2$	0.8	-1.1	1.8	-1.1	0.8	-1.1	0.5	-1.2	0.5	-1.2	0.5	-1.2

Notes:

1. C_N denotes net pressures (contributions from top and bottom surfaces).
2. Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50%. Obstructed wind flow denotes objects below roof inhibiting wind flow (>50% blockage).
3. For values of θ other than those shown, linear interpolation is permitted.
4. Plus and minus signs signify pressures acting towards and away from the top roof surface, respectively.
5. Components and cladding elements shall be designed for positive and negative pressure coefficients shown.
6. Notation:
   - a : 10% of least horizontal dimension or 0.4h, whichever is smaller but not less than 4% of least horizontal dimension or 3 ft. (0.9 m)
   - h : mean roof height, ft. (m)
   - L : horizontal dimension of building, measured in along wind direction, ft. (m)
   - θ : angle of plane of roof from horizontal, degrees

Other Structures -- Method 2		All Heights
Figure 6-20	**Force Coefficients, C_f**	**Solid Freestanding Walls & Solid Signs**



C_f, CASE A & CASE B

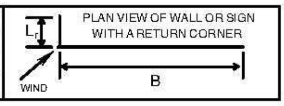
Clearance Ratio, s/h	Aspect Ratio, B/s											
	≤ 0.05	0.1	0.2	0.5	1	2	4	5	10	20	30	≥ 45
1	1.80	1.70	1.65	1.55	1.45	1.40	1.35	1.35	1.30	1.30	1.30	1.30
0.9	1.85	1.75	1.70	1.60	1.55	1.50	1.45	1.45	1.40	1.40	1.40	1.40
0.7	1.90	1.85	1.75	1.70	1.65	1.60	1.60	1.55	1.55	1.55	1.55	1.55
0.5	1.95	1.85	1.80	1.75	1.75	1.70	1.70	1.70	1.70	1.70	1.70	1.75
0.3	1.95	1.90	1.85	1.80	1.80	1.80	1.80	1.80	1.80	1.85	1.85	1.85
0.2	1.95	1.90	1.85	1.80	1.80	1.80	1.80	1.80	1.85	1.90	1.90	1.95
≤ 0.16	1.95	1.90	1.85	1.85	1.80	1.80	1.85	1.85	1.85	1.90	1.90	1.95

C_f, CASE C

Region (horizontal distance from windward edge)	Aspect Ratio, B/s									Region (horizontal distance from windward edge)	Aspect Ratio, B/s	
	2	3	4	5	6	7	8	9	10		13	≥ 45
0 to s	2.25	2.60	2.90	3.10*	3.30*	3.40*	3.55*	3.65*	3.75*	0 to s	4.00*	4.30*
s to 2s	1.50	1.70	1.90	2.00	2.15	2.25	2.30	2.35	2.45	s to 2s	2.60	2.55
2s to 3s		1.15	1.30	1.45	1.55	1.65	1.70	1.75	1.85	2s to 3s	2.00	1.95
3s to 10s			1.10	1.05	1.05	1.05	1.05	1.00	0.95	3s to 4s	1.50	1.85
										4s to 5s	1.35	1.85
										5s to 10s	0.90	1.10
										>10s	0.55	0.55

*Values shall be multiplied by the following reduction factor when a return corner is present:

L_r/s	Reduction Factor
0.3	0.90
1.0	0.75
≥ 2	0.60



Notes:

1. The term "signs" in notes below also applies to "freestanding walls".
2. Signs with openings comprising less than 30% of the gross area are classified as solid signs. Force coefficients for solid signs with openings shall be permitted to be multiplied by the reduction factor $(1 - (1 - \varepsilon)^{1.5})$.
3. To allow for both normal and oblique wind directions, the following cases shall be considered:
   For s/h < 1 :
   CASE A: resultant force acts normal to the face of the sign through the geometric center.
   CASE B: resultant force acts normal to the face of the sign at a distance from the geometric center toward the windward edge equal to 0.2 times the average width of the sign.
   For B/s ≥ 2, CASE C must also be considered:
   CASE C: resultant forces act normal to the face of the sign through the geometric centers of each region.
   For s/h = 1 :
   The same cases as above except that the vertical locations of the resultant forces occur at a distance above the geometric center equal to 0.05 times the average height of the sign.
4. For CASE C where s/h > 0.8, force coefficients shall be multiplied by the reduction factor (1.8 - s/h).
5. Linear interpolation is permitted for values of s/h, B/s and L_r/s other than shown
6. Notation:
   B: horizontal dimension of sign, in feet (meters);
   h: height of the sign, in feet (meters);
   s: vertical dimension of the sign, in feet (meters);
   ε: ratio of solid area to gross area;
   L_r: horizontal dimension of return corner, in feet (meters)

Other Structures – Method 2		All Heights
Figure 6-21	Force Coefficients, C_f	Chimneys, Tanks, Rooftop Equipment, & Similar Structures

Cross-Section	Type of Surface	h/D		
		1	7	25
Square (wind normal to face)	All	1.3	1.4	2.0
Square (wind along diagonal)	All	1.0	1.1	1.5
Hexagonal or octagonal	All	1.0	1.2	1.4
Round $(D\sqrt{q_z} > 2.5)$ $(D\sqrt{q_z} > 5.3, D \text{ in m}, q_z \text{ in N/m}^2)$	Moderately smooth	0.5	0.6	0.7
	Rough (D'/D = 0.02)	0.7	0.8	0.9
	Very rough (D'/D = 0.08)	0.8	1.0	0.2
Round $(D\sqrt{q_z} \le 2.5)$ $(D\sqrt{q_z} \le 5.3, D \text{ in m}, q_z \text{ in N/m}^2)$	All	0.7	0.8	1.2

Notes:

1. The design wind force shall be calculated based on the area of the structure projected on a plane normal to the wind direction. The force shall be assumed to act parallel to the wind direction.
2. Linear interpolation is permitted for *h/D* values other than shown.
3. Notation:

   D: diameter of circular cross-section and least horizontal dimension of square, hexagonal or octagonal cross-sections at elevation under consideration, in feet (meters);

   D': depth of protruding elements such as ribs and spoilers, in feet (meters); and

   h: height of structure, in feet (meters); and

   q_z: velocity pressure evaluated at height z above ground, in pounds per square foot (N/m^2).

Other Structures – Method 2		All Heights
Figure 6-21	Force Coefficients, C_f	Chimneys, Tanks, Rooftop Equipment, & Similar Structures

Cross-Section	Type of Surface	h/D		
		1	7	25
Square (wind normal to face)	All	1.3	1.4	2.0
Square (wind along diagonal)	All	1.0	1.1	1.5
Hexagonal or octagonal	All	1.0	1.2	1.4
Round $(D\sqrt{q_z} > 2.5)$ $(D\sqrt{q_z} > 5.3, D \text{ in m}, q_z \text{ in N/m}^2)$	Moderately smooth	0.5	0.6	0.7
	Rough (D'/D = 0.02)	0.7	0.8	0.9
	Very rough (D'/D = 0.08)	0.8	1.0	0.2
Round $(D\sqrt{q_z} \leq 2.5)$ $(D\sqrt{q_z} \leq 5.3, D \text{ in m}, q_z \text{ in N/m}^2)$	All	0.7	0.8	1.2

Notes:

1. The design wind force shall be calculated based on the area of the structure projected on a plane normal to the wind direction. The force shall be assumed to act parallel to the wind direction.
2. Linear interpolation is permitted for *h/D* values other than shown.
3. Notation:

   D: diameter of circular cross-section and least horizontal dimension of square, hexagonal or octagonal cross-sections at elevation under consideration, in feet (meters);

   D': depth of protruding elements such as ribs and spoilers, in feet (meters); and

   h: height of structure, in feet (meters); and

   q_z: velocity pressure evaluated at height z above ground, in pounds per square foot (N/m^2).

Other Structures – Method 2		All Heights
Figure 6-22	**Force Coefficients, C_f**	**Open Signs & Lattice Frameworks**

$\in$	Flat-Sided Members	Rounded Members	
		$D\sqrt{q_z} \leq 2.5$ $(D\sqrt{q_z} \leq 5.3)$	$D\sqrt{q_z} > 2.5$ $(D\sqrt{q_z} > 5.3)$
< 0.1	2.0	1.2	0.8
0.1 to 0.29	1.8	1.3	0.9
0.3 to 0.7	1.6	1.5	1.1

Notes:

1. Signs with openings comprising 30% or more of the gross area are classified as open signs.
2. The calculation of the design wind forces shall be based on the area of all exposed members and elements projected on a plane normal to the wind direction. Forces shall be assumed to act parallel to the wind direction.
3. The area A_f consistent with these force coefficients is the solid area projected normal to the wind direction.
4. Notation:

   $\in$: ratio of solid area to gross area;

   D: diameter of a typical round member, in feet (meters);

   q_z: velocity pressure evaluated at height z above ground in pounds per square foot (N/m^2).

Other Structures – Method 2		All Heights
Figure 6-23	**Force Coefficients, C_f**	**Trussed Towers**
Open Structures		

Tower Cross Section	C_f
Square	$4.0\in^2 - 5.9\in + 4.0$
Triangle	$3.4\in^2 - 4.7\in + 3.4$

Notes:

1. For all wind directions considered, the area A_f consistent with the specified force coefficients shall be the solid area of a tower face projected on the plane of that face for the tower segment under consideration.
2. The specified force coefficients are for towers with structural angles or similar flat-sided members.
3. For towers containing rounded members, it is acceptable to multiply the specified force coefficients by the following factor when determining wind forces on such members:

   $0.51\in^2 + 0.57$, but not > 1.0
4. Wind forces shall be applied in the directions resulting in maximum member forces and reactions. For towers with square cross-sections, wind forces shall be multiplied by the following factor when the wind is directed along a tower diagonal:

   $1 + 0.75\in$, but not > 1.2
5. Wind forces on tower appurtenances such as ladders, conduits, lights, elevators, etc., shall be calculated using appropriate force coefficients for these elements.
6. Loads due to ice accretion as described in Section 11 shall be accounted for.
7. Notation:

   $\in$: ratio of solid area to gross area of one tower face for the segment under consideration.

Terrain Exposure Constants

Table 6-2

Exposure	α	z_g (ft)	$\hat{a}$	$\hat{b}$	$\bar{\alpha}$	$\bar{b}$	c	ℓ (ft)	$\bar{\epsilon}$	z_{min} (ft)*
B	7.0	1200	1/7	0.84	1/4.0	0.45	0.30	320	1/3.0	30
C	9.5	900	1/9.5	1.00	1/6.5	0.65	0.20	500	1/5.0	15
D	11.5	700	1/11.5	1.07	1/9.0	0.80	0.15	650	1/8.0	7

*z_{min} = minimum height used to ensure that the equivalent height $\bar{z}$ is greater of 0.6*h* or z_{min}. For buildings with $h \le z_{min}$, $\bar{z}$ shall be taken as z_{min}.

In metric

Exposure	α	z_g (m)	$\hat{a}$	$\hat{b}$	$\bar{\alpha}$	$\bar{b}$	c	ℓ (m)	$\bar{\epsilon}$	z_{min} (m)*
B	7.0	365.76	1/7	0.84	1/4.0	0.45	0.30	97.54	1/3.0	9.14
C	9.5	274.32	1/9.5	1.00	1/6.5	0.65	0.20	152.4	1/5.0	4.57
D	11.5	213.36	1/11.5	1.07	1/9.0	0.80	0.15	198.12	1/8.0	2.13

*z_{min} = minimum height used to ensure that the equivalent height $\bar{z}$ is greater of 0.6*h* or z_{min}. For buildings with $h \le z_{min}$, $\bar{z}$ shall be taken as z_{min}.

Velocity Pressure Exposure Coefficients, K_h and K_z

Table 6-3

Height above ground level, z		Exposure (Note 1)			
		B		C	D
ft	(m)	Case 1	Case 2	Cases 1 & 2	Cases 1 & 2
0-15	(0-4.6)	0.70	0.57	0.85	1.03
20	(6.1)	0.70	0.62	0.90	1.08
25	(7.6)	0.70	0.66	0.94	1.12
30	(9.1)	0.70	0.70	0.98	1.16
40	(12.2)	0.76	0.76	1.04	1.22
50	(15.2)	0.81	0.81	1.09	1.27
60	(18)	0.85	0.85	1.13	1.31
70	(21.3)	0.89	0.89	1.17	1.34
80	(24.4)	0.93	0.93	1.21	1.38
90	(27.4)	0.96	0.96	1.24	1.40
100	(30.5)	0.99	0.99	1.26	1.43
120	(36.6)	1.04	1.04	1.31	1.48
140	(42.7)	1.09	1.09	1.36	1.52
160	(48.8)	1.13	1.13	1.39	1.55
180	(54.9)	1.17	1.17	1.43	1.58
200	(61.0)	1.20	1.20	1.46	1.61
250	(76.2)	1.28	1.28	1.53	1.68
300	(91.4)	1.35	1.35	1.59	1.73
350	(106.7)	1.41	1.41	1.64	1.78
400	(121.9)	1.47	1.47	1.69	1.82
450	(137.2)	1.52	1.52	1.73	1.86
500	(152.4)	1.56	1.56	1.77	1.89

Notes:

1. **Case 1:** a. All components and cladding.
   b. Main wind force resisting system in low-rise buildings designed using Figure 6-10.

   Case 2: a. All main wind force resisting systems in buildings except those in low-rise buildings designed using Figure 6-10.
   b. All main wind force resisting systems in other structures.

2. The velocity pressure exposure coefficient K_z may be determined from the following formula:

   For 15 ft. $\le z \le z_g$ — $K_z = 2.01\,(z/z_g)^{2/\alpha}$

   For $z < 15$ ft. — $K_z = 2.01\,(15/z_g)^{2/\alpha}$

   Note: z shall not be taken less than 30 feet for Case 1 in exposure B.

3. α and z_g are tabulated in Table 6-2.
4. Linear interpolation for intermediate values of height z is acceptable.
5. Exposure categories are defined in 6.5.6.

Wind Directionality Factor, K_d

Table 6-4

Structure Type	Directionality Factor K_d*
Buildings	
Main Wind Force Resisting System	0.85
Components and Cladding	0.85
Arched Roofs	0.85
Chimneys, Tanks, and Similar Structures	
Square	0.90
Hexagonal	0.95
Round	0.95
Solid Signs	0.85
Open Signs and Lattice Framework	0.85
Trussed Towers	
Triangular, square, rectangular	0.85
All other cross sections	0.95

*Directionality Factor K_d has been calibrated with combinations of loads specified in Section 2. This factor shall only be applied when used in conjunction with load combinations specified in 2.3 and 2.4.

MNBC -2025 TECHNICAL WORKING GROUP (TWG-3)
Participants List

1	U Saw Htwe Zaw	Group Leader
2	U Saw Pyae Aung	Secretary
3	U Bo Bo Kyaw	Member
4	U Chan Aye	Member
5	Daw Chan Nyein Thu	Member
6	Ei Ei Phyu	Member
7	U Hla Naing	Member
8	Daw Hnin Shwe Yee	Member
9	Daw Htet Ei Ei Khin	Member
10	U Htin Aung	Member
11	Dr.Khin Aye Mon	Member
12	U Khin Maung Tint	Member
13	Dr.Khin Maung Zaw	Member
14	Daw Khin Myo Thet	Member
15	Dr.Kyaw Kyaw	Member
16	Daw Mya Mya Win	Member
17	Dr.Mya Sandar Win	Member
18	Daw Myat Mon Oo	Member
19	U Myint Thein	Member
20	U Myo Htet Kyaw	Member
21	U Myo Min Latt	Member
22	Dr.Ni Ni Moe Kyaw	Member
23	Daw Nilar Aye	Member
24	Daw Nu Nu Win	Member
25	Dr.Nyan Myint Kyaw	Member
26	U Nyunt Maung San	Member
27	Daw Phyo Kay Thi	Member
28	Daw San San Thwin	Member
29	U Shwe Kyaw Hla	Member
30	U Shwe Than Soe	Member
31	U Soe Myint	Member
32	U Soe Naing	Member
33	Daw Soe Soe Tin	Member
34	Dr.Su Su Kyi	Member
35	U Thein Zaw	Member
36	Dr.Thet Mon San	Member
37	Daw Thiri Soe	Member
38	U Thura Naing	Member
39	Dr.Toe Toe Win	Member
40	U Wai Phyo Lin	Member
41	U Wai Yar Aung	Member
42	Daw Win Mon Mon Lwin	Member
43	Daw Yi Kyi Pyar	Member
44	Daw Yin Min Htike	Member
45	Dr.Yu Maung	Member
46	Daw Yu Yu Swe	Member
47	U Zarli Chan	Member
48	U Zaw Zaw Aung	Member

MYANMAR NATIONAL BUILDING CODE 2025

PART 4

SOIL AND FOUNDATION

MYANMAR NATIONAL BUILDING CODE 2025

PART 4 SOIL AND FOUNDATION

TABLE OF CONTENTS

SECTION 4.9 DRIVEN PILE FOUNDATIONS

LIST OF TABLES

LIST OF FIGURES

SECTION 4.1
GENERAL

4.1.1 SCOPE

This section covers soil and foundation design for all buildings such as individual footings, combined footings, strip footings, rafts, piles and other foundation systems to ensure safety and serviceability without exceeding the permissible stresses of foundation material and the bearing capacity of the supporting soil. Some parameters related to seismic – resistant designs are also included.

Design of soil and foundation must be carried out based on the principles of suiting measures to local conditions, using local materials, protecting the environment and economizing resources. The design shall be painstakingly performed with comprehensive consideration given to type of structures, availability of materials and construction technology, and geotechnical investigation results.

4.1.2 REFERENCED CODES AND STANDARDS

The other codes and standards referenced in this PART shall be considered part of the requirements of this code to the prescribed extent of each reference. Version of referenced code or standard shall conform to IBC 2006 unless otherwise specified in this PART. Where conflicts occur between the requirements of this PART and referenced codes and standards, the requirements of this PART shall apply.

4.1.3 DESIGN

Allowable bearing pressures, allowable stresses and design formulae provided in this section shall be used with the allowable stress design load combinations specified in Structural Design Section 3.2.1. The quality and design of materials used structurally in excavations, footings and foundations shall conform to the requirements specified in this code (see Section on Structural Design, Concrete, Masonry and Steel). Safety during construction and the protection of adjacent public and private properties shall govern the design and construction of excavations and fills.

4.1.3.1 Foundation design for seismic overturning

Where the foundation is proportioned using the load combinations specified in Structural Design Section 3.4.2, and the computation of the seismic overturning moment is by the equivalent lateral-force method or the model analysis, the proportioning shall be in accordance with Section 3.4.2.

4.1.3.1.1 Reduction of foundation overturning

Overturning effects at the soil foundation interface are permitted to be reduced by 25 percent for foundations of structures that satisfy both of the following conditions:

a) The structure is designed in accordance with the Equivalent Lateral Force Analysis as set forth in Structural Design Section 3.4.2.
b) The structure is not an inverted pendulum or cantilevered column type structure.

Overturning effects at the soil-foundation interface are permitted to be reduced by 10 percent for foundations of structures designed in accordance with the modal analysis requirements of Structural Design Section 3.4.2.

4.1.4 LIQUEFACTION

Liquefaction potential of project area shall be evaluated by design professional using currently established procedures. Provisions against probable liquefaction effect shall be proposed by design professional to ensure safety and performance of project or as required by building authorities.

4.1.5 DEFINITIONS AND NOTATIONS

For the purpose of this PART, the following definitions and notations shall apply.

4.1.5.1 Definitions

Unless otherwise expressly stated, the following words and terms shall, for the purposes of this PART, have the meanings shown in this section. Where terms are not defined in this PART and are defined in other PARTS or referenced codes and standards, such terms shall have the meanings ascribed to them as in those PARTS or referenced codes and standards. Where terms are not defined through the methods authorized by this section, such terms shall have ordinarily accepted meanings such as the context implies.

BASEMENT. That portion of a building that is partly or completely below grade plane. A basement shall be considered as a story above grade plane where the finished surface of the floor above the basement is:

1. More than 6 feet (1829 mm) above grade plane; or
2. More than 12 feet (3658 mm) above the finished ground level at any point.

BUILDING. Any structure used or intended for supporting or sheltering any use or occupancy.

BUILDING OFFICIAL. The officer or other designated authority charged with the administration and enforcement of this code, or a duly authorized representative.

CONSTRUCTION DOCUMENTS. Written, graphic and pictorial documents prepared or assembled for describing the design, location and physical characteristics of the elements of a project necessary for obtaining a building permit.

GRADE PLANE. A reference plane representing the average of finished ground level adjoining the building at exterior walls. Where the finished ground level slopes away from the exterior walls, the reference plane shall be established by the lowest points within the area between the building and the lot line or, where the lot line is more than 6 feet (1829 mm) from the building, between the building and a point 6 feet (1829 mm) from the building.

LIGHT-FRAME CONSTRUCTION. A type of construction whose vertical and horizontal structural elements are primarily formed by a system of repetitive wood or light gage steel framing members.

REGISTERED DESIGN PROFESSIONAL. An individual who is registered or licensed to practice their respective design profession as defined by the statutory requirements of the professional registration laws of the jurisdiction in which the project is to be constructed.

STORY (also STOREY). That portion of a building included between the upper surface of a floor and the upper surface of the floor or roof next above. It is measured as the vertical distance from top to top of two successive tiers of beams or finished floor surfaces and, for the topmost story, from the top of the floor finish to the top of the ceiling joists or, where there is not a ceiling, to the top of the roof rafters.

STORY ABOVE GRADE PLANE. Any story having its finished floor surface entirely above grade plane, except that a basement shall be considered as a story above grade plane where the finished surface of the floor above the basement is:

1. More than 6 feet (1829 mm) above grade plane; or
2. More than 12 feet (3658 mm) above the finished ground level at any point.

4.1.5.1.1 Deep foundations

The following words and terms shall, for the purposes of sections related to pier and pile foundations, have the meanings shown herein.

FLEXURAL LENGTH. Flexural length is the length of the pile from the first point of zero lateral deflection to the underside of the pile cap or grade beam.

MICROPILES. Micropiles are 12-inch-diameter (305 mm) or less bored, grouted-in-place piles incorporating steel pipe (casing) and/or steel reinforcement.

PIER FOUNDATIONS. Pier foundations consist of isolated masonry or cast-in-place concrete structural elements extending into firm materials. Piers are relatively short in comparison to their width, with lengths less than or equal to 12 times the least horizontal dimension of the pier. Piers derive their load-carrying capacity through skin friction, through end bearing, or a combination of both.

Belled piers. Belled piers are cast-in-place concrete piers constructed with a base that is larger than the diameter of the remainder of the pier. The belled base is designed to increase the load-bearing area of the pier in end bearing.

PILE FOUNDATIONS. Pile foundations consist of concrete, wood or steel structural elements either driven into the ground or cast in place. Piles are relatively slender in comparison to their length, with lengths exceeding 12 times the least horizontal dimension. Piles derive their load-carrying capacity through skin friction, end bearing or a combination of both.

Augered uncased piles. Augered uncased piles are constructed by depositing concrete into an uncased augered hole, either during or after the withdrawal of the auger.

Caisson piles. Caisson piles are cast-in-place concrete piles extending into bedrock. The upper portion of a caisson pile consists of a cased pile that extends to the bedrock. The lower portion of the caisson pile consists of an uncased socket drilled into the bedrock.

Concrete-filled steel pipe and tube piles. Concrete-filled steel pipe and tube piles are constructed by driving a steel pipe or tube section into the soil and filling the pipe or tube section with concrete. The steel pipe or tube section is left in place during and after the deposition of the concrete.

Drilled uncased piles. Drilled uncased piles are constructed by depositing concrete into an uncased drilled hole which is supported by a steel casing or other approved method during installation process.

Driven uncased piles. Driven uncased piles are constructed by driving a steel shell into the soil to shore an unexcavated hole that is later filled with concrete. The steel casing is lifted out of the hole during the deposition of the concrete.

Enlarged base piles. Enlarged base piles are cast-in-place concrete piles constructed with a base that is larger than the diameter of the remainder of the pile. The enlarged base is designed to increase the load-bearing area of the pile in end bearing.

Steel-cased piles. Steel-cased piles are constructed by driving a steel shell into the soil to shore an unexcavated hole. The steel casing is left permanently in place and filled with concrete.

Timber piles. Timber piles are round, tapered timbers with the small (tip) end embedded into the soil.

SPECIAL INSPECTION. Inspection as herein required of the materials, installation, fabrication, erection or placement of components and connections requiring special expertise to ensure compliance with approved construction documents and referenced standards.

SPECIAL INSPECTION, CONTINUOUS. The full-time observation of work requiring special inspection by an approved special inspector who is present in the area where the work is being performed.

SPECIAL INSPECTION, PERIODIC. The part-time or intermittent observation of work requiring special inspection by an approved special inspector who is present in the area where the work has been or is being performed and at the completion of the work.

4.1.5.2 Notations

A_g *Pile cross-sectional area, square inches*

A_{ch} *Core area defined by spiral outside diameter*

A_{sh} *Cross-sectional area of transverse reinforcement*

CLSM *Controlled low- strength material*

E *Modulus of Elasticity*

f_c'	*Specified compressive strength of concrete*
f_{yh}	*Yield strength of spiral reinforcement*
f_{pc}	*Effective stress on the gross section*
F_y	*Minimum specified yield strength of steel*
F_b	*Bending at fiber stress*
F_v	*Longitudinal shear*
F_c	*Axial compression*
F_{cb}	*Axial compression when combined with bending*
$F_{c(per)}$	*Compression perpendicular to grain*
$F_{t(par)}$	*Tension parallel to grain*
$F_{t(per)}$	*Tension perpendicular to grain*
h_c	*Cross-sectional dimension of pile core measured center to center of hoop reinforcement*
P	*Axial load on pile, pounds*
SPT	*Standard Penetration Test*
s	*Spacing of transverse reinforcement measured along length of pile*
ρ_s	*Spiral reinforcement index (vol. spiral/vol. core)*

SECTION 4.2
GEOTECHNICAL INVESTIGATION

4.2.1 GENERAL

Geotechnical investigations shall be conducted in conformance with Sections 4.2.2 through 4.2.6. Where required by the building official, the classification and investigation of the soil shall be made by a registered design professional.

4.2.2 INVESTIGATIONS REQUIRED

The owner or applicant shall submit a geotechnical investigation to the building official where required in Sections 4.2.2.1 through 4.2.2.7.

Exception:

The building official need not require a geotechnical investigation where satisfactory data from adjacent areas is available that demonstrates an investigation is not necessary for any of the conditions in Sections 4.2.2.1 through 4.2.2.7.

4.2.2.1 Questionable soil

Where the classification, strength or compressibility of the soil are in doubt or where a load-bearing value superior to that specified in this code is claimed, the building official shall require that the necessary investigation be made. Such investigation shall comply with the provisions of Sections 4.2.4 through 4.2.6.

4.2.2.2 Expansive soils

In areas likely to have expansive soil, the building official shall require soil tests to determine where such soils do exist.

4.2.2.3 Ground-water table

A subsurface soil investigation shall be performed to determine whether the existing ground-water table is above or within 5 feet (1524 mm) below the elevation of the lowest floor level where such floor is located below the finished ground level adjacent to the foundation.

4.2.2.4 Pile and pier foundations

Pile and pier foundations shall be designed and installed on the basis of a foundation investigation and report as specified in Sections 4.2.4 through 4.2.6 and Section 4.8.2.2.

4.2.2.5 Rock strata

Where subsurface explorations at the project site indicate variations or doubtful characteristics in the structure of the rock upon which foundations are to be constructed, a sufficient number of borings shall be made to a depth of not less than 10 feet (3048 mm) below the level of the foundations to provide assurance of the soundness of the foundation bed and its load-bearing capacity. All necessary activities for this case shall be done in accordance with relevant engineering practices.

4.2.2.6 Seismic Design Category C

Where a structure is determined to be in Seismic Design Category C in accordance with Section 3.4, an investigation shall be conducted and shall include an evaluation of the following potential hazards resulting from earthquake motions: slope instability, liquefaction and surface rupture due to faulting or lateral spreading.

4.2.2.7 Seismic Design Category D, E or F

Where the structure is determined to be in Seismic Design Category D, E or F, in accordance with Section 3.4, the soils investigation requirements for Seismic Design Category C, given in Section 4.2.2.6, shall be met, in addition to the following. The investigation shall include:

1. A determination of lateral pressures on basement and retaining walls due to earthquake motions.
2. An assessment of potential consequences of any liquefaction and soil strength loss, including estimation of differential settlement, lateral movement or reduction in foundation soil-bearing capacity, and shall address mitigation measures. Such measures shall be given consideration in the design of the structure and can include but are not limited to ground stabilization, selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated displacements or any combination of these measures. The potential for liquefaction and soil strength loss shall be evaluated for site peak ground acceleration magnitudes and source characteristics consistent with the design earthquake ground motions. Peak ground acceleration shall be determined from a site-specific study taking into account soil amplification effects, as specified in Chapter 21 of ASCE 7-05.

   Exception:

   A site-specific study need not be performed, provided that peak ground acceleration equal to *SDS/2.5* is used, where SDS is determined in accordance with Section 3.4.1.4.

4.2.3 Soil classification

Where required, soils shall be classified in accordance with Section 4.2.3.1 or 4.2.3.2.

4.2.3.1 General

For the purposes of this chapter, the definition and classification of soil materials for use in Table 4.6.2 shall be in accordance with ASTM D 2487.

4.2.3.2 Expansive soils

Soils meeting all four of the following provisions shall be considered expansive, except that tests to show compliance with Items 1, 2 and 3 shall not be required if the test prescribed in Item 4 is conducted:

1. Plasticity index (PI) of 15 or greater, determined in accordance with ASTM D 4318.
2. More than 10 percent of the soil particles pass a No. 200 sieve (75 μm), determined in accordance with ASTM D 422.
3. More than 10 percent of the soil particles are less than 5 micrometers in size, determined in accordance with ASTM D 422.
4. Expansion index greater than 20, determined in accordance with ASTM D 4829.

4.2.4 Investigation

Soil classification shall be based on observation and any necessary tests of the materials disclosed by borings, test pits or other subsurface exploration made in appropriate locations. Additional studies shall be made as necessary to evaluate slope stability, soil strength, position and adequacy of load-bearing soil, the effect of moisture variation on soil bearing capacity, compressibility, liquefaction and expansiveness.

4.2.4.1 Exploratory boring

The scope of the soil investigation including the types of borings or soundings, the equipment used to drill and sample, the in-situ testing equipment and the laboratory testing program shall be determined by registered geotechnical professional and/or design professional in compliance with requirements of concerning building authorities.

4.2.4.2 Number and location of borings

Site investigation shall be carried out to sufficient extent to establish adequate information for the significant soil/rock strata and ground variation. Location of borings shall be determined by registered geotechnical professional and/or design professional. Number of borings shall be as follows:

1. Minimum of 2 borings for every project.
2. One boring for every 2500 sq-ft (or 250 sq-m) for built-over area ≤ 10,000 sq-ft (or 1,000 sq-m).
3. One additional boring for every extra 5,000 sq-ft (or 500 sq-m) for large area projects >10,000 sq-ft (or 1,000 sq-m).
4. Additional borings for irregular soil conditions as required by design professional and/or concerning building authority.

4.2.4.3 Depth of boring

Site investigation shall be carried out to sufficient depth to establish adequate information for the significant soil/ rock strata and ground variation.
Depth of boring shall meet the most critical condition of following criteria.

1. For shallow foundations, minimum depth of boring shall be larger value of 1.5 times lesser dimension of the shallow foundation or 30ft (10 m).
2. For deep foundations, minimum depth of boring shall be as follows:
    a) $\boldsymbol{20\,S^{0.7}}$ (ft.) or $\boldsymbol{6\,S^{0.7}}$ (m) where $\boldsymbol{S}$ = number of storeys including basements.
    b) The boring shall not be terminated until soil layer in which SPT refusal condition (ASTM D 1586) is encountered.
3. For any type of foundation in seismic design purposes (Seismic Design Categories A to F), minimum depth of boring shall be 100 ft. (or 30 m) to enable proper determination of Site Class in the proposed project.
4. Boreholes should penetrate more than 5 meters into hard stratum in which SPT refusal condition (ASTM D 1586) is observed or at least 5 times pile diameter beyond the intended founding level for deep foundations.
5. In all cases, depth of boring shall be decided by the requirements of design professional and/or building authority to provide adequate information for design purposes.

4.2.5 SOIL BORING AND SAMPLING

The soil boring and sampling procedure and apparatus shall be in accordance with generally accepted engineering practice. The registered design professional shall have a fully qualified representative on the site during all boring and sampling operations.

4.2.6 REPORTS

The soil classification and design load-bearing capacity shall be shown on the construction document. A written report of the investigation shall be submitted that includes, but need not be limited to, the following information:

1. A plot showing the location of test borings and/or excavations.
2. A complete record of the soil samples.

3. A complete record of the soil profile.
4. Elevation of the water table, if encountered.
5. Physical and mechanical properties of soil samples. Seismic-related properties and chemical properties shall be provided where required by design professional or building authorities.
6. Recommendations for foundation type and design criteria, including but not limited to: bearing capacity of natural or compacted soil; provisions to mitigate the effects of expansive soils; mitigation of the effects of liquefaction, differential settlement and varying soil strength; and the effects of adjacent loads.
7. Information related to pile and pier foundation in accordance with Section 4.8.2.2.
8. Special design and construction provisions for footings or foundations founded on expansive soils, as necessary.
9. Compacted fill material properties and testing in accordance with Section 4.3.5.

SECTION 4.3
EXCAVATION, GRADING AND FILL

4.3.1 EXCAVATIONS NEAR FOOTING OR FOUNDATIONS

Excavations for any purpose shall not remove lateral support from any footing or foundation without first underpinning or protecting the footing or foundation against settlement or lateral translation.

4.3.2 PLACEMENT OF BACKFILL

Excavations outside the foundation shall be backfilled with soil that is free of organic material, construction debris, cobbles and boulders or shall be backfilled with a controlled low-strength material (*CLSM*). The backfill shall be placed in lifts and compacted in a manner that does not damage the foundation, the waterproofing or the damp-proofing material.

4.3.3 SITE GRADING

The ground immediately adjacent to the foundation shall be sloped away from the building at a slope of not less than one unit vertical in 20 units horizontal (5 percent slope) for a minimum distance of 10 feet (3048 mm) measured perpendicular to the face of the wall. If physical obstruction or allotment boundaries prohibit 10 feet (3048mm) of horizontal distance, a 5 percent slope shall be provided to an approved alternative method of diverting water away from the foundation. Swales used for this purpose shall be sloped a minimum of 2 percent where located within 10 feet (3048 mm) of the building foundation. Impervious surfaces within 10 feet (3048 mm) of the building foundation shall be sloped a minimum of 2 percent away from the building.

4.3.4 GRADING AND FILLING IN FLOOD HAZARD AREAS

In flood hazard areas established in Section 3.2.5, grading and/or fill shall not be approved:-

1. Unless such fill is placed, compacted and sloped to minimize shifting, slumping and erosion during the rise and fall of flood water and, as applicable, wave action.
2. In floodways, unless it has been demonstrated through hydrologic and hydraulic analyses, performed by a registered design professional in accordance with standard engineering practice, that the proposed grading or fill, or both, will not result in increased flood levels during the occurrence of the design flood.
3. In flood hazard areas subject to high velocity wave action, unless such fill is conducted and/or placed to avoid diversion of water and waves toward any building or structure.
4. Where design flood elevations are specified but floodways have not been designated, unless it has been demonstrated that the cumulative effect of the proposed flood hazard area encroachment, when combined with all other existing and anticipated flood hazard area encroachments, will not increase the design flood elevation more than 1 foot (305 mm) at any point.

4.3.5 COMPACTED FILL MATERIAL

Where footings bear onto compacted fill material, the compacted fill shall comply with the provisions of an approved report, which shall contain the following:

1. Specifications for the preparation of the site prior to placement of compacted fill material.
2. Specifications for material to be used as compacted fill.
3. Test methods to be used to determine the maximum dry density and optimum moisture content of the material to be used as compacted fill.
4. Maximum allowable thickness of each lift of compacted fill material.
5. Field test methods for determining the in – place dry density of the compacted fill.

6. The minimum acceptable in-place dry density expressed as a percentage of the maximum dry density determined in accordance with Item 3.
7. The number and frequency of field tests required to determine compliance with Item 6.

Exception:

Compacted fill material less than 12 inches (305mm) in depth need not comply with an approved report, provided it has been compacted to a minimum of 90 percent Modified Proctor in accordance with ASTM D1557. The compaction shall be verified by a qualified inspector approved by the building official.

4.3.6 CONTROLLED LOW-STRENGTH MATERIAL

Where footings will bear on controlled low-strength material (*CLSM*), the *CLSM* shall comply with the provisions of an approved report, which shall contain the following:

1. Specifications for the preparation of the site prior to placement of the *CLSM*.
2. Specifications for the *CLSM*.
3. Laboratory or field test method(s) to be used to determine the compressive strength or bearing capacity of the *CLSM*.
4. Test methods for determining the acceptance of the *CLSM* in the field.
5. Number and frequency of field tests required to determine compliance with Item 4.

SECTION 4.4
DAMPPROOFING AND WATERPROOFING

4.4.1 GENERAL

Walls or portions that retain earth and enclose interior spaces and floors below grade shall be dampproofed and waterproofed in accordance with this section.

4.4.1.1 Story above grade plane

Where a basement is considered a story above graded plane and the basement floor and wall is partially below the finished ground level for 25 percent or more of the perimeter, the floor and walls shall be damp proofed and a foundation drain shall be installed. The foundation drain shall be installed around the portion of the perimeter where the basement floor is below ground level.

4.4.1.2 Under-floor space

Unless an approved drainage system is provided, the ground level of the under-floor space shall be as high as the outside finished ground level where the ground water table rises to within 6 inches (152 mm) of the ground level at the outside building perimeter, or that the surface water does not readily drain from the building site.

4.4.1.2.1 Flood hazard areas

For buildings and structures in flood hazard areas, the finished ground level of an under-floor space such as crawl space shall be equal to or higher than the outside finished ground level.

4.4.1.3 Ground-water control

The floor and walls shall be damp proofed, where the ground water table is lowered and maintained at an elevation not less than 6 inches (152 mm) below the bottom of the lowest floor. The design of the system to lower the ground-water table shall be based on accepted principles of engineering.

4.4.2 DAMP PROOFING

Floors and walls shall be damp proofed where the ground-water investigation indicates that a hydrostatic pressure will not occur.

4.4.2.1 Floors

Damp proofing materials for floors shall be installed between the floor and the base course, except where a separate floor is provided above a concrete slab. Damp proofing materials shall be used locally available materials or other approved methods or materials. Joints in the membrane shall be lapped and sealed in accordance with the manufacturer's installation instructions.

4.4.2.2 Walls

Damp proofing materials for walls shall be installed on the exterior surface of the wall, and shall extend from the top of the footing to above ground level. Damp proofing materials for walls shall be used locally available materials or other approved materials.

4.4.2.2.1 Surface preparation of walls

All the holes and recesses on the concrete walls shall be sealed by bituminous material or other approved methods or materials prior to the application of damp proofing materials. Unit masonry walls shall be parged on the exterior surface below ground level with not less than 0.375 inches (10 mm) of Portland cement. The parging shall be coved at the footing.

Exception:

Parging of unit masonry walls is not required where a material is approved for direct application to the masonry.

4.4.3 WATER PROOFING

Floors and walls shall be water proofed where the ground-water investigation indicates that a hydrostatic pressure condition exists, and design does not include a ground-water control system.

4.4.3.1 Floors

Concrete floors are required to be water proofed and designed and constructed to resist the hydrostatic pressures to which the floors will be subjected. Waterproofing shall be accomplished by placing a membrane of locally available materials or other approved methods or materials. Joints in the membrane shall be lapped and sealed in accordance with the manufacturer's installation instruction.

4.4.3.2 Walls

Concrete walls and masonry walls are required to be water proofed and shall be designed and constructed to withstand hydrostatic pressures and other lateral loads to which the wall be subjected. Water proofing shall be applied from the bottom of the wall to not less than 12 inches (305 mm) above the maximum elevation of the ground-water table. The remainder of the wall shall be damp proofed. Water proofing materials for walls shall be used locally available materials or other approved materials. Joints in the membrane shall be lapped and sealed in accordance with the manufacturer's installation instruction.

4.4.3.2.1 Surface preparation of walls

The walls shall be prepared prior to application of waterproofing materials on concrete or masonry walls.

4.4.3.3 Joints and penetrations

Joints in walls and floors, joints between the wall and floor and penetrations of the wall and floor shall be made water-tight utilizing approved methods and materials.

4.4.4 SUBSOIL DRAINAGE SYSTEM

Where a hydrostatic pressure condition does not exist, damp proofing shall be provided and a base shall be installed under the floor and a drain installed around the foundation perimeter. A subsoil drainage system designed and constructed in accordance with Section 4.4.1.3 shall be deemed adequate for lowering the ground-water table.

4.4.4.1 Floor base course.

Floors of basements, except as provided for in Section 4.4.1.1, shall be placed over a floor base course not less than 4 inches (102 mm) in thickness that consists of gravel or crushed stone containing not more than 10 percent of material that passes through a No. 4 (4.75 mm) sieve.

Exception:

Where a site is located in well-drained gravel or sand/gravel mixture soils, a floor base course is not required.

4.4.4.2 Foundation drain.

A drain shall be placed around the perimeter of a foundation that consists of gravel or crushed stone containing not more than 10-percent material that passes through a No. 4 (4.75 mm) sieve. The drain shall extend a minimum of 12 inches (305 mm) beyond the outside edge of the footing. The thickness shall be such that the bottom of the drain is not higher than the bottom of the base under the floor, and that the top of the drain is not less than 6 inches (152 mm) above the top of the footing. The top of the drain shall be covered with an approved filter membrane material. Where a drain tile or perforated pipe is used, the invert of the pipe or tile shall not be higher than the floor elevation. The top of joints or the top of perforations shall be protected with an approved filter membrane material. The pipe or tile shall be placed on not less than 2 inches (51 mm) of gravel or crushed stone complying with Section 4.4.4.1 and shall be covered with not less than 6 inches (152 mm) of the same material.

4.4.4.3 Drainage discharge.

The floor base and foundation perimeter drain shall discharge by gravity or mechanical means into an approved drainage system that complies with the requirements of concerning building authority.

Exception:

Where a site is located in well-drained gravel or sand/gravel mixture soils, a dedicated drainage system is not required.

SECTION 4.5
EARTH RETAINING SYSTEMS

4.5.1 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 safety factor of 1.5 against lateral sliding and overturning. The design shall be done by the registered design professional.

4.5.2 TEMPORARY EARTH RETAINING SYSTEMS

Temporary earth retaining system shall be designed to meet requirements of specific construction site. The system shall include all necessary supporting systems to ensure stability of excavation. The design shall be done by the registered design professional.

4.5.2.1 Temporary earth retaining systems intended to be integrated with permanent structure

When a temporary earth retaining system is intended to be integrated with permanent structure, the design of that system shall satisfy requirements of both temporary and permanent earth retaining structure. The design shall be done by the registered design professional.

SECTION 4.6
ALLOWABLE LOAD BEARING VALUES OF SOIL

4.6.1 GENERAL

Unless otherwise specified by concerning building authorities, **t**he presumptive load-bearing values provided in Table 4.6.1 shall be used with the allowable stress design load combinations specified in Section 3.2.

4.6.2 PRESUMPTIVE LOAD-BEARING VALUES

The maximum allowable foundation pressure, lateral pressure or lateral sliding-resistance values for supporting soils near the surface shall not exceed the values specified in Table 4.6.1 unless data to substantiate the use of a higher value are submitted and approved.

Presumptive load-bearing values shall apply to materials with similar physical characteristics and dispositions.

Mud, organic silt, organic clays, peat or unprepared fill shall not be assumed to have a presumptive load-bearing capacity unless data to substantiate the use of such a value are submitted.

Exception:

A presumptive load-bearing capacity is permitted to be used where the building official deems the load-bearing capacity of mud, organic silt or unprepared fill is adequate for the support of lightweight and temporary structures.

4.6.3 LATERAL SLIDING RESISTANCE

The resistance of structural walls to lateral sliding shall be calculated by combining the values derived from the lateral bearing and the lateral sliding resistance shown in Table 4.6.1 unless data to substantiate the use of higher values are submitted for approval.

For clay, sandy clay, silty clay and clayey silt, in no case shall the lateral sliding resistance exceed one-half the dead load.

4.6.3.1 Increases in allowable lateral sliding resistance

The resistance values derived from the table are permitted to be increased by the tabular value for each additional foot (305 mm) of depth to a maximum of 15 times the tabular value.

Isolated poles for uses such as flagpoles or signs and poles used to support buildings that are not adversely affected by a 0.5 inch (12.7 mm) motion at the ground surface due to short-term lateral loads are permitted to be designed using lateral-bearing values equal to two times the tabular values.

Table 4.6.1 Allowable foundation and lateral pressure

Class of Materials	Allowable Foundation Pressure (psf)	Lateral Bearing (psf/f below natural grade)	Lateral Sliding	
			Coefficient of Friction[a]	Resistance (psf)[b]
1. Crystalline bedrock	12000	120	0.70	–
2. Sedimentary and foliated rock	4000	400	0.35	–
3. Sandy gravel and/or gravel (GW and GP)	3000	200	0.35	–
4. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM and GC)	2000	150	0.25	–
5. Clay, sandy clay, silty clay, clayey silt, silt and sandy silt (CL, ML, MH and CH)	1500[c]	100	–	130

For SI: 1 pound per square foot = 0.0479 kPa,1 pound per square foot per foot = 0.157 kPa/m.

---

a. Coefficient to be multiplied by the dead load.

b. Lateral sliding resistance value to be multiplied by the contact area, as limited by Section 4.6.3.

c. Where the building official determines that in-place soils with an allowable bearing capacity of less than 1,500 psf are likely to be present at the site, the allowable bearing capacity shall be determined by a soil investigation.

SECTION 4.7
FOOTINGS AND FOUNDATIONS

4.7.1 GENERAL

Footings and foundations shall be designed and constructed in accordance with Sections 4.7.1 through 4.7.9. Footings and foundations shall be built on undisturbed soil, compacted fill material or CLSM. Compacted fill material shall be placed in accordance with Section 4.3.5. CLSM shall be placed in accordance with Section 4.3.6.

The top surface of footings shall be level. The bottom surface of footings is permitted to have a slope not exceeding one unit vertical in 10 units horizontal (10-percent slope). Footings shall be stepped where it is necessary to change the elevation of the top surface of the footing or where the surface of the ground slopes more than one unit vertical in 10 units horizontal (10-percent slope).

4.7.2 DEPTH OF FOOTINGS

The minimum depth of footings below the undisturbed ground surface shall be 24 inches (610 mm). Where applicable, the depth of footings shall also conform to Sections 4.7.2.1 through 4.7.2.3.

4.7.2.1 Frost protection

Except where otherwise protected from frost, foundation walls, piers and other permanent supports of buildings and structures shall be protected by one or more of the following methods:

1. Extending below the frost line of the locality;
2. Constructing in accordance with ASCE 32; or
3. Erecting on solid rock.

Exception:

Free-standing buildings meeting all of the following conditions shall not be required to be protected:

1. Classified in Occupancy Category I, in accordance with Section 3.1.3.5;
2. Area of 600 square feet (56 m^2) or less for light-frame construction or 400 square feet (37 m^2) or less for other than light-frame construction; and
3. Eave height of 10 feet (3048 mm) or less.

Footings shall not bear on frozen soil unless such frozen condition is of a permanent character.

4.7.2.2 Isolated footings

Footings shall be so located that the line drawn between the lower edges of adjoining footings shall not have a slope steeper than 30 degrees (0.52 rad) with the horizontal, unless the material supporting the higher footing is braced or retained or otherwise laterally supported in an approved manner or a greater slope has been properly established by engineering analysis.

4.7.2.3 Shifting or moving soils

Where it is known that the shallow subsoils are of a shifting or moving character, footings shall be carried to a sufficient depth to ensure stability.

4.7.3 FOOTINGS ON OR ADJACENT TO SLOPES

The placement of buildings and structures on or adjacent to slopes steeper than one unit vertical in three units horizontal (33.3-percent slope) shall conform to Sections 4.7.3.1 through 4.7.3.5.

4.7.3.1 Building clearance from ascending slopes

In general, buildings below slopes shall be set a sufficient distance from the slope to provide protection from slope drainage, erosion and shallow failures. Except as provided in Section 4.7.3.5 and Figure 4.7.1, the following criteria will be assumed to provide this protection. Where the existing slope is steeper than one unit vertical in one unit horizontal (100-percent slope), the toe of the slope shall be assumed to be at the intersection of a horizontal plane drawn from the top of the foundation and a plane drawn tangent to the slope at an angle of 45 degrees (0.79 rad) to the horizontal. (See Figure 4.7.1) Where a retaining wall is constructed at the toe of the slope, the height of the slope shall be measured from the top of the wall to the top of the slope.

4.7.3.2 Footing setback from descending slope surface

Footings on or adjacent to slope surfaces shall be founded in firm material with an embedment and set back from the slope surface sufficient to provide vertical and lateral support for the footing without detrimental settlement. Except as provided for in Section 4.7.3.5 and Figure 4.7.1, the following setback is deemed adequate to meet the criteria. Where the slope is steeper than 1 unit vertical in 1 unit horizontal (100-percent slope), the required setback shall be measured from an imaginary plane 45 degrees (0.79 rad) to the horizontal, projected upward from the toe of the slope. (See Figure 4.7.1)

4.7.3.3 Pools

The setback between pools regulated by this code and slopes shall be equal to one-half the building footing setback distance required by this section. That portion of the pool wall within a horizontal distance of 7 feet (2134 mm) from the top of the slope shall be capable of supporting the water in the pool without soil support.

4.7.3.4 Foundation elevation

On graded sites, the top of any exterior foundation shall extend above the elevation of the street gutter at point of discharge or the inlet of an approved drainage device a minimum of 12 inches (305 mm) plus 2 percent. Alternate elevations are permitted subject to the approval of the building official, provided it can be demonstrated that required drainage to the point of discharge and away from the structure is provided at all locations on the site.

4.7.3.5 Alternate setback and clearance

Alternate setbacks and clearances are permitted, subject to the approval of the building official. The building official is permitted to require an investigation and recommendation of a registered design professional to demonstrate that the intent of this section has been satisfied. Such an investigation shall include consideration of material, height of slope, slope gradient, load intensity and erosion characteristics of slope material.

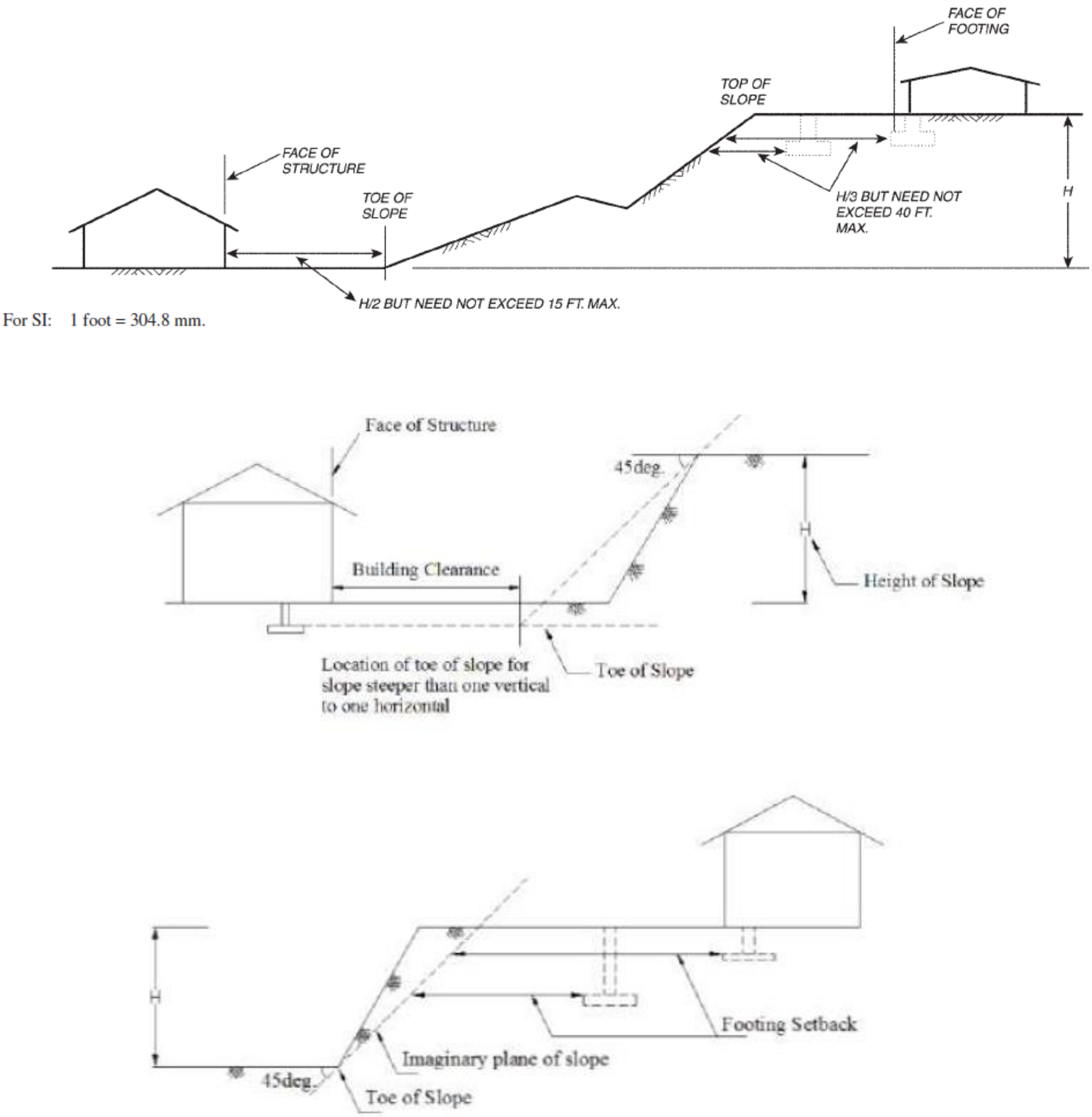


Figure 4.7.1 Foundation Clearances from Slopes

4.7.4 FOOTINGS

Footings shall be designed and constructed in accordance with Sections 4.7.4.1 through 4.7.4.3.

4.7.4.1 Design

Footings shall be so designed that the allowable bearing capacity of the soil is not exceeded, and that differential settlement is minimized. The minimum width of footings shall be 12 inches (305 mm).
Footings in areas with expansive soils shall be designed in accordance with the provisions of Section 4.7.8.

4.7.4.1.1 Design loads

Footings shall be designed for the most unfavorable effects due to the combinations of loads specified in Section 3.2.1. The dead load is permitted to include the weight of foundations, footings and overlying fill.

Reduced live loads, as specified in Sections 3.2.3.9 and 3.2.3.11, are permitted to be used in the design of footings.

4.7.4.1.2 Vibratory loads

Where machinery operations or other vibrations are transmitted through the foundation, consideration shall be given in the footing design to prevent detrimental disturbances of the soil.

4.7.4.2 Concrete footings

The design, materials and construction of concrete footings shall comply with Sections 4.7.4.2.1 through 4.7.4.2.6 and the provisions of Section 3.5.

Exception:

Where a specific design is not provided, concrete footings supporting walls of light-frame construction are permitted to be designed in accordance with Table 4.7.1.

Table 4.7.1 Footings supporting walls of light-frame construction[abcde]

Number of Floors supported by the Footing[f]	Width of Footing (inches)	Thickness of Footing (inches)
1	12	6
2	15	6
3	18	8[g]

For SI: 1inch= 25.4 mm, 1 foot = 304.8mm.

a. Depth of footings shall be in accordance with Section4.7.2.

b. The ground under the floor is permitted to be excavated to the elevation of the top of the footing.

c. Interior-stud-bearing walls are permitted to be supported by isolated footings. The footing width and length shall be twice the width shown in this table, and footings shall be spaced not more than 6 feet on center.

d. See Section3.5.4 for additional requirements for footings of structures assigned to Seismic Design Category C, D, E or F.

e. For thickness of foundation walls, see Section4.7.5.

f. Footings are permitted to support a roof in addition to the stipulated number of floors. Footings supporting roof only shall be as required for supporting one floor.

g. Plain concrete footings for low occupancies (Group R-3 in IBC 2006 Chapter 3) are permitted to be 6 inches thick.

4.7.4.2.1 Concrete strength

Concrete in footings shall have a specified compressive strength (*fc'*) of not less than 2,500 pounds per square inch (psi) (17 237 kPa) at 28 days.

4.7.4.2.2 Footing seismic ties

Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 3.4, individual spread footings founded on soil defined in Section 3.4.1.4.2 as Site Class E or F shall be interconnected by ties. Ties shall be capable of carrying, in tension or compression, a force equal to the product of the larger footing load times the seismic coefficient, S_{DS}, divided by 10 unless it is demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade.

4.7.4.2.3 Plain concrete footings

The edge thickness of plain concrete footings supporting walls of other than light-frame construction shall not be less than 8 inches (203 mm) where placed on soil.

Exception:

For plain concrete footings supporting low occupancies (Group R-3 in IBC 2006 Chapter 3), the edge thickness is permitted to be 6 inches (152 mm), provided that the footing does not extend beyond a distance greater than the thickness of the footing on either side of the supported wall.

4.7.4.2.4 Placement of concrete

Concrete footings shall not be placed through water unless a tremie or other method approved by the building official is used. Where placed under or in the presence of water, the concrete shall be deposited by approved means to ensure minimum segregation of the mix and negligible turbulence of the water.

4.7.4.2.5 Protection of concrete

Concrete footings shall be protected from freezing during depositing and for a period of not less than five days thereafter. Water shall not be allowed to flow through the deposited concrete.

4.7.4.2.6 Forming of concrete

Concrete footings are permitted to be cast against the earth where, in the opinion of the building official, soil conditions do not require forming. Where forming is required, it shall be in accordance with Chapter 6 of ACI 318-08.

4.7.4.3 Steel grillage footings

Grillage footings of structural steel shapes shall be separated with approved steel spacers and be entirely encased in concrete with at least 6 inches (152 mm) on the bottom and at least 4 inches (102 mm) at all other points. The spaces between the shapes shall be completely filled with concrete or cement grout.

4.7.5 FOUNDATION WALLS

Concrete and masonry foundation walls shall be designed in accordance with Section 3.5 and Chapter 21 of IBC 2006, respectively. Foundation walls that are laterally supported at the top and bottom and within the parameters of Table 4.7.2 through 4.7.6 are permitted to be designed and constructed in accordance with Section 4.7.5.1 through 4.7.5.5.

4.7.5.1 Foundation wall thickness

The minimum thickness of concrete and masonry foundation walls shall comply with Section 4.7.5.1.1 through 4.7.5.1.3.

4.7.5.1.1 Thickness at top of foundation wall

The thickness of foundation walls shall not be less than the thickness of the wall supported, except that foundation walls of at least 8-inch (203 mm) nominal width are permitted to support brick-veneered frame walls and 10-inch-wide (254 mm) cavity walls provided the requirements of Section 4.7.5.1.2 are met. Corbeling of masonry shall be in accordance with Section 2104.2 of IBC 2006. Where an 8-inch (203 mm) wall is corbeled, the top corbel shall not extend higher than the bottom of the floor framing and shall be a full course of headers at least 6 inches (152 mm) in length or the top course bed joint shall be tied to the vertical wall projection. The tie shall be W2.8 (4.8 mm) and spaced at a maximum horizontal distance of 36 inches (914 mm); the hollow space behind the corbelled masonry shall be filled with mortar or grout.

4.7.5.1.2 Thickness based on soil loads, unbalanced backfill height and wall height

The thickness of foundation walls shall comply with the requirements of Table 4.7.6 for concrete walls, Table 4.7.2 for plain masonry walls or Table 4.7.3, 4.7.4 or 4.7.5 for masonry walls with reinforcement. When using the tables, masonry shall be laid in running bond and the mortar shall be Type M or S (see IBC 2006 Chapter 21).

Unbalanced backfill height is the difference in height between the exterior finish ground level and the lower of the top of the concrete footing that supports the foundation wall or the interior finish ground level. Where an interior concrete slab on grade is provided and is in contact with the interior surface of the foundation wall, the unbalanced backfill height is permitted to be measured from the exterior finish ground level to the top of the interior concrete slab.

4.7.5.1.3 Rubble stone

Foundation walls of rough or random rubble stone shall not be less than 16 inches (406 mm) thick. Rubble stone shall not be used for foundations for structures in Seismic Design Category C, D, E or F.

4.7.5.2 Foundation wall materials

Concrete foundation walls constructed in accordance with Table 4.7.6 shall comply with Section 4.7.5.2.1. Masonry foundation walls constructed in accordance with Table 4.7.2, 4.7.3, 4.7.4 or 4.7.5 shall comply with Section 4.7.5.2.2.

4.7.5.2.1 Concrete foundation walls

Concrete foundation walls shall comply with the following:

1. The size and spacing of vertical reinforcement shown in Table 4.7.6 is based on the use of reinforcement with a minimum yield strength of 60,000 psi (414 MPa). Vertical reinforcement with a minimum yield strength of 40,000 psi (276 MPa) or 50,000 psi (345 MPa) is permitted, provided the same size bar is used and the spacing shown in the table is reduced by multiplying the spacing by 0.67 or 0.83, respectively.
2. Vertical reinforcement, when required, shall be placed nearest the inside face of the wall a distance, *d,* from the outside face (soil side) of the wall. The distance, *d,* is equal to the wall thickness, *t,* minus 1.25 inches (32 mm) plus one-half the bar diameter, d_b [$d = t - (1.25 + d_b/2)$]. The reinforcement shall be placed within a tolerance of ±3/8 inch (9.5 mm) where *d* is less than or equal to 8 inches (203 mm) or ±1/2 inch (2.7 mm) where *d* is greater than 8 inches (203 mm).

3. In lieu of the reinforcement shown in Table 4.7.6, smaller reinforcing bar sizes with closer spacings that provide an equivalent cross-sectional area of reinforcement per unit length of wall are permitted.
4. Concrete cover for reinforcement measured from the inside face of the wall shall not be less than 3/4 inch (19.1 mm). Concrete cover for reinforcement measured from the outside face of the wall shall not be less than 1.5 inches (38 mm) for No. 5 bars and smaller and not less than 2 inches (51 mm) for larger bars.
5. Concrete shall have a specified compressive strength, $f_c{}'$, of not less than 2,500 psi (17.2 MPa) at 28 days.
6. The unfactored axial load per linear foot of wall shall not exceed $1.2tf_c{}'$, where t is the specified wall thickness in inches.

4.7.5.2.2 Masonry foundation walls

Masonry foundation walls shall comply with the following:

1. Vertical reinforcement shall have a minimum yield strength of 60,000 psi (414 MPa).
2. The specified location of the reinforcement shall equal or exceed the effective depth distance, d, noted in Tables 4.7.3, 4.7.4 and 4.7.5 and shall be measured from the face of the exterior (soil) side of the wall to the center of the vertical reinforcement. The reinforcement shall be placed within the tolerances specified in ACI 530.1/ASCE 6/TMS 402, Article 3.4 B7 of the specified location.
3. Grout shall comply with Section 2103.12 of IBC 2006.
4. Concrete masonry units shall comply with ASTM C 90.
5. Clay masonry units shall comply with ASTM C 652 for hollow brick, except compliance with ASTM C 62 or ASTM C 216 is permitted when solid masonry units are installed in accordance with Table 4.7.2 for plain masonry.
6. Masonry units shall be installed with Type M or S mortar in accordance with Section 2103.8 of IBC 2006.
7. The unfactored axial load per linear foot of wall shall not exceed $1.2tf_m$ where t is the specified wall thickness in inches and f_m is the specified compressive strength of masonry in pounds per square inch.

4.7.5.3 Alternative foundation wall reinforcement

In lieu of the reinforcement provisions for masonry foundation walls in Table 4.7.3, 4.7.4 or 4.7.5, alternative reinforcing bar sizes and spacings having an equivalent cross-sectional area of reinforcement per linear foot (mm) of wall are permitted to be used, provided the spacing of reinforcement does not exceed 72 inches (1829 mm) and reinforcing bar sizes do not exceed No. 11.

4.7.5.4 Hollow masonry walls

At least 4 inches (102 mm) of solid masonry shall be provided at girder supports at the top of hollow masonry unit foundation walls.

4.7.5.5 Seismic requirements

Tables 4.7.2 through 4.7.6 shall be subject to the following limitations in Sections 4.7.5.5.1 and 4.7.5.5.2 based on the seismic design category assigned to the structure as defined in Section 3.4.

4.7.5.5.1 Seismic requirements for concrete foundation walls

Concrete foundation walls designed using Table 4.7.6 shall be subject to the following limitations:

1. Seismic Design Categories A and B

No additional seismic requirements, except provide not less than two No. 5 bars around window and door openings. Such bars shall extend at least 24 inches (610 mm) beyond the corners of the openings.

2. Seismic Design Categories C, D, E and F

Tables shall not be used except as allowed for plain concrete members in Section 3.5.4.1.18.

4.7.5.5.2 Seismic requirements for masonry foundation walls

Masonry foundation walls designed using Tables 4.7.2 through 4.7.5 shall be subject to the following limitations:

1. Seismic Design Categories A and B: No additional seismic requirements.
2. Seismic Design Category C: A design using Tables 4.7.2 through 4.7.5 is subject to the seismic requirements of Section 2106.4 of IBC 2006.
3. Seismic Design Category D: A design using Tables 4.7.3 through 4.7.5 is subject to the seismic requirements of Section 2106.5 of IBC 2006.
4. Seismic Design Categories E and F: A design using Tables 4.7.3 through 4.7.5 is subject to the seismic requirements of Section 2106.6 of IBC 2006.

Table 4.7.2 Plain Masonry Foundation Walls[abc]

Maximum Wall Height (feet)	Maximum Unbalanced Backfill Height[e] (feet)	Minimum Nominal Wall Thickness (inches)		
		Soil classes and lateral soil load[a] (psf per foot below natural grade)		
		GW, GP, SW and SP soils 30	GM, GC, SM, SM-SC and ML soils 45	SC, ML-CL and Inorganic CL soils 60
7	4 (or less)	8	8	8
	5	8	10	10
	6	10	12	10 (solid[c])
	7	12	10 (solid[c])	10 (solid[c])
8	4 (or less)	8	8	8
	5	8	10	12
	6	10	12	12 (solid[c])
	7	12	12 (solid[c])	Note d
	8	10 (solid[c])	12 (solid[c])	Note d
9	4 (or less)	8	8	8
	5	8	10	12
	6	12	12	12 (solid[c])
	7	12 (solid[c])	12 (solid[c])	Note d
	8	12 (solid[c])	Note d	Note d
	9	Note d	Note d	Note d

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot per foot = 0.157kPa/m.

a. For design lateral soil loads, see Section 3.2.2. Soil classes are in accordance with the Unified Soil Classification System and design lateral soil loads are for moist soil conditions without hydrostatic pressure.

b. Provisions for this table are based on construction requirements specified in Section 4.7.5.2.2.

c. Solid grouted hollow units or solid masonry units.

d. A design in compliance with Chapter 21 of IBC 2006 or reinforcement in accordance with Table 4.7.3 is required.

e. For height of unbalanced backfill, see Section 4.7.5.1.2.

Table 4.7.3 8-inch Masonry Foundation Walls with Reinforcement where d ≥ 5 inches[abc]

Maximum Wall Height (feet-inches)	Maximum Unbalanced Backfill Height[d] (feet-inches)	Vertical Reinforcement		
		Soil classes and lateral soil load[a] (psf per foot below natural grade)		
		GW, GP, SW and SP soils 30	GM, GC, SM, SM-SC and ML soils 45	SC, ML-CL and Inorganic CL soils 60
7-4	4-0 (or less)	#4 @ 48" o.c.	#4 @ 48" o.c.	#4 @ 48" o.c.
	5-0	#4 @ 48" o.c.	#4 @ 48" o.c.	#4 @ 48" o.c.
	6-0	#4 @ 48" o.c.	#5 @ 48" o.c.	#5 @ 48" o.c.
	7-4	#5 @ 48" o.c.	#6 @ 48" o.c.	#7 @ 48" o.c.
8-0	4-0 (or less)	#4 @ 48" o.c.	#4 @ 48" o.c.	#4 @ 48" o.c.
	5-0	#4 @ 48" o.c.	#4 @ 48" o.c.	#4 @ 48" o.c.
	6-0	#4 @ 48" o.c.	#5 @ 48" o.c.	#5 @ 48" o.c.
	7-0	#5 @ 48" o.c.	#6 @ 48" o.c.	#7 @ 48" o.c.
	8-0	#5 @ 48" o.c.	#6 @ 48" o.c.	#7 @ 48" o.c.
8-8	4-0 (or less)	#4 @ 48" o.c.	#4 @ 48" o.c.	#4 @ 48" o.c.
	5-0	#4 @ 48" o.c.	#4 @ 48" o.c.	#5 @ 48" o.c.
	6-0	#4 @ 48" o.c.	#5 @ 48" o.c.	#6 @ 48" o.c.
	7-0	#5 @ 48" o.c.	#6 @ 48" o.c.	#7 @ 48" o.c.
	8-8	#6 @ 48" o.c.	#7 @ 48" o.c.	#8 @ 48" o.c.
9-4	4-0 (or less)	#4 @ 48" o.c.	#4 @ 48" o.c.	#4 @ 48" o.c.
	5-0	#4 @ 48" o.c.	#4 @ 48" o.c.	#5 @ 48" o.c.
	6-0	#4 @ 48" o.c.	#5 @ 48" o.c.	#6 @ 48" o.c.
	7-0	#5 @ 48" o.c.	#6 @ 48" o.c.	#7 @ 48" o.c.
	8-0	#6 @ 48" o.c.	#7 @ 48" o.c.	#8 @ 48" o.c.
	9-4	#7 @ 48" o.c.	#8 @ 48" o.c.	#9 @ 48" o.c.
10-0	4-0 (or less)	#4 @ 48" o.c.	#4 @ 48" o.c.	#4 @ 48" o.c.
	5-0	#4 @ 48" o.c.	#4 @ 48" o.c.	#5 @ 48" o.c.
	6-0	#4 @ 48" o.c.	#5 @ 48" o.c.	#6 @ 48" o.c.
	7-0	#5 @ 48" o.c.	#6 @ 48" o.c.	#7 @ 48" o.c.
	8-0	#6 @ 48" o.c.	#7 @ 48" o.c.	#8 @ 48" o.c.
	9-0	#7 @ 48" o.c.	#8 @ 48" o.c.	#9 @ 48" o.c.
	10-0	#7 @ 48" o.c.	#9 @ 48" o.c.	#9 @ 48" o.c.

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot per foot = 0.157kPa/m.

a. For design lateral soil loads, see Section 3.2.2. Soil classes are in accordance with the Unified Soil Classification System and design lateral soil loads are for moist soil conditions without hydrostatic pressure.

b. Provisions for this table are based on construction requirements specified in Section 4.7.5.2.2.

c. For alternative reinforcement, see Section 4.7.5.3.

d. For height of unbalanced backfill, see Section 4.7.5.1.2.

Table 4.7.4 10-inch Masonry Foundation Walls with Reinforcement where d ≥ 6.75 inches[abc]

Maximum Wall Height (feet-inches)	Maximum Unbalanced Backfill Height[d] (feet-inches)	Vertical Reinforcement — Soil classes and lateral soil load[a] (psf per foot below natural grade)		
		GW, GP, SW and SP soils 30	GM, GC, SM, SM-SC and ML soils 45	SC, ML-CL and Inorganic CL soils 60
7-4	4-0 (or less)	#4 @ 56" o.c.	#4 @ 56" o.c.	#4 @ 56" o.c.
	5-0	#4 @ 56" o.c.	#4 @ 56" o.c.	#4 @ 56" o.c.
	6-0	#4 @ 56" o.c.	#4 @ 56" o.c.	#5 @ 56" o.c.
	7-4	#4 @ 56" o.c.	#5 @ 56" o.c.	#6 @ 56" o.c.
8-0	4-0 (or less)	#4 @ 56" o.c.	#4 @ 56" o.c.	#4 @ 56" o.c.
	5-0	#4 @ 56" o.c.	#4 @ 56" o.c.	#4 @ 56" o.c.
	6-0	#4 @ 56" o.c.	#4 @ 56" o.c.	#5 @ 56" o.c.
	7-0	#4 @ 56" o.c.	#5 @ 56" o.c.	#6 @ 56" o.c.
	8-0	#5 @ 56" o.c.	#6 @ 56" o.c.	#7 @ 56" o.c.
8-8	4-0 (or less)	#4 @ 56" o.c.	#4 @ 56" o.c.	#4 @ 56" o.c.
	5-0	#4 @ 56" o.c.	#4 @ 56" o.c.	#4 @ 56" o.c.
	6-0	#4 @ 56" o.c.	#4 @ 56" o.c.	#5 @ 56" o.c.
	7-0	#4 @ 56" o.c.	#5 @ 56" o.c.	#6 @ 56" o.c.
	8-8	#5 @ 56" o.c.	#7 @ 56" o.c.	#8 @ 56" o.c.
9-4	4-0 (or less)	#4 @ 56" o.c.	#4 @ 56" o.c.	#4 @ 56" o.c.
	5-0	#4 @ 56" o.c.	#4 @ 56" o.c.	#4 @ 56" o.c.
	6-0	#4 @ 56" o.c.	#5 @ 56" o.c.	#5 @ 56" o.c.
	7-0	#4 @ 56" o.c.	#5 @ 56" o.c.	#6 @ 56" o.c.
	8-0	#5 @ 56" o.c.	#6 @ 56" o.c.	#7 @ 56" o.c.
	9-4	#6 @ 56" o.c.	#7 @ 56" o.c.	#8 @ 56" o.c.
10-0	4-0 (or less)	#4 @ 56" o.c.	#4 @ 56" o.c.	#4 @ 56" o.c.
	5-0	#4 @ 56" o.c.	#4 @ 56" o.c.	#4 @ 56" o.c.
	6-0	#4 @ 56" o.c.	#5 @ 56" o.c.	#5 @ 56" o.c.
	7-0	#5 @ 56" o.c.	#6 @ 56" o.c.	#7 @ 56" o.c.
	8-0	#5 @ 56" o.c.	#7 @ 56" o.c.	#8 @ 56" o.c.
	9-0	#6 @ 56" o.c.	#7 @ 56" o.c.	#9 @ 56" o.c.
	10-0	#7 @ 56" o.c.	#8 @ 56" o.c.	#9 @ 56" o.c.

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot per foot = 0.157kPa/m.

a. For design lateral soil loads, see Section3.2.2. Soil classes are in accordance with the Unified Soil Classification System and design lateral soil loads are for moist soil conditions without hydrostatic pressure.

b. Provisions for this table are based on construction requirements specified in Section 4.7.5.2.2.

c. For alternative reinforcement, see Section 4.7.5.3.

d. For height of unbalanced fill, see Section 4.7.5.1.2.

Table 4.7.5 12-inch Masonry Foundation Walls with Reinforcement where d ≥ 8.75 inches[abc]

Maximum Wall Height (feet-inches)	Maximum Unbalanced Backfill Height[d] (feet-inches)	Vertical Reinforcement		
		Soil classes and lateral soil load[a] (psf per foot below natural grade)		
		GW, GP, SW and SP soils 30	GM, GC, SM, SM-SC and ML soils 45	SC, ML-CL and Inorganic CL soils 60
7-4	4-0 (or less)	#4 @ 72" o.c.	#4 @ 72" o.c.	#4 @ 72" o.c.
	5-0	#4 @ 72" o.c.	#4 @ 72" o.c.	#4 @ 72" o.c.
	6-0	#4 @ 72" o.c.	#4 @ 72" o.c.	#5 @ 72" o.c.
	7-4	#4 @ 72" o.c.	#5 @ 72" o.c.	#6 @ 72" o.c.
8-0	4-0 (or less)	#4 @ 72" o.c.	#4 @ 72" o.c.	#4 @ 72" o.c.
	5-0	#4 @ 72" o.c.	#4 @ 72" o.c.	#4 @ 72" o.c.
	6-0	#4 @ 72" o.c.	#4 @ 72" o.c.	#5 @ 72" o.c.
	7-0	#4 @ 72" o.c.	#5 @ 72" o.c.	#6 @ 72" o.c.
	8-0	#5 @ 72" o.c.	#6 @ 72" o.c.	#7 @ 72" o.c.
8-8	4-0 (or less)	#4 @ 72" o.c.	#4 @ 72" o.c.	#4 @ 72" o.c.
	5-0	#4 @ 72" o.c.	#4 @ 72" o.c.	#4 @ 72" o.c.
	6-0	#4 @ 72" o.c.	#4 @ 72" o.c.	#5 @ 72" o.c.
	7-0	#4 @ 72" o.c.	#5 @ 72" o.c.	#6 @ 72" o.c.
	8-8	#5 @ 72" o.c.	#7 @ 72" o.c.	#8 @ 72" o.c.
9-4	4-0 (or less)	#4 @ 72" o.c.	#4 @ 72" o.c.	#4 @ 72" o.c.
	5-0	#4 @ 72" o.c.	#4 @ 72" o.c.	#4 @ 72" o.c.
	6-0	#4 @ 72" o.c.	#5 @ 72" o.c.	#5 @ 72" o.c.
	7-0	#4 @ 72" o.c.	#5 @ 72" o.c.	#6 @ 72" o.c.
	8-0	#5 @ 72" o.c.	#6 @ 72" o.c.	#7 @ 72" o.c.
	9-4	#6 @ 72" o.c.	#7 @ 72" o.c.	#8 @ 72" o.c.
10-0	4-0 (or less)	#4 @ 72" o.c.	#4 @ 72" o.c.	#4 @ 72" o.c.
	5-0	#4 @ 72" o.c.	#4 @ 72" o.c.	#4 @ 72" o.c.
	6-0	#4 @ 72" o.c.	#5 @ 72" o.c.	#5 @ 72" o.c.
	7-0	#4 @ 72" o.c.	#6 @ 72" o.c.	#6 @ 72" o.c.
	8-0	#5 @ 72" o.c.	#6 @ 72" o.c.	#7 @ 72" o.c.
	9-0	#6 @ 72" o.c.	#7 @ 72" o.c.	#8 @ 72" o.c.
	10-0	#7 @ 72" o.c.	#8 @ 72" o.c.	#9 @ 72" o.c.

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot per foot = 0.157kPa/m.

---

a. For design lateral soil loads, see Section 3.2.2. Soil classes are in accordance with the Unified Soil Classification System and design lateral soil loads are for moist soil conditions without hydrostatic pressure.

b. Provisions for this table are based on construction requirements specified in Section 4.7.5.2.2.

c. For alternative reinforcement, see Section 4.7.5.3.

d. For height of unbalanced backfill, see Section 4.7.5.1.2.

Table 4.7.6 Concrete Foundation Walls[bc]

Maximum Wall Height (feet)	Maximum Unbalanced Backfill Height[e] (feet)	Vertical Reinforcement and Spacing (inches)								
		Design lateral soil load[a] (psf per foot of depth)								
		30			45			60		
		Minimum wall thickness (inches)								
		7.5	9.5	11.5	7.5	9.5	11.5	7.5	9.5	11.5
5	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
6	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
	6	PC	PC	PC	PC	PC	PC	PC	PC	PC
7	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
	6	PC	PC	PC	PC	PC	PC	#5@48"	PC	PC
	7	PC	PC	PC	#5@46"	PC	PC	#6@48"	PC	PC
8	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
	6	PC	PC	PC	PC	PC	PC	#5@43"	PC	PC
	7	PC	PC	PC	#5@41"	PC	PC	#6@43"	PC	PC
	8	#5@47"	PC	PC	#6@43"	PC	PC	#6@32"	#6@44"	PC
9	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
	6	PC	PC	PC	PC	PC	PC	#5@39"	PC	PC
	7	PC	PC	PC	#5@37"	PC	PC	#6@38"	#5@37"	PC
	8	#5@41"	PC	PC	#6@38"	#5@37"	PC	#7@39"	#6@39"	#4@48"
	9[d]	#6@46"	PC	PC	#7@41"	#6@41"	PC	#7@31"	#7@41"	#6@39"
10	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
	6	PC	PC	PC	PC	PC	PC	#5@37"	PC	PC
	7	PC	PC	PC	#6@48"	PC	PC	#6@35"	#6@48"	PC
	8	#5@38"	PC	PC	#7@47"	#6@47"	PC	#7@35"	#7@48"	#6@45"
	9[d]	#6@41"	#4@48"	PC	#7@37"	#7@48"	#4@48"	#6@22"	#7@37"	#7@47"
	10[d]	#7@45"	#6@45"	PC	#7@31"	#7@40"	#6@38"	#6@22"	#7@30"	#7@38"

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot = 0.157 kPa/m.

---

a. For design lateral soil loads for different classes of soil, see Section 3.2.2.

b. Provisions for this table are based on construction requirements specified in Section 4.7.5.2.1.

c. "PC" means plain concrete.

d. Where design lateral soil loads from Table 3.2.1 are used, the requirements for 30 and 45 psf per foot of depth are not applicable. See Section 3.2.2.

e. For height of unbalanced backfill, see Section 4.7.5.1.2.

4.7.5.6 Foundation wall drainage

Foundation walls shall be designed to support the weight of the full hydrostatic pressure of undrained backfill unless a drainage system is installed in accordance with Sections 4.4.4.2 and 4.4.4.3.

4.7.5.7 Pier and curtain wall foundations

Except in Seismic Design Categories D, E and F, pier and curtain wall foundations are permitted to be used to support light-frame construction not more than two stories in height, provided the following requirements are met:

1. All load-bearing walls shall be placed on continuous concrete footings bonded integrally with the exterior wall footings.
2. The minimum actual thickness of a load-bearing masonry wall shall not be less than 4 inches (102 mm) nominal or 3.625 inches (92 mm) actual thickness, and shall be bonded integrally with piers spaced 6 feet (1829 mm) on center.
3. Piers shall be constructed in accordance with Chapter 21 of IBC 2006 and the following:
   - 3.1. The unsupported height of the masonry piers shall not exceed 10 times their least dimension.
   - 3.2. Where structural clay tile or hollow concrete masonry units are used for piers supporting beams and girders, the cellular spaces shall be filled solidly with concrete or Type M or S mortar (see IBC 2006 Chapter 21).

   Exception:

   Unfilled hollow piers are permitted where the unsupported height of the pier is not more than four times its least dimension.

   - 3.3. Hollow piers shall be capped with 4 inches (102 mm) of solid masonry or concrete or the cavities of the top course shall be filled with concrete or grout.
4. The maximum height of a 4-inch (102 mm) load-bearing masonry foundation wall supporting wood frame walls and floors shall not be more than 4 feet (1219 mm) in height.
5. The unbalanced fill for 4-inch (102 mm) foundation walls shall not exceed 24 inches (610 mm) for solid masonry, nor 12 inches (305 mm) for hollow masonry.

4.7.6 DESIGNS EMPLOYING LATERAL BEARING

Designs to resist both axial and lateral loads employing posts or poles as columns embedded in earth or embedded in concrete footings in the earth shall conform to the requirements of Section 4.7.6.1 through 4.7.6.3.

4.7.6.1 Limitations

The design procedures outlined in this section are subject to the following limitations:

1. The frictional resistance for structural walls and slabs on silts and clays shall be limited to one-half of the normal force imposed on the soil by the weight of the footing or slab.
2. Posts embedded in earth shall not be used to provide lateral support for structural or nonstructural materials such as plaster, masonry or concrete unless bracing is provided that develops the limited deflection required.

4.7.6.2 Design criteria

The depth to resist lateral loads shall be determined by the design criteria established in Sections 4.7.6.2.1 through 4.7.6.2.3, or by other methods approved by the building official.

4.7.6.2.1 Nonconstrained

The following formula shall be used in determining the depth of embedment required to resist lateral loads where no constraint is provided at the ground surface, such as rigid floor or rigid ground surface pavement, and where no lateral constraint is provided above the ground surface, such as a structural diaphragm.

$$d = 0.5\{1 + [1 + (4.36\, h/A)]^{1/2}\} \qquad \textbf{(Equation 18-1)}$$

where:

A ***= 2.34 P/S_1 b.***

b ***= Diameter of round post or footing or diagonal dimension of square post or footing, feet (m).***

d ***= Depth of embedment in earth in feet (m) but not over 12 feet (3658 mm) for purpose of computing lateral pressure.***

h ***= Distance in feet (m) from ground surface to point of application of "P."***

P ***= Applied lateral force in pounds (kN).***

S_1 ***= Allowable lateral soil-bearing pressure as set forth in Section 4.6.3 based on a depth of one-third the depth of embedment in pounds per square foot (psf) (kPa).***

4.7.6.2.2 Constrained

The following formula shall be used to determine the depth of embedment required to resist lateral loads where constraint is provided at the ground surface, such as a rigid floor or pavement.

$$d^2 = 4.25(Ph/S_3\, b) \qquad \textbf{(Equation 18-2)}$$

or alternatively

$$d^2 = 4.25\,(M_g/S_3\, b) \qquad \textbf{(Equation 18-3)}$$

where:

M_g ***= Moment in the post at grade, in foot-pounds (kN-m).***

S_3 ***= Allowable lateral soil-bearing pressure as set forth in Section 4.6.3 based on a depth equal to the depth of embedment in pounds per square foot (kPa).***

4.7.6.2.3 Vertical load

The resistance to vertical loads shall be determined by the allowable soil-bearing pressure set forth in Table 4.6.2.

4.7.6.3 Backfill

The backfill in the annular space around columns not embedded in poured footings shall be by one of the following methods:

1. Backfill shall be of concrete with an ultimate strength of 2,000 psi (13.8 MPa) at 28 days. The hole shall not be less than 4 inches (102 mm) larger than the diameter of the column at its bottom or 4 inches (102 mm) larger than the diagonal dimension of a square or rectangular column.
2. Backfill shall be of clean sand. The sand shall be thoroughly compacted by tamping in layers not more than 8 inches (203 mm) in depth.
3. Backfill shall be of controlled low-strength material (CLSM).

4.7.7 DESIGN FOR EXPANSIVE SOILS

Footings or foundations for buildings and structures founded on expansive soils shall be designed in accordance with Section 4.7.7.1 or 4.7.7.2.

Footing or foundation design need not comply with Section 4.7.7.1 or 4.7.7.2 where the soil is removed in accordance with Section 4.7.7.3, nor where the building official approves stabilization of the soil in accordance with Section 4.7.7.4.

4.7.7.1 Foundations

Footings or foundations placed on or within the active zone of expansive soils shall be designed to resist differential volume changes and to prevent structural damage to the supported structure. Deflection and racking of the supported structure shall be limited to that which will not interfere with the usability and serviceability of the structure.
Foundations placed below where volume change occurs or below expansive soil shall comply with the following provisions:

1. Foundations extending into or penetrating expansive soils shall be designed to prevent uplift of the supported structure.
2. Foundations penetrating expansive soils shall be designed to resist forces exerted on the foundation due to soil volume changes or shall be isolated from the expansive soil.

4.7.7.2 Slab-on-ground foundations

Moments, shears and deflections for use in designing slab-on-ground, mat or raft foundations on expansive soils shall be determined in accordance with *WRI/CRSI Design of Slab-on-Ground Foundations* or *PTI Standard Requirements for Analysis of Shallow Concrete Foundations on Expansive Soils*. Using the moments, shears and deflections determined above, nonprestressed slabs-on-ground, mat or raft foundations on expansive soils shall be designed in accordance with *WRI/CRSI Design of Slab-on-Ground Foundations* and post-tensioned slab-on-ground, mat or raft foundations on expansive soils shall be designed in accordance with *PTI Standard Requirements for Design of Shallow Post-Tensioned Concrete Foundations on Expansive Soils*. It shall be permitted to analyze and design such slabs by other methods that account for soil-structure interaction, the deformed shape of the soil support, the plate or stiffened plate action of the slab as well as both center lift and edge lift conditions. Such alternative methods shall be rational and the basis for all aspects and parameters of the method shall be available for peer review.

4.7.7.3 Removal of expansive soil

Where expansive soil is removed in lieu of designing footings or foundations in accordance with Section 4.7.7.1 or 4.7.7.2, the soil shall be removed to a depth sufficient to ensure a constant moisture content in the remaining soil. Fill material shall not contain expansive soils and shall comply with Section 4.3.5 or 4.3.6.

Exception:

Expansive soil need not be removed to the depth of constant moisture, provided the confining pressure in the expansive soil created by the fill and supported structure exceeds the swell pressure.

4.7.7.4 Stabilization

Where the active zone of expansive soils is stabilized in lieu of designing footings or foundations in accordance with Section 4.7.7.1 or 4.7.7.2, the soil shall be stabilized by chemical, dewatering, presaturation or equivalent techniques.

4.7.8 SEISMIC REQUIREMENTS

See Section 3.5.4 for additional requirements for footings and foundations of structures assigned to Seismic Design Category C, D, E or F.

For structures assigned to Seismic Design Category D, E or F, provisions of ACI 318-08, Sections 21.12.1 to 21.12.3, shall apply when not in conflict with the provisions of Section 4.7.

Concrete shall have a specified compressive strength of not less than 3,000 psi (20.68 MPa) at 28 days.

Exceptions:

1. Temporary and low occupancies (Group U and R in IBC 2006 Chapter 3) of light-framed construction and two stories or less in height are permitted to use concrete with a specified compressive strength of not less than 2,500 psi (17.2 MPa) at 28 days.
2. Detached one- and two-family dwellings of light-frame construction and two stories or less in height are not required to comply with the provisions of ACI 318-08, Sections 21.12.1 to 21.12.3.

SECTION 4.8
GENERAL REQUIREMENTS FOR PIER AND PILE FOUNDATIONS

4.8.1 GENERAL

Pier and pile foundations shall be designed and installed on the basis of a geotechnical investigation as defined in section 4.2, unless sufficient data upon which to base the design and installation is available. The investigation and report provisions of Section 4.2 shall be expanded to include, but not be limited to, the following:

1. Recommended pier or pile types and installed capacities.
2. Recommended center-to-center spacing of piers or piles.
3. Driving criteria.
4. Installation procedures.
5. Field inspection and reporting procedures (to include procedures for verification of the installed bearing capacity where required).
6. Pier or pile load test requirements.
7. Durability of pier or pile materials.
8. Designation of bearing stratum or strata.
9. Reductions for group action, where necessary.

4.8.2 SPECIAL TYPES OF PILES

The use of types of piles not specifically mentioned herein is permitted, subject to the approval of the building official, upon the submission of acceptable test data, calculations and other information relating to the structural properties and load capacity of such piles. The allowable stresses shall not in any case exceed the limitations specified herein.

4.8.3 PILE CAPS

Pile caps shall be of reinforced concrete and shall include all elements to which piles are connected, including grade beams and mats. The soil immediately below the pile cap shall not be considered as carrying any vertical load. The tops of piles shall be embedded not less than 3 inches (76 mm) into pile caps and the caps shall extend at least 4 inches (102 mm) beyond the edges of piles. The tops of piles shall be cut back to sound material before capping.

4.8.4 STABILITY

Piers or piles shall be braced to provide lateral stability in all directions. Three or more piles connected by a rigid cap shall be considered braced, provided that the piles are located in radial directions from the centroid of the group not less than 60 degrees apart. A two-pile group in a rigid cap shall be considered to be braced along the axis connecting the two piles. Methods used to brace piers or piles shall be subject to the approval of the building official. Piles supporting walls shall be driven alternately in lines spaced at least 1 foot (305 mm) apart and located symmetrically under the center of gravity of the wall load carried, unless effective measures are taken to provide for eccentricity and lateral forces, or the wall piles are adequately braced to provide for lateral stability. A single row of piles without lateral bracing is permitted for one- and two-family dwellings and lightweight construction not exceeding two stories or 35 feet (10668 mm) in height, provided the centers of the piles are located within the width of the foundation wall.

4.8.5 STRUCTURAL INTEGRITY

Piers or piles shall be installed in such a manner and sequence as to prevent distortion or damage that may adversely affect the structural integrity of piles being installed or already in place.

4.8.6 SPLICES

Splices shall be constructed so as to provide and maintain true alignment and position of the component parts of the pier or pile during installation and subsequent thereto and shall be of adequate strength to transmit the vertical and lateral loads and moments occurring at the location of the splice during driving and under service loading. Splices shall develop not less than 50 percent of the least capacity of the pier or pile in bending. In addition, splices occurring in the upper 10 feet (3048 mm) of the embedded portion of the pier or pile shall be capable of resisting at allowable working stresses the moment and shear that would result from an assumed eccentricity of the pier or pile load of 3 inches (76 mm), or the pier or pile shall be braced in accordance with Section 4.8.4 to other piers or piles that do not have splices in the upper 10 feet (3048 mm) of embedment.

4.8.7 ALLOWABLE PIER OR PILE LOADS

4.8.7.1 Determination of allowable loads

The allowable axial and lateral loads on piers or piles shall be determined by an approved formula, load tests or method of analysis.

4.8.7.2 Driving criteria

Allowable compressive load on any pile shall be determined by the application of an approved driving formula. Allowable loads shall be verified by load tests in accordance with Section 4.8.7.3. The formula or wave equation load shall be determined for gravity-drop or power-actuated hammers and the hammer energy used shall be the maximum consistent with the size, strength and weight of the driven piles. The introduction of fresh hammer cushion or pile cushion material just prior to final penetration is not permitted.

4.8.7.3 Load tests

Where design compressive loads per pier or pile are greater than those permitted by Section 4.8.9 or where the design load for any pier or pile foundation is in doubt or where required by the building official, control test piers or piles shall be tested in accordance with ASTM D 1143-07 or ASTM D 4945-08. At least one pier or pile shall be test loaded in each area of uniform subsoil conditions. Where required by the building official, additional piers or piles shall be load tested where necessary to establish the safe design capacity. The resulting allowable loads shall not be more than one-half of the ultimate axial load capacity of the test pier or pile as assessed by one of the published methods with consideration for the test type, duration and subsoil. The ultimate axial load capacity shall be determined by a registered design professional with consideration given to tolerable total and differential settlements at design load in accordance with settlement analysis. In subsequent installation of the balance of foundation piles, all piles shall be deemed to have a supporting capacity equal to the control pile where such piles are of the same type, size and relative length as the test pile; are installed using the same or comparable methods and equipment as the test pile; are installed in similar subsoil conditions as the test pile; and, for driven piles, where the rate of penetration (e.g., net displacement per blow) of such piles is equal to or less than that of the test pile driven with the same hammer through a comparable driving distance.

4.8.7.3.1 Load test evaluation

It shall be permitted to evaluate pile load tests with any of the currently recognized methods in geotechnical engineering.

4.8.7.3.2 Non-destructive testing

For quality assurance of concrete piles, non-destructive integrity test may be carried out prior to construction of cap beam or caps.

4.8.7.4 Allowable frictional resistance

The assumed frictional resistance developed by any pier or uncased cast-in-place pile shall not exceed one-sixth of the bearing value of the soil material as set forth in Table 4.6.1, up to a maximum of 500 psf (24 kPa), unless a greater value is allowed by the building official after a soil investigation is submitted or a greater value is substantiated by a load test.

4.8.7.5 Uplift capacity

Where required by the design, the uplift capacity of a single pier or pile shall be determined by an approved method of analysis based on a minimum factor of safety of three or by load tests conducted in accordance with ASTM D 3689. The maximum allowable uplift load shall not exceed the ultimate load capacity as determined in Section 4.8.7.3 divided by a factor of safety of two. For pile groups subjected to uplift, the allowable working uplift load for the group shall be the lesser of:

1. The proposed individual pile uplift working load times the number of piles in the group.
2. Two-thirds of the effective weight of the pile group and the soil contained within a block defined by the perimeter of the group and the length of the pile.

4.8.7.6 Load-bearing capacity

Piers, individual piles and groups of piles shall develop ultimate load capacities of at least twice the design working loads in the designated load-bearing layers. Analysis shall show that no soil layer underlying the designated load-bearing layers causes the load-bearing capacity safety factor to be less than two.

4.8.7.7 Bent piers or piles

The load-bearing capacity of piers or piles discovered to have a sharp or sweeping bend shall be determined by an approved method of analysis or by load testing a representative pier or pile.

4.8.7.8 Overloads on piers or piles

The maximum compressive load on any pier or pile due to mislocation shall not exceed 110 percent of the allowable design load.

4.8.8 LATERAL SUPPORT

4.8.8.1 General

Any soil other than fluid soil shall be deemed to afford sufficient lateral support to the pier or pile to prevent buckling and to permit the design of the pier or pile in accordance with accepted engineering practice and the applicable provisions of this code.

4.8.8.2 Unbraced piles

Piles standing unbraced in air, water or in fluid soils shall be designed as columns in accordance with the provisions of this code. Such piles driven into firm ground can be considered fixed and laterally supported at 5 feet (1524 mm) below the ground surface and in soft material at 10 feet (3048 mm) below the ground surface unless otherwise prescribed by the building official after a foundation investigation by an approved agency.

4.8.8.3 Allowable lateral load

Where required by the design, the lateral load capacity of a pier, a single pile or a pile group shall be determined by an approved method of analysis or by lateral load tests to at least twice the proposed design working load. The resulting allowable load shall not be more than one-half of that test load that produces a gross lateral movement of 1 inch (25.4 mm) at the ground surface.

4.8.9 USE OF HIGHER ALLOWABLE PIER OR PILE STRESSES

Allowable stresses greater than those specified for piers or for each pile type are permitted where supporting data justifying such higher stresses is filed with the building official. Such substantiating data shall include:

1. A soils investigation in accordance with Section 4.2.
2. Pier or pile load tests in accordance with Section 4.8.7.3, regardless of the load supported by the pier or pile.

The design and installation of the pier or pile foundation shall be under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and pier or pile foundations who shall certify to the building official that the piers or piles as installed satisfy the design criteria.

4.8.10 PILES IN SUBSIDING AREAS

Where piles are installed through subsiding fills or other subsiding strata and derive support from underlying firmer materials, consideration shall be given to the downward frictional forces that may be imposed on the piles by the subsiding upper strata. Where the influence of subsiding fills is considered as imposing loads on the pile, the allowable stresses specified in respective sections are permitted to be increased where satisfactory substantiating data are submitted. All necessary site investigation activities and installation activities for piles in subsiding areas shall be carried out under supervision of a registered design professional. All design activities for piles in subsiding areas shall be carried out by a registered design professional.

4.8.11 NEGATIVE SKIN FRICTION OR DOWN DRAG FORCE

When a soil stratum, through which a pile shaft has penetrated into an underlying hard stratum, compresses as a result of either its being unconsolidated or its being under a newly placed fill or as a result of re-moulding during driving of the pile, a drag down force is generated along the pile shaft up to a point in depth where the surrounding soil does not move downwards relative to the pile shaft. Recognition of the existence of such a phenomenon shall be made and a suitable reduction shall be made to the allowable load, where appropriate.

4.8.12 SETTLEMENT ANALYSIS

The settlement of piers, individual piles or groups of piles shall be estimated based on approved methods of analysis. The predicted settlement shall cause neither harmful distortion of, nor instability in, the structure, nor cause any stresses to exceed allowable values.

4.8.13 PRE-EXCAVATION

The use of jetting, auguring or other methods of pre-excavation shall be subject to the approval of the building official. Where permitted, pre-excavation shall be carried out in the same manner as used for piers or piles subject to load tests and in such a manner that will not impair the carrying capacity of the piers or piles already in place or damage adjacent structures. Pile tips shall be driven below the pre-excavated depth until the required resistance or penetration is obtained.

4.8.14 INSTALLATION SEQUENCE

Piles shall be installed in such sequence as to avoid compaction of the surrounding soil to the extent that other piles cannot be installed properly, and to prevent ground movements that are capable of damaging adjacent structures.

4.8.15 USE OF VIBRATORY DRIVERS

Vibratory drivers shall only be used to install piles where the pile load capacity is verified by load tests in accordance with Section 4.8.7.3. The installation of production piles shall be controlled according to power consumption, rate of penetration or other approved means that ensure pile capacities equal or exceed those of the test piles.

4.8.16 PILE DRIVABILITY

Pile cross sections shall be of sufficient size and strength to withstand driving stresses without damage to the pile, and to provide sufficient stiffness to transmit the required driving forces.

4.8.17 PROTECTION OF PILE MATERIALS

Where boring records or site conditions indicate possible deleterious action on pier or pile materials because of soil constituents, changing water levels or other factors, the pier or pile materials shall be adequately protected by materials, methods or processes approved by the building official. Protective materials shall be applied to the piles so as not to be rendered ineffective by driving. The effectiveness of such protective measures for the particular purpose shall have been thoroughly established by satisfactory service records or other evidence.

4.8.18 USE OF EXISTING PIERS OR PILES

Piers or piles left in place where a structure has been demolished shall not be used for the support of new construction unless satisfactory evidence is submitted to the building official, which indicates that the piers or piles are sound and meet the requirements of this code. Such piers or piles shall be load tested or redriven to verify their capacities. The design load applied to such piers or piles shall be the lowest allowable load as determined by tests or re-driving data.

4.8.19 HEAVED PILES

Piles that have heaved during the driving of adjacent piles shall be redriven as necessary to develop the required capacity and penetration, or the capacity of the pile shall be verified by load tests in accordance with Section 4.8.7.3.

4.8.20 IDENTIFICATION

Pier or pile materials shall be identified for conformity to the specified grade with this identity maintained continuously from the point of manufacture to the point of installation or shall be tested by an approved agency to determine conformity to the specified grade. The approved agency shall furnish an affidavit of compliance to the building official.

4.8.21 PIER OR PILE LOCATION PLAN

A plan showing the location and designation of piers or piles by an identification system shall be filed with the building official prior to installation of such piers or piles. Detailed records for piers or individual piles shall bear an identification corresponding to that shown on the plan.

4.8.22 SPACING OF PILES

The center to center spacing of a pile is considered from two aspects as follows:

a) Practical aspects of installing the piles; and
b) The nature of the load transfer to the soil and possible reduction in bearing capacity of a group of piles thereby.

In the case of piles founded on a very hard stratum and deriving their capacity mainly from end bearing, the spacing will be governed by the competency of the end bearing strata. The minimum spacing in such cases shall be 2.5 times the diameter of the shaft. In case of piles resting on rock, a spacing of two times the diameter may be adopted. Piles deriving their bearing capacity mainly from friction shall be sufficiently apart to ensure that the zones of soil from which the piles derive their support do not overlap to such an extent that their bearing values are reduced. Generally, the spacing in such cases shall not be less than three times the diameter of the shaft. In the case of loose sand or filling, closer spacing than in dense sand may be possible, in driven piles since displacement during the piling may be absorbed by vertical and horizontal compaction of the strata. The minimum spacing in such strata may be two times the diameter of the shaft.

4.8.23 SPECIAL INSPECTION

4.8.23.1 Pier foundations

Special inspections shall be performed during installation and testing of pier foundations as required by Table 4.8.1. The approved soils report, required by Section 4.2, and the documents prepared by the registered design professional in responsible charge shall be used to determine compliance.

4.8.23.2 Pile foundations

Special inspections shall be performed during installation and testing of pile foundations as required by Table 4.8.2. The approved soils report, required by Section 4.2, and the documents prepared by the registered design professional in responsible charge shall be used to determine compliance.

4.8.24 SEISMIC DESIGN OF PIERS OR PILES

4.8.24.1 Seismic Design Category C

Where a structure is assigned to Seismic Design Category C in accordance with Section 3.4 of Part 3 (Structural Design), the following shall apply. Individual pile caps, piers or piles shall be interconnected by ties. Ties shall be capable of carrying, in tension and compression, a force equal to the product of the larger pile cap or column load times the seismic coefficient, S_{DS}, divided by 10 unless it can be demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade, reinforced concrete slabs on grade, confinement by competent rock, hard cohesive soils or very dense granular soils.

Exception:

Piers supporting foundation walls, isolated interior posts detailed so the pier is not subject to lateral loads, lightly loaded exterior decks and patios of temporary and low occupancies (Group U and R-3 in IBC 2006 Chapter 3) not exceeding two stories of light-frame construction, are not subject to interconnection if it can be shown the soils are of adequate stiffness, subject to the approval of the building official.

4.8.24.1.1 Connection to pile cap

Concrete piles and concrete-filled steel pipe piles shall be connected to the pile cap by embedding the pile reinforcement or field-placed dowels anchored in the concrete pile in the pile cap for a distance equal to the development length. For deformed bars, the development length is the full development length for compression or tension, in the case of uplift, without reduction in length for excess area. Alternative measures for laterally confining concrete and maintaining toughness and ductile-like behavior at the top of the pile will be permitted provided the design is such that any hinging occurs in the confined region. Ends of hoops, spirals and ties shall be terminated with seismic hooks, turned into the confined concrete core. The minimum transverse steel ratio for confinement shall not be less than one-half of that required for columns. For resistance to uplift forces, anchorage of steel pipe (round HSS sections), concrete-filled steel pipe or H-piles to the pile cap shall be made by means other than concrete bond to the bare steel section.

Exception:

Anchorage of concrete-filled steel pipe piles is permitted to be accomplished using deformed bars developed into the concrete portion of the pile. Splices of pile segments shall develop the full strength of the pile, but the splice need not develop the nominal strength of the pile in tension, shear and bending when it has been designed to resist axial and shear forces and moments from the load combinations of Structural Design Section 3.2.

4.8.24.1.2 Design details

Pier or pile moments, shears and lateral deflections used for design shall be established considering the nonlinear interaction of the shaft and soil, as recommended by a registered design professional. Where the ratio of the depth of embedment of the pile-to-pile diameter or width is less than or equal to six, the pile may be assumed to be rigid. Pile group effects from soil on lateral pile nominal strength shall be included where pile center-to-center spacing in the direction of lateral force is less than eight pile diameters. Pile group effects on vertical nominal strength shall be included where pile center- to-center spacing is less than three pile diameters. The pile uplift soil nominal strength shall be taken as the pile uplift strength as limited by the frictional force developed between the soil and the pile. Where a minimum length for reinforcement or the extent of closely spaced confinement reinforcement is specified at the top of the pier or pile, provisions shall be made so that those specified lengths or extents are maintained after pier or pile cutoff.

4.8.24.2 Seismic Design Category D, E or F

Where a structure is assigned to Seismic Design Category D, E or F in accordance with Structural Design Section 3.4, the requirements for Seismic Design Category C given in Section 4.8.24.1 shall be met, in addition to the following. Provisions of ACI 318-08, Section 21.12.4, shall apply when not in conflict with the provisions of Sections 4.8, 4.9, 4.10, 4.11 and 4.12. Concrete shall have a specified compressive strength of not less than 3,000 psi (20.68 MPa) at 28 days.

Exceptions:

1. Temporary and low occupancies (Group U or R in IBC 2006 Chapter 3) of light-framed construction and two stories or less in height are permitted to use concrete with a specified compressive strength of not less than 2,500 psi (17.2MPa) at 28 days.
2. Detached one- and two-family dwellings of light-frame construction and two stories or less in height are not required to comply with the provisions of ACI 318-08, Section 21.12.4.
3. Section 21.12.4.4 (a) of ACI 318-08 need not apply to concrete piles.

4.8.24.2.1 Design details for piers, piles and grade beams

Piers or piles shall be designed and constructed to withstand maximum imposed curvatures from earthquake ground motions and structure response. Curvatures shall include free-field soil strains modified for soil-pile-structure interaction coupled with pier or pile deformations induced by lateral pier or pile resistance to structure seismic forces. Concrete piers or piles on Site Class E or F sites, as determined in Structural Design Section 3.4, shall be designed and detailed in accordance with Sections 21.6.4.2, 21.6.4.3 and 21.6.4.4 of ACI 318-08 within seven pile diameters of the pile cap and the interfaces of soft to medium stiff clay or liquefiable strata. For precast prestressed concrete piles, detailing provisions as given in Section 4.9.2.3.2.1 and 4.9.2.3.2.2 shall apply. Grade beams shall be designed as beams in accordance with ACI 318-08, Chapter 21. When grade beams have the capacity to resist the forces from the load combinations in Structural Design Section 3.2.1.5, they need not conform to ACI 318-08, Chapter 21.

4.8.24.2.2 Connection to pile cap

For piles required to resist uplift forces or provide rotational restraint, design of anchorage of piles into the pile cap shall be provided considering the combined effect of axial forces due to uplift and bending moments due to fixity to the pile cap. Anchorage shall develop a minimum of 25 percent of the strength of the pile in tension. Anchorage into the pile cap shall be capable of developing the following:

1. In the case of uplift, the lesser of the nominal tensile strength of the longitudinal reinforcement in a concrete pile, or the nominal tensile strength of a steel pile, or the pile uplift soil nominal strength factored by 1.3 or the axial tension force resulting from the load combinations of Structural Design Section 3.2.
2. In the case of rotational restraint, the lesser of the axial and shear forces, and moments resulting from the load combinations of Structural Design Section 3.2 or development of the full axial, bending and shear nominal strength of the pile.

4.8.24.2.3 Flexural strength

Where the vertical lateral-force-resisting elements are columns, the grade beam or pile cap flexural strengths shall exceed the column flexural strength. The connection between batter piles and grade beams or pile caps shall be designed to resist the nominal strength of the pile acting as a short column. Batter piles and their connection shall be capable of resisting forces and moments from the load combinations of Structural Design Section 3.2.

Table 4.8.1 Required Verification and Inspection of Pier Foundations

Verification and Inspection Task	Continuous during Task Listed	Periodically during Task Listed
1. Observe drilling operations and maintain complete and accurate records for each pier	x	-
2. Verify placement locations and plumbness, confirm pier diameters, bell diameters (if applicable), lengths, embedment into bedrock (if applicable) and adequate end capacity.	x	-
3. For concrete piers, perform additional inspections as specified in special inspection required for concrete construction	See Section 1704.4 of IBC 2006	See Section 1704.4 of IBC 2006
4. For masonry piers, perform additional inspections as specified in special inspection required for masonry construction	See Section 1704.5 of IBC 2006	See Section 1704.5 of IBC 2006

Table 4.8.2 Required Verification and Inspection of Pile Foundations

Verification and Inspection Task	Continuous During Task Listed	Periodically During Task Listed
1. Verify pile materials, sizes and lengths comply with the requirements.	x	
2. Determine capacities of test piles and conduct additional load tests, as required.	x	-
3. Observe driving operations and maintain complete and accurate records for each pile.	x	-
4. Verify placement locations and plumpness, confirm type and size of hammer, record number of blows per foot of penetration, determine required capacity, record tip and butt elevations and document any pile damage.	x	-

5. For steel piles, perform additional inspections as specified in special inspection required for steel construction	See Section 1704.3 of IBC 2006	See Section 1704.3 of IBC 2006
6. For concrete piles and concrete-filled piles, perform additional inspections as specified in special inspection required for concrete construction.	See Section 1704.4 of IBC 2006	See Section 1704.4 of IBC 2006
7. For specialty piles, perform additional inspections as determined by the registered design professional in responsible charge.	-	-
8. For augured uncased piles and caisson piles, perform inspections in accordance with pier foundations.	See Table 4.8.1	See Table 4.8.1

SECTION 4.9
DRIVEN PILE FOUNDATIONS

4.9.1 TIMBER PILES

Timber piles shall be designed with the prevailing code. Only structural timber shall be used for piles.

4.9.1.1 Materials

Round timber piles shall conform to ASTM D 25.

4.9.1.2 Preservative treatment

Timber piles used to support permanent structures shall be treated unless it is established that the tops of the untreated timber piles will be below the lowest ground-water level assumed to exist during the life of the structure. Preservative-treated timber piles shall be subject to a quality control program administered by an approved agency. Pile cutoffs shall be treated.

4.9.1.3 Defective piles

Any substantial sudden increase in rate of penetration of a timber pile shall be investigated for possible damage. If the sudden increase in rate of penetration cannot be correlated to soil strata, the pile shall be removed for inspection or rejected.

4.9.1.4 Allowable stresses

The allowable stresses of timber pile shall not exceed values specified in Table 4.9.1.

Table 4.9.1 Allowable Working Stresses for Sawn Timbers (psi)

Symbol	Description	Pyinkado	Teak	Padauk	In/Kanyin
F_b	Bending at fiber stress	2500	2000	2500	1500
F_v	Longitudinal shear	240	120	175	130
F_c	Axial compression	1900	1200	1700	760
F_{cb}	Axial compression when combine with bending	1900	1200	1700	760
$F_{c(per)}$	Compression perpendicular to grain	970	450	1050	400
$F_{t(par)}$	Tension parallel to grain when reduced by notches, daps, connectors or abrupt changes in section	1600	960	1350	610
$F_{t(par)}$	Tension parallel to grain when no stress concentration exists	1900	1200	1700	760
$F_{t(per)}$	Tension perpendicular to grain	60	40	60	60
E	Modulus of Elasticity	2.0×10^6	1.44×10^6	1.65×10^6	1.3×10^6

Note: 1 psi = 6.8966 kPa

4.9.2 PRECAST CONCRETE PILES

4.9.2.1 The materials, reinforcement and installation of precast concrete piles

It shall conform to Sections 4.9.2.1.1 through 4.9.2.1.4.

4.9.2.1.1 Design and manufacture

Piles shall be designed and manufactured in accordance with accepted engineering practice to resist all stresses induced by handling, driving and service loads.

4.9.2.1.2 Minimum dimension

The minimum lateral dimension shall be 6 inches (152 mm).

4.9.2.1.3 Reinforcement

Longitudinal steel shall be arranged in a symmetrical pattern and be laterally tied with steel ties or wire spiral spaced not more than 4 inches (102 mm) apart, center to center, for a distance of 2 feet (610 mm) from the ends of the pile; and not more than 6 inches (152 mm) elsewhere except that at the ends of each pile, the first five ties or spirals shall be spaced 1 inch (25.4 mm) center to center. The gage of ties and spirals shall be as follows:

1. For piles having a diameter of 16 inches (406 mm) or less, wire shall not be smaller 6 mm.
2. For piles having a diameter of more than 16 inches (406 mm) and less than 20 inches (508 mm), wire shall not be smaller than 8 mm.
3. For piles having a diameter of 20 inches (508 mm) and larger, wire shall not be smaller than 9 mm.

4.9.2.1.4 Installation

Piles shall be handled and driven so as not to cause injury or overstressing, which affects durability or strength.

4.9.2.2 Precast non-prestressed piles

Precast non-prestressed concrete piles shall conform to Sections 4.9.2.2.1 through 4.9.2.2.5.

4.9.2.2.1 Materials

Concrete shall have a 28-day specified compressive strength (f_c') of not less than 3,000 psi (20.68 MPa).

4.9.2.2.2 Minimum reinforcement

The minimum amount of longitudinal reinforcement shall be 0.8 percent of the concrete section and shall consist of at least four bars.

4.9.2.2.2.1 Seismic reinforcement in Seismic Design Category C

Where a structure is assigned to Seismic Design Category C in accordance with Section 3.4, the following shall apply. Longitudinal reinforcement with a minimum steel ratio of 0.01 shall be provided throughout the length of precast concrete piles. Within three pile diameters from the bottom of the pile cap, the longitudinal reinforcement shall be confined with closed ties or spirals of a minimum 3/8 inch (10 mm) diameter. Ties or spirals shall be provided at a maximum spacing of eight times the diameter of the smallest longitudinal bar, not to exceed 6 inches (152 mm). Throughout the remainder of the pile, the closed ties or spirals shall have a maximum spacing of 16 times the smallest longitudinal bar diameter not to exceed 8 inches (203 mm).

4.9.2.2.2.2 Seismic reinforcement in Seismic Design Category D, E or F

Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 3.4, the requirements for Seismic Design Category C in Section 4.9.2.2.2.1 shall apply except as modified by this section. Transverse confinement reinforcement consisting of closed ties or equivalent spirals shall be provided in accordance with Sections 21.6.4.2, 21.6.4.3 and 21.6.4.4 of ACI 318-08 within three pile diameters from the bottom of the pile cap. For other than Site Class E or F, or liquefiable sites and where spirals are used as the transverse reinforcement, a volumetric ratio of spiral reinforcement of not less than one-half that required by Section 21.6.4.4(a) of ACI 318-08 shall be permitted.

4.9.2.2.3 Allowable stresses

The allowable compressive stress in the concrete shall not exceed 33 percent of the 28-day specified compressive strength ($f_c^{'}$) applied to the gross cross-sectional area of the pile. The allowable compressive stress in the reinforcing steel shall not exceed 40 percent of the yield strength of the steel (F_y) or a maximum of 30,000 psi (207 MPa). The allowable tensile stress in the reinforcing steel shall not exceed 50 percent of the yield strength of the steel (F_y) or a maximum of 24,000 psi (165 MPa).

4.9.2.2.4 Installation

A precast concrete pile shall not be driven before the concrete has attained a compressive strength of at least 75 percent of the 28-day specified compressive strength ($f_c^{'}$), but not less than the strength sufficient to withstand handling and driving forces.

4.9.2.2.5 Concrete cover

Reinforcement for piles that are not manufactured under plant conditions shall have a concrete cover of not less than 2 inches (51 mm). Reinforcement for piles manufactured under plant control conditions shall have a concrete cover of not less than 1.25 inches (32 mm) for No. 5 bars and smaller, and not less than 1.5 inches (38 mm) for No. 6 through No. 11 bars except that longitudinal bars spaced less than 1.5 inches (38 mm) clear distance apart shall be considered bundled bars for which the minimum concrete cover shall be equal to that for the equivalent diameter of the bundled bars. Reinforcement for piles exposed to seawater shall have a concrete cover of not less than 3 inches (76 mm).

4.9.2.3 Precast prestressed piles

Precast prestressed concrete piles shall conform to the requirements of Sections 4.9.2.3.1 through 4.9.2.3.5.

4.9.2.3.1 Materials

Prestressing steel shall conform to ASTM A 416. Concrete shall have a 28-day specified compressive strength ($f_c^{'}$) of not less than 5,000 psi (34.48 MPa).

4.9.2.3.2 Design

Precast prestressed piles shall be designed to resist stresses induced by handling and driving as well as by loads. The effective prestress in the pile shall not be less than 400 psi (2.76MPa) for piles up to 30 feet (9144 mm) in length, 550 psi (3.79 MPa) for piles up to 50 feet (15 240 mm) in length and 700 psi (4.83 MPa) for piles greater than 50 feet (15 240 mm) in length. Effective prestress shall be based on an assumed loss of 30,000 psi (207 MPa) in the prestressing steel. The tensile stress in the prestressing steel shall not exceed the values specified in ACI 318-08.

4.9.2.3.2.1 Design in Seismic Design Category C

Where a structure is assigned to Seismic Design Category C in accordance with Section 3.4, the following shall apply. The minimum volumetric ratio of spiral reinforcement shall not be less than 0.007 or the amount required by the following formula for the upper 20 feet (6096 mm) of the pile.

$$\rho_s = 0.12 f_c^{'}/f_{yh} \qquad \textbf{(Equation 4.9.1)}$$

where:

f_c' = Specified compressive strength of concrete, psi (MPa).

f_{yh} = Yield strength of spiral reinforcement ≤ 85,000 psi (586 MPa).

ρ_s = Spiral reinforcement index (vol. spiral/vol. core).

At least one-half the volumetric ratio required by Equation 4.9.1 shall be provided below the upper 20 feet (6096 mm) of the pile.

The pile cap connection by means of dowels as indicated in Section 4.8.24 is permitted. Pile cap connection by means of developing pile reinforcing strand is permitted provided that the pile reinforcing strand results in a ductile connection.

4.9.2.3.2.2 Design in Seismic Design Category D, E or F

Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 3.4, the requirements for Seismic Design Category C in Section 4.9.2.3.2.1 shall be met, in addition to the following:

1. Requirements in ACI 318-08, Chapter 21, need not apply, unless specifically referenced.
2. Where the total pile length in the soil is 35 feet (10668 mm) or less, the lateral transverse reinforcement in the ductile region shall occur through the length of the pile. Where the pile length exceeds 35 feet (10 668 mm), the ductile pile region shall be taken as the greater of 35 feet (10668 mm) or the distance from the underside of the pile cap to the point of zero curvature plus three times the least pile dimension.
3. In the ductile region, the center-to-center spacing of the spirals or hoop reinforcement shall not exceed one-fifth of the least pile dimension, six times the diameter of the longitudinal strand, or 8 inches (203 mm), whichever is smaller.
4. Circular spiral reinforcement shall be spliced by lapping one full turn and bending the end of the spiral to a 90-degree hook or by use of a mechanical or welded splice complying with Section 12.14.3 of ACI 318-08.
5. Where the transverse reinforcement consists of circular spirals, the volumetric ratio of spiral transverse reinforcement in the ductile region shall comply with the following:

$$\rho_s = 0.25(f_c'/f_{yh})(A_g/A_{ch} - 1.0)[0.5 + 1.4P/f_c'A_g] \qquad \textbf{(Equation 4.9.2)}$$

but not less than:

$$\rho_s = 0.12(f_c'/f_{yh})[0.5 + 1.4P/f_c'A_g] \qquad \textbf{(Equation 4.9.3)}$$

and need not exceed:

$$\rho_s = 0.021 \qquad \textbf{(Equation 4.9.4)}$$

where:

A_g = Pile cross-sectional area, square inches (mm^2).

A_{ch} = Core area defined by spiral outside diameter, square inches (mm^2).

f_c' = Specified compressive strength of concrete, psi (MPa).

f_{yh} = Yield strength of spiral reinforcement ≤ 85,000 psi (586 MPa).

P = Axial load on pile, pounds (kN), as determined from Equations 3.2.5 and 3.2.6.

ρ_s = Volumetric ratio (vol. spiral/ vol. core).

This required amount of spiral reinforcement is permitted to be obtained by providing an inner and outer spiral.

6. When transverse reinforcement consists of rectangular hoops and cross ties, the total cross-sectional area of lateral transverse reinforcement in the ductile region with spacing, and perpendicular to dimension, hc, shall conform to:

$$Ash = 0.3sh_c(f_c'/f_{yh})(A_g/A_{ch} - 1.0)[0.5 + 1.4P/f_c'A_g] \quad \text{(Equation 4.9.5)}$$

but not less than:

$$A_{sh} = 0.12sh_c(f_c'/f_{yh})[0.5 + 1.4P/f_c'A_g] \quad \text{(Equation 4.9.6)}$$

where:

f_{yh} = ≤ 70,000 psi (483 MPa)

h_c = Cross-sectional dimension of pile core measured center to center of hoop reinforcement, inch (mm).

s = Spacing of transverse reinforcement measured along length of pile, inch (mm).

A_{sh} = Cross-sectional area of transverse reinforcement, square inches (mm2).

f_c' = Specified compressive strength of concrete, psi (MPa).

The hoops and cross ties shall be equivalent to deformed bars not less than 10 mm in size. Rectangular hoop ends shall terminate at a corner with seismic hooks.

Outside of the length of the pile requiring transverse confinement reinforcing, the spiral or hoop reinforcing with a volumetric ratio not less than one-half of that required for transverse confinement reinforcing shall be provided.

4.9.2.3.3 Allowable stresses

The allowable design compressive stress, f_c', in concrete shall be determined as follows:

$$f_c' = 0.33\, f_c' - 0.27f_{pc} \quad \text{(Equation 4.9.7)}$$

where:

f_c' = The 28-day specified compressive strength of the concrete.

f_{pc} = The effective prestress stress on the gross section.

4.9.2.3.4 Installation

A prestressed pile shall not be driven before the concrete has attained a compressive strength of at least 75 percent of the 28-day specified compressive strength (f_c'), but not less than the strength sufficient to withstand handling and driving forces.

4.9.2.3.5 Concrete cover

Prestressing steel and pile reinforcement shall have a concrete cover of not less than 1¼ inches (32 mm) for square piles of 12 inches (305 mm) or smaller size and 1½ inches (38 mm) for larger piles, except that for piles exposed to seawater, the minimum protective concrete cover shall not be less than 2½ inches (64 mm).

4.9.3 STRUCTURAL STEEL PILES

Structural steel piles shall conform to the requirements of Sections 4.9.3.1 through 4.9.3.4.

4.9.3.1 Materials

Structural steel piles, steel pipe and fully welded steel piles fabricated from plates shall conform to ASTM A36, ASTM A252, ASTM A283, ASTM A572, ASTM A588, ASTM A690, ASTM A913 or ASTM A992.

4.9.3.2 Allowable stresses

The allowable axial stresses shall not exceed 35 percent of the minimum specified yield strength (F_y).

Exception:

Where justified in accordance with Section 4.8.9, the allowable axial stress is permitted to be increased above $0.35F_y$, but shall not exceed $0.5F_y$.

4.9.3.3 Dimensions of H-piles

Sections of H-piles shall comply with the following:

1. The flange projections shall not exceed 14 times the minimum thickness of metal in either the flange or the web and the flange widths shall not be less than 80 percent of the depth of the section.
2. The nominal depth in the direction of the web shall not be less than 8 inches (203 mm).
3. Flanges and web shall have a minimum nominal thickness of 3/8 inch (10 mm).

4.9.3.4 Dimensions of steel pipe piles

Steel pipe piles driven open ended shall have a nominal outside diameter of not less than 8 inches (203 mm). The pipe shall have a minimum cross section of 0.34 square inch (219 mm^2) to resist each 1,000 foot-pounds (1356 N-m) of pile hammer energy, or shall have the equivalent strength for steels having a yield strength greater than 35,000 psi (241 MPa) or the wave equation analysis shall be permitted to be used to assess compression stresses induced by driving to evaluate if the pile section is appropriate for the selected hammer. Where pipe wall thickness less than 0.179 inch (4.6 mm) is driven open ended, a suitable cutting shoe shall be provided.

SECTION 4.10
CAST-IN-PLACE CONCRETE PILE FOUNDATIONS

4.10.1 GENERAL

The materials, reinforcement and installation of cast-in-place concrete piles shall conform to Sections 4.10.1.1 through 4.10.1.3.

4.10.1.1 Materials

Concrete shall have a 28-day specified compressive strength (f_c') of not less than 3000 psi (MPa). Where concrete is placed through a funnel hopper at the top of the pile, the concrete mix shall be designed and proportioned so as to produce a cohesive workable mix having a slump of not less than 4 inches (102 mm) and not more than 8 inches (203 mm). Where concrete is to be pumped, the mix design including slump shall be adjusted to produce a pumpable concrete.

4.10.1.2 Reinforcement

Reinforcement shall be provided to meet requirements of rational analysis or building authorities. Except for steel dowels embedded 5 feet (1524 mm) or less in the pile and as provided in Section 4.10.3.4, reinforcement where required shall be assembled and tied together and shall be placed in the pile as a unit before the reinforced portion of the pile is filled with concrete except in augured uncased cast-in-place piles. Tied reinforcement in augured uncased cast-in-place piles shall be placed after piles are concreted, while the concrete is still in a semi fluid state.

4.10.1.2.1 Reinforcement in Seismic Design Category C

Where a structure is assigned to Seismic Design Category C in accordance with Section 3.4, the following shall apply unless otherwise specified by building authorities. A minimum longitudinal reinforcement ratio of 0.0025 shall be provided for uncased cast-in-place concrete drilled or augured piles, piers or caissons in the top one-third of the pile length, a minimum length of 10 feet (3048 mm) below the ground or that required by analysis, whichever length is greatest. The minimum reinforcement ratio, but no less than that ratio required by rational analysis, shall be continued throughout the flexural length of the pile. There shall be a minimum of four longitudinal bars with closed ties (or equivalent spirals) of a minimum 3/8 inch (9 mm) diameter provided at 16-longitudinal-bar diameter maximum spacing. Transverse confinement reinforcement with a maximum spacing of 6 inches (152 mm) or 8-longitudinal- bar diameters, whichever is less, shall be provided within a distance equal to three times the least pile dimension from the bottom of the pile cap.

4.10.1.2.2 Reinforcement in Seismic Design Category D, E or F

Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 3.4, the requirements for Seismic Design Category C given above shall be met, in addition to the following requirements unless otherwise specified by building authorities. A minimum longitudinal reinforcement ratio of 0.005 shall be provided for uncased cast-in-place drilled or augured concrete piles, piers or caissons in the top one-half of the pile length, a minimum length of 10 feet (3048 mm) below ground or throughout the flexural length of the pile, whichever length is greatest. The flexural length shall be taken as the length of the pile to a point where the concrete section cracking moment strength multiplied by 0.4 exceeds the required moment strength at that point. There shall be a minimum of four longitudinal bars with transverse confinement reinforcement provided in the pile in accordance with Sections 21.6.4.2, 21.6.4.3 and 21.6.4.4 of ACI 318-08 within three times the least pile dimension from the bottom of the pile cap. A transverse spiral reinforcement ratio of not less than one-half of that required in Section 21.6.4.4 (a) of ACI 318-08 for other than Class E, F or liquefiable sites is permitted. Tie spacing throughout the remainder of the concrete section shall neither exceed 12-longitudinal-bar diameters, one-half the least dimension of the section, nor 12 inches (305 mm). Ties shall be a minimum of 10 mm bars for piles with a least dimension up to 20 inches (508 mm), and 12 mm bars for larger piles.

4.10.1.3 Concrete placement

Concrete shall be placed in such a manner as to ensure the exclusion of any foreign matter and to secure a full-sized shaft. Concrete shall not be placed through water except where a tremie or other approved method is used. When depositing concrete from the top of the pile, the concrete shall not be chuted directly into the pile but shall be poured in a rapid and continuous operation through a funnel hopper centered at the top of the pile.

4.10.2 ENLARGED BASE PILES

Enlarged base piles shall conform to the requirements of Sections 4.10.2.1 through 4.10.2.5.

4.10.2.1 Materials

The maximum size for coarse aggregate for concrete shall be 3/4 inch (19.1 mm). Concrete to be compacted shall have a zero slump.

4.10.2.2 Allowable stresses

The maximum allowable design compressive stress for concrete not placed in a permanent steel casing shall be 25 percent of the 28-day specified compressive strength (f_c'). Where the concrete is placed in a permanent steel casing, the maximum allowable concrete stress shall be 33 percent of the 28-day specified compressive strength (f_c').

4.10.2.3 Installation

Enlarged bases formed either by compacting concrete or driving a precast base shall be formed in or driven into granular soils. Piles shall be constructed in the same manner as successful prototype test piles driven for the project. Pile shafts extending through peat or other organic soil shall be encased in a permanent steel casing. Where a cased shaft is used, the shaft shall be adequately reinforced to resist column action or the annular space around the pile shaft shall be filled sufficiently to reestablish lateral support by the soil. Where pile heave occurs, the pile shall be replaced unless it is demonstrated that the pile is undamaged and capable of carrying twice its design load.

4.10.2.4 Load-bearing capacity

Pile load-bearing capacity shall be verified by load tests in accordance with Section 4.8.7.3.

4.10.2.5 Concrete cover

The minimum concrete cover shall be 2½ inches (64 mm) for uncased shafts and 1 inch (25 mm) for cased shafts.

4.10.3 DRILLED OR AUGURED UNCASED PILES

Drilled or augured uncased piles shall conform to Sections 4.10.3.1 through 4.10.3.5.

4.10.3.1 Allowable stresses

The allowable design stress in the concrete of drilled or augured uncased piles shall not exceed 33 percent of the 28-day specified compressive strength (f_c'). The allowable compressive stress of reinforcement shall not exceed 40 percent of the yield strength of the steel or 25,500 psi (175.8 MPa).

4.10.3.2 Dimensions

The pile length shall not exceed 30 times the average diameter. The minimum diameter shall be 12 inches (305 mm).

Exception:

The length of the pile is permitted to exceed 30 times the diameter, provided that the design and installation of the pile foundation are under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and pile foundations. The registered design professional shall certify to the building official that the piles were installed in compliance with the approved construction documents.

4.10.3.3 Installation

Open-drilled hole shall be supported by a steel liner or other approved method to offset any hydrostatic or lateral soil pressure. Concrete placement shall be in accordance with Section 4.10.1.3. Where pile shafts are formed through unstable soils and concrete is placed in an open-drilled hole, a steel liner shall be inserted in the hole prior to placing the concrete. Where the steel liner is withdrawn during concreting, the level of concrete shall be maintained above the bottom of the liner at a sufficient height to offset any hydrostatic or lateral soil pressure.

Where concrete is placed by pumping through a hollow- stem auger, the auger shall be permitted to rotate in a clockwise direction during withdrawal. The auger shall be withdrawn in continuous increments. Concreting pumping pressures shall be measured and maintained high enough at all times to offset hydrostatic and lateral earth pressures. Concrete volumes shall be measured to ensure that the volume of concrete placed in each pile is equal to or greater than the theoretical volume of the hole created by the auger.

Where the installation process of any pile is interrupted or a loss of concreting pressure occurs, the pile shall be redrilled to 5 feet (1524 mm) below the elevation of the tip of the auger when the installation was interrupted or concrete pressure was lost and reformed.

Augured cast-in-place piles shall not be installed within six pile diameters center to center of a pile filled with concrete less than 12 hours old, unless approved by the building official. If the concrete level in any completed pile drops due to installation of an adjacent pile, the pile shall be replaced.

4.10.3.4 Reinforcement

Reinforcements shall be provided to meet requirements of Section 4.10.1.2. All pile reinforcement shall have a concrete cover of not less than 2.5 inches (64 mm). For piles installed with a hollow-stem auger where full-length longitudinal steel reinforcement is placed, the reinforcement shall be placed through the hollow stem of the auger prior to filling the pile with concrete.

Exception:

Where physical constraints do not allow the placement of the longitudinal reinforcement prior to filling the pile with concrete or where partial-length longitudinal reinforcement is placed, the reinforcement is allowed to be placed after the piles are completely concreted but while concrete is still in a semifluid state.

4.10.3.5 Reinforcement in Seismic Design Category C, D, E or F

Where a structure is assigned to Seismic Design Category C, D, E or F in accordance with Section 3.4, the corresponding requirements of Sections 4.10.1.2.1 and 4.10.1.2.2 shall be met.

4.10.4 DRIVEN UNCASED PILES

Driven uncased piles shall conform to Sections 4.10.4.1 through 4.10.4.4.

4.10.4.1 Allowable stresses

The allowable design stress in the concrete shall not exceed 25 percent of the 28-day specified compressive strength (f_c') applied to a cross-sectional area not greater than the inside area of the drive casing or mandrel.

4.10.4.2 Dimensions

The pile length shall not exceed 30 times the average diameter. The minimum diameter shall be 12 inches (305 mm).

Exception:

The length of the pile is permitted to exceed 30 times the diameter, provided that the design and installation of the pile foundation is under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and pile foundations. The registered design professional shall certify to the building official that the piles were installed in compliance with the approved design.

4.10.4.3 Installation

Piles shall not be driven within six pile diameters center to center in granular soils or within one-half the pile length in cohesive soils of a pile filled with concrete not less than 48 hours old unless approved by the registered design professional the building official. If the concrete surface in any completed pile rises or drops, the pile shall be replaced. Piles shall not be installed in soils that could cause pile heave.

4.10.4.4 Concrete cover

Pile reinforcement shall have a concrete cover of not less than 2.5 inches (64 mm), measured from the inside face of the drive casing or mandrel.

4.10.5 STEEL-CASED PILES

Steel-cased piles shall comply with the requirements of Sections 4.10.5.1 through 4.10.5.4.

4.10.5.1 Materials

Pile shells or casings shall be of steel and shall be sufficiently strong to resist collapse and sufficiently water-tight to exclude any foreign materials during the placing of concrete. Steel shells shall have a sealed tip with a diameter of not less than 8 inches (203 mm).

4.10.5.2 Allowable stresses

The allowable design compressive stress in the concrete shall not exceed 33 percent of the 28-day specified compressive strength (f_c'). The allowable concrete compressive stress shall be $0.40\ f_c'$ for that portion of the pile meeting the conditions specified in Sections 4.10.5.2.1 through 4.10.5.2.4.

4.10.5.2.1 Shell thickness

The thickness of the steel shell shall not be less than manufacturer's standard gage No. 14 gage (0.068 inch) (1.75 mm) minimum.

4.10.5.2.2 Shell type

The shell shall be seamless or provided with seams of strength equal to the basic material and be of a configuration that will provide confinement to the cast-in-place concrete.

4.10.5.2.3 Strength

The ratio of steel yield strength (F_y) to 28-day specified compressive strength (f_c') shall not be less than six.

4.10.5.2.4 Diameter

The nominal pile diameter shall not be greater than 16 inches (406 mm).

4.10.5.3 Installation

Steel shells shall be mandrel driven their full length in contact with the surrounding soil. The steel shells shall be driven in such order and with such spacing as to ensure against distortion of or injury to piles already in place. A pile shall not be driven within four and one-half average pile diameters of a pile filled with concrete less than 24 hours old unless approved by the building official. Concrete shall not be placed in steel shells within heave range of driving.

4.10.5.4 Reinforcement

Reinforcement shall not be placed within 1 inch (25 mm) of the steel shell. Reinforcing shall be required for unsupported pile lengths or where the pile is designed to resist uplift or unbalanced lateral loads.

4.10.5.4.1 Seismic reinforcement

Where a structure is assigned to Seismic Design Category C, D, E or F in accordance with Section 3.4, the reinforcement requirements for drilled or augured uncased piles in Section 4.10.3.5 shall be met.

Exception:

A spiral-welded metal casing of a thickness no less than the manufacturer's standard gage No. 14 gauge [0.068 inch (1.7 mm)] is permitted to provide concrete confinement in lieu of the closed ties or equivalent spirals required in an uncased concrete pile. Where used as such, the metal casing shall be protected against possible deleterious action due to soil constituents, changing water levels or other factors indicated by boring records of site conditions.

4.10.6 CONCRETE-FILLED STEEL PIPE AND TUBE PILES

Concrete-filled steel pipe and tube piles shall conform to the requirements of Sections 4.10.6.1 through 4.10.6.5.

4.10.6.1 Materials

Steel pipe and tube sections used for piles shall conform to ASTM A252 or ASTM A283. Concrete shall conform to Section 4.10.1.1. The maximum coarse aggregate size shall be 3/4 inch (19.1 mm).

4.10.6.2 Allowable stresses

The allowable design compressive stress in the concrete shall not exceed 33 percent of the 28-day specified compressive strength (f_c'). The allowable design compressive stress in the steel shall not exceed 35 percent of the minimum specified yield strength of the steel (F_y), provided F_y shall not be assumed greater than 36,000 psi (248 MPa) for computational purposes.

Exception:

Where justified in accordance with Section 4.8.9, the allowable stresses are permitted to be increased to $0.50\ F_y$.

4.10.6.3 Minimum dimensions

Piles shall have a nominal outside diameter of not less than 8 inches (203 mm) and a minimum wall thickness in accordance with Section 4.5.3.2.3.4. For mandrel-driven pipe piles, the minimum wall thickness shall be 1/10 inch (2.5 mm).

4.10.6.4 Reinforcement

Reinforcement steel shall conform to Section 4.8.9. Reinforcement shall not be placed within 1 inch (25 mm) of the steel casing.

4.10.6.4.1 Seismic reinforcement

Where a structure is assigned to Seismic Design Category C, D, E or F in accordance with Section 3.4, the following shall apply. Minimum reinforcement no less than 0.01 times the cross-sectional area of the pile concrete shall be provided in the top of the pile with a length equal to two times the required cap embedment anchorage into the pile cap, but not less than the tension development length of the reinforcement. The wall thickness of the steel pipe shall not be less than 3/16 inch (5 mm).

4.10.6.5 Placing concrete

The placement of concrete shall conform to Section 4.10.1.3, but is permitted to be chuted directly into smooth-sided pipes and tubes without a centering funnel hopper.

4.10.7 CAISSON PILES

Caisson piles shall conform to the requirements of Sections 4.10.7.1 through 4.10.7.6.

4.10.7.1 Construction

Caisson piles shall consist of a shaft section of concrete-filled pipe extending to bedrock with an uncased socket drilled into the bedrock and filled with concrete. The caisson pile shall have a full-length structural steel core or a stub core installed in the rock socket and extending into the pipe portion a distance equal to the socket depth.

4.10.7.2 Materials

Pipe and steel cores shall conform to the material requirements in Section 4.9.3. Pipes shall have a minimum wall thickness of 3/8 inch (9.5 mm) and shall be fitted with a suitable steel-driving shoe welded to the bottom of the pipe. Concrete shall have a 28-day specified compressive strength (f_c') of not less than 4,000 psi (27.58 MPa). The concrete mix shall be designed and proportioned so as to produce a cohesive workable mix with a slump of 4 inches to 6 inches (102 mm to 152 mm).

4.10.7.3 Design

The depth of the rock socket shall be sufficient to develop the full load-bearing capacity of the caisson pile with a minimum safety factor of two, but the depth shall not be less than the outside diameter of the pipe. The design of the rock socket is permitted to be predicated on the sum of the allowable load-bearing pressure on the bottom of the socket plus bond along the sides of the socket. The minimum outside diameter of the caisson pile shall be 18 inches (457 mm), and the diameter of the rock socket shall be approximately equal to the inside diameter of the pile.

4.10.7.4 Structural core

The gross cross-sectional area of the structural steel core shall not exceed 25 percent of the gross area of the caisson. The minimum clearance between the structural core and the pipe shall be 2 inches (51 mm). Where cores are to be spliced, the ends shall be milled or ground to provide full contact and shall be full-depth welded.

4.10.7.5 Allowable stresses

The allowable design compressive stresses shall not exceed the following:

concrete,	$0.33\ f_c'$
steel pipe,	$0.35\ F_y$
structural steel core,	$0.50\ F_y$

4.10.7.6 Installation

The rock socket and pile shall be thoroughly cleaned of foreign materials before filling with concrete. Steel cores shall be bedded in cement grout at the base of the rock socket. Concrete shall not be placed through water except where a tremie or other approved method is used

4.10.8 MICRO PILES

Micro piles shall conform to the requirements of Sections 4.10.8.1 through 4.10.8.5.

4.10.8.1 Construction

Micro piles shall consist of a grouted section reinforced with steel pipe or steel reinforcing. Micro piles shall develop their load-carrying capacity through a bond zone in soil, bedrock or a combination of soil and bedrock. The full length of the micro pile shall contain either a steel pipe or steel reinforcement.

4.10.8.2 Materials

Grout shall have a 28-day specified compressive strength (f_c') of not less than 4,000 psi (27.58 MPa). The grout mix shall be designed and proportioned so as to produce a pumpable mixture. Reinforcing steel shall be deformed bars in accordance with ASTM A615 Grade 60 or 75 or ASTM A722 Grade 150. Pipe/casing shall have a minimum wall thickness of 3/16 inch (4.8 mm) and as required to meet Section 4.8.6. Pipe/casing shall meet the tensile requirements of ASTM A252 Grade 3, except the minimum yield strength shall be as used in the design submittal [typically 50,000 psi to 80,000 psi (345 MPa to 552 MPa)] and minimum elongation shall be 15 percent.

4.10.8.3 Allowable stresses

The allowable design compressive stress on grout shall not exceed $0.33\ f_c'$. The allowable design compressive stress on steel pipe and steel reinforcement shall not exceed the lesser of $0.40\ F_y$, or 32,000 psi (220 MPa). The allowable design tensile stress for steel reinforcement shall not exceed $0.60\ F_y$. The allowable design tensile stress for the cement grout shall be zero.

4.10.8.4 Reinforcement

For piles or portions of piles grouted inside a temporary or permanent casing or inside a hole drilled into bedrock or a hole drilled with grout, the steel pipe or steel reinforcement shall be designed to carry at least 40 percent of the design compression load. Piles or portions of piles grouted in an open hole in soil without temporary or permanent casing and without suitable means of verifying the hole diameter during grouting shall be designed to carry the entire compression load in the reinforcing steel. Where a steel pipe is used for reinforcement, the portion of the cement grout enclosed within the pipe is permitted to be included at the allowable stress of the grout.

4.10.8.4.1 Seismic reinforcement

Where a structure is assigned to Seismic Design Category C, a permanent steel casing shall be provided from the top of the pile down 120 percent times the flexural length. The flexural length is the length of the pile from the first point of zero lateral deflection to the underside of the pile cap or grade beam. Where a structure is assigned to Seismic Design Category D, E or F, the pile shall be considered as an alternative system. The alternative pile system design, supporting documentation and test data shall be submitted to the building official for review and approval.

4.10.8.5 Installation

The pile shall be permitted to be formed in a hole advanced by rotary or percussive drilling methods, with or without casing. The pile shall be grouted with a fluid cement grout. The grout shall be pumped through a tremie pipe extending to the bottom of the pile until grout of suitable quality returns at the top of the pile. The following requirements apply to specific installation methods:

1. For piles grouted inside a temporary casing, the reinforcing steel shall be inserted prior to withdrawal of the casing. The casing shall be withdrawn in a controlled manner with the grout level maintained at the top of the pile to ensure that the grout completely fills the drill hole.
2. During withdrawal of the casing, the grout level inside the casing shall be monitored to check that the flow of grout inside the casing is not obstructed. For a pile or portion of a pile grouted in an open drill hole in soil without temporary casing, the minimum design diameter of the drill hole shall be verified by a suitable device during grouting.
3. For piles designed for end bearing, a suitable means shall be employed to verify that the bearing surface is properly cleaned prior to grouting.
4. Subsequent piles shall not be drilled near piles that have been grouted until the grout has had sufficient time to harden.
5. Piles shall be grouted as soon as possible after drilling is completed.
6. For piles designed with casing full length, the casing must be pulled back to the top of the bond zone and reinserted or some other suitable means shall be employed to verify grout coverage outside the casing.

SECTION 4.11
COMPOSITE PILES

4.11.1 GENERAL

Composite piles shall conform to the requirements of Sections 4.11.2 through 4.11.5.

4.11.2 DESIGN

Composite piles consisting of two or more approved pile types shall be designed to meet the conditions of installation and requirements of concerning building authorities.

4.11.3 LIMITATION OF LOAD

The maximum allowable load shall be limited by the capacity of the weakest section incorporated in the pile.

4.11.4 SPLICES

Splices between concrete and steel or wood sections shall be designed to prevent separation both before and after the concrete portion has set, and to ensure the alignment and transmission of the total pile load. Splices shall be designed to resist uplift caused by upheaval during driving of adjacent piles, and shall develop the full compressive strength and not less than 50 percent of the tension and bending strength of the weaker section.

4.11.5 SEISMIC REINFORCEMENT

Where a structure is assigned to Seismic Design Category C, D, E or F in accordance with Section 3.4, the following shall apply. Where concrete and steel are used as part of the pile assembly, the concrete reinforcement shall comply with that given in Sections 4.10.1.2.1 and 4.10.1.2.2 or the steel section shall comply with Section 4.10.6.4.1.

SECTION 4.12
PIER FOUNDATIONS

4.12.1 GENERAL

Isolated and multiple piers used as foundations shall conform to the requirements of Sections 4.12.2 through 4.12.10, as well as the applicable provisions of Section 4.8.

4.12.2 LATERAL DIMENSIONS AND HEIGHT

The minimum dimension of isolated piers used as foundations shall be 2 feet (610 mm), and the height shall not exceed 12 times the least horizontal dimension.

4.12.3 MATERIALS

Concrete shall have a 28-day specified compressive strength (f_c') of not less than 2,500 psi (17.24 MPa). Where concrete is placed through a funnel hopper at the top of the pier, the concrete mix shall be designed and proportioned so as to produce a cohesive workable mix having a slump of not less than 4 inches (102 mm) and not more than 6 inches (152 mm). Where concrete is to be pumped, the mix design including slump shall be adjusted to produce a pumpable concrete.

4.12.4 REINFORCEMENT

Except for steel dowels embedded 5 feet (1524 mm) or less in the pier, reinforcement where required shall be assembled and tied together and shall be placed in the pier hole as a unit before the reinforced portion of the pier is filled with concrete.

Exception:

Reinforcement is permitted to be wet set and the 2½ inch (64 mm) concrete cover requirement be reduced to 2 inches (51 mm) for temporary and low occupancies (Group U and R-3 in IBC 2006 Chapter 3) not exceeding two stories of light-frame construction, provided the construction method can be demonstrated to the satisfaction of the building official.

Reinforcement shall conform to the requirements of Sections 4.12.1.2.1 and 4.12.1.2.2.

Exceptions:

1. Isolated piers supporting posts of temporary and low occupancies (Group U and R-3 in IBC 2006 Chapter 3) not exceeding two stories of light-frame construction are permitted to be reinforced as required by rational analysis but not less than a minimum of one No. 4 bar, without ties or spirals, when detailed so the pier is not subject to lateral loads and the soil is determined to be of adequate stiffness.
2. Isolated piers supporting posts and bracing from decks and patios appurtenant to temporary and low occupancies (Group U and R-3 in IBC 2006 Chapter 3) not exceeding two stories of light-frame construction are permitted to be reinforced as required by rational analysis but not less than one No. 4 bar, without ties or spirals, when the lateral load, E, to the top of the pier does not exceed 200 pounds (890 N) and the soil is determined to be of adequate stiffness.
3. Piers supporting the concrete foundation wall of temporary and low occupancies (Group U and R-3 in IBC 2006 Chapter 3) not exceeding two stories of light-frame construction are permitted to be reinforced as required by rational analysis but not less than two No. 4 bars, without ties or spirals, when it can be shown the concrete pier will not rupture when designed for the maximum seismic load, E_m, and the soil is determined to be of adequate stiffness.

4. Closed ties or spirals where required by Section 4.12.1.2.2 are permitted to be limited to the top 3 feet (914 mm) of the piers 10 feet (3048 mm) or less in depth supporting temporary and low occupancies (Group U and R-3 in IBC 2006 Chapter 3) of Seismic Design Category D, not exceeding two stories of light-frame construction.

4.12.5 CONCRETE PLACEMENT

Concrete shall be placed in such a manner as to ensure the exclusion of any foreign matter and to secure a full-sized shaft. Concrete shall not be placed through water except where a tremie or other approved method is used. When depositing concrete from the top of the pier, the concrete shall not be chute directly into the pier but shall be poured in a rapid and continuous operation through a funnel hopper centered at the top of the pier.

4.12.6 BELLED BOTTOMS

Where pier foundations are belled at the bottom, the edge thickness of the bell shall not be less than that required for the edge of footings. When the sides of the bell slope at an angle less than 60 degrees (1 rad) from the horizontal, the effects of vertical shear shall be considered.

4.12.7 MASONRY

Where the unsupported height of foundation piers exceeds six times the least dimension, the allowable working stress on piers of unit masonry shall be reduced in accordance with ACI 530/ASCE 5/TMS 402.

4.12.8 CONCRETE

Where adequate lateral support is not provided, and the unsupported height to least lateral dimension does not exceed three, piers of plain concrete shall be designed and constructed as pilasters in accordance with ACI 318-08. Where the unsupported height to least lateral dimension exceeds three, piers shall be constructed of reinforced concrete, and shall conform to the requirements for columns in ACI 318-08.

Exception:

Where adequate lateral support is furnished by the surrounding materials as defined in Section 4.8.8, piers are permitted to be constructed of plain or reinforced concrete. The requirements of ACI 318-08 for bearing on concrete shall apply.

4.12.9 STEEL SHELL

Where concrete piers are entirely encased with a circular steel shell, and the area of the shell steel is considered reinforcing steel, the steel shall be protected under the conditions specified in Section 4.8.17. Horizontal joints in the shell shall be spliced to comply with Section 4.8.6.

4.12.10 DEWATERING

Where piers are carried to depths below water level, the piers shall be constructed by a method that will provide accurate preparation and inspection of the bottom, and the depositing or construction of sound concrete or other masonry in the dry.

SECTION 4.13
OTHER REQUIREMENTS

4.13.1 SOIL IMPROVEMENT

Suitable improvement methods should be applied according to the requirements of proposed sites. Necessity and application of suitable soil improvement program shall be determined by registered geotechnical professional.

4.13.2 INSTRUMENTATION AND MONITORING

Suitable instrumentation and monitoring methods should be applied according to the requirements of proposed sites. Necessity and application of suitable instrumentation and monitoring scheme shall be determined by registered geotechnical professional.

APPENDIX A
USEFUL INFORMATION FOR MYANMAR

Figure A.1 Geological Map of Myanmar (Bender F., et al, 1981)

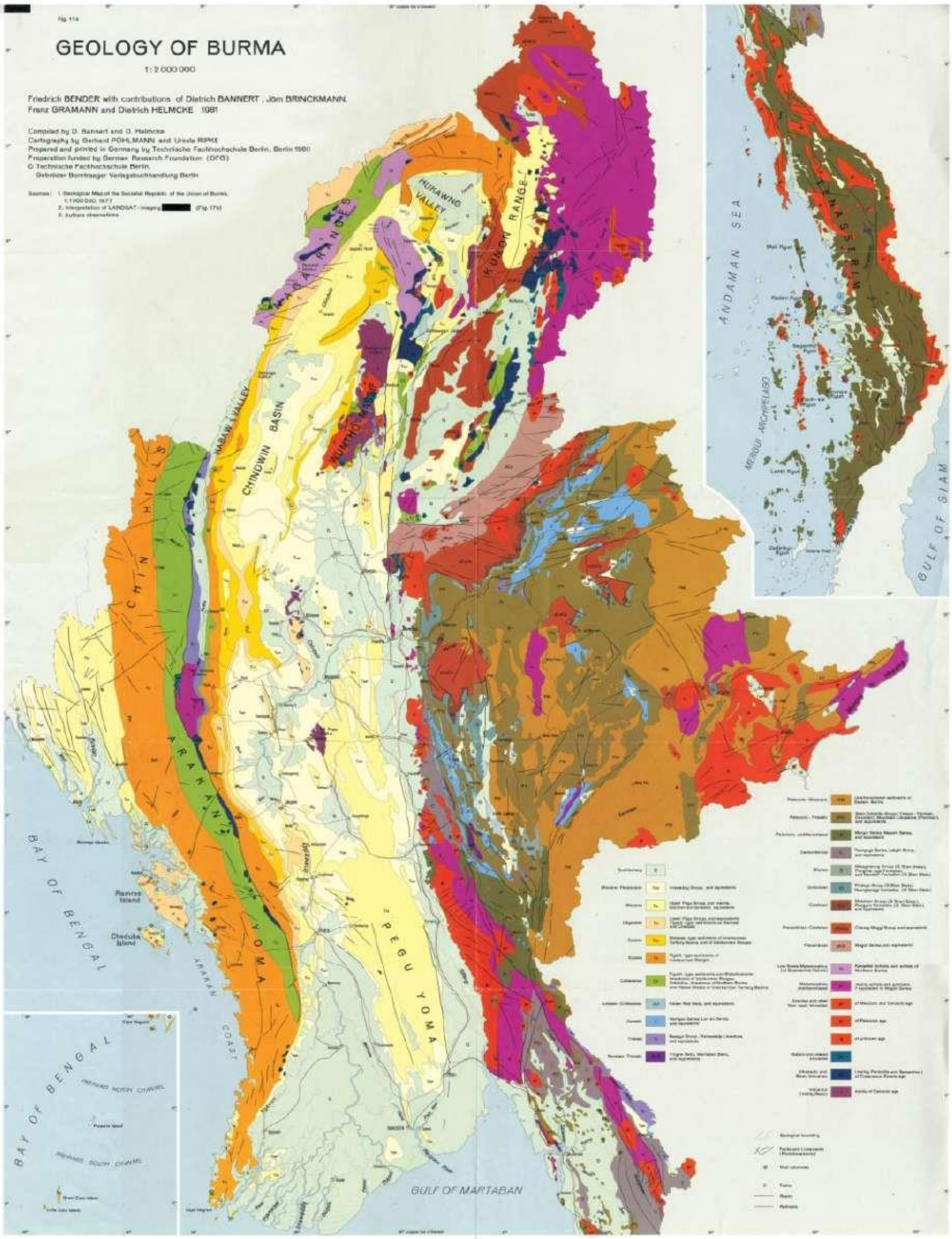


Figure A.2 Tectonic Map of Myanmar and its surrounding (MGS, 2007)

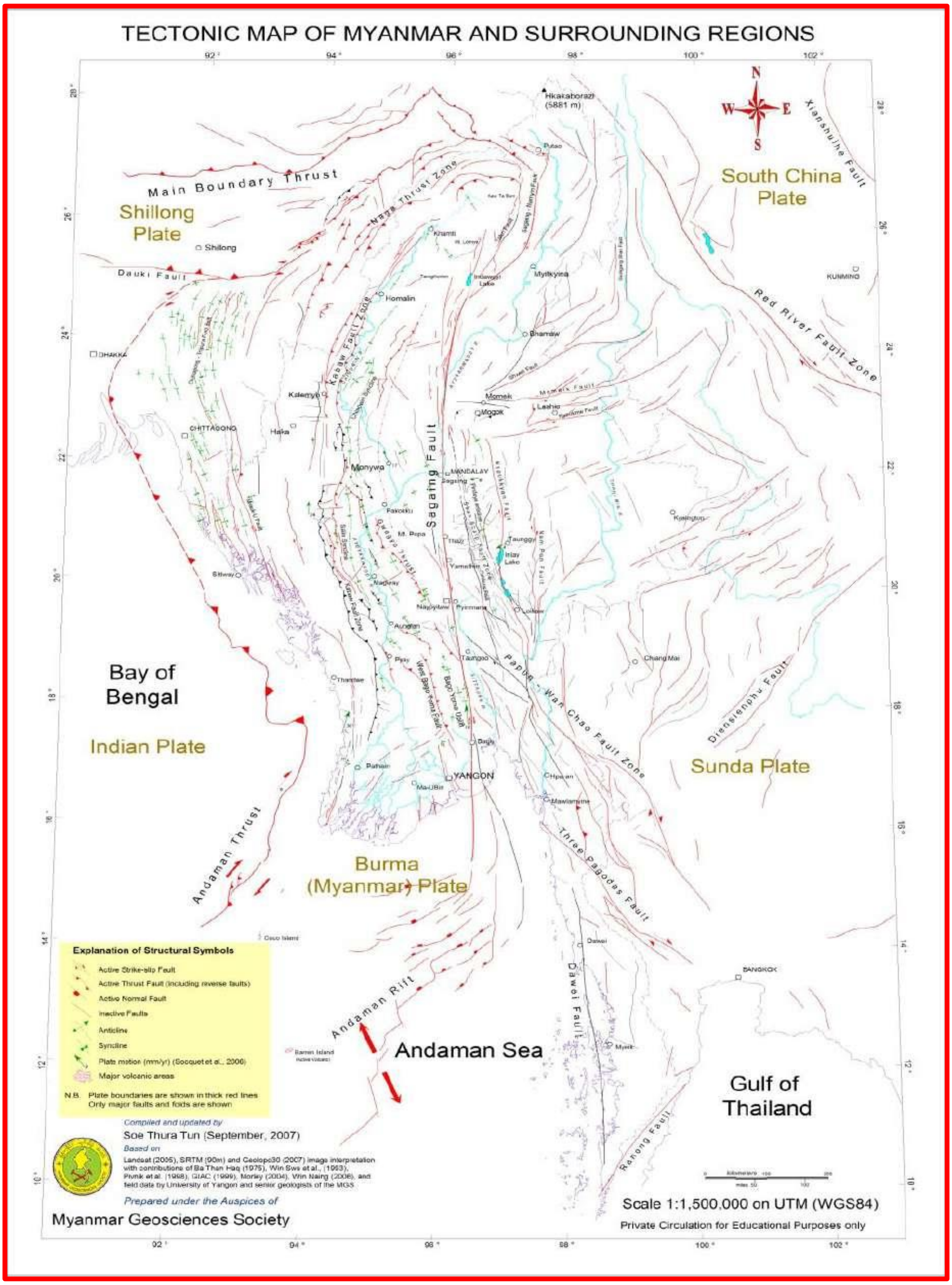
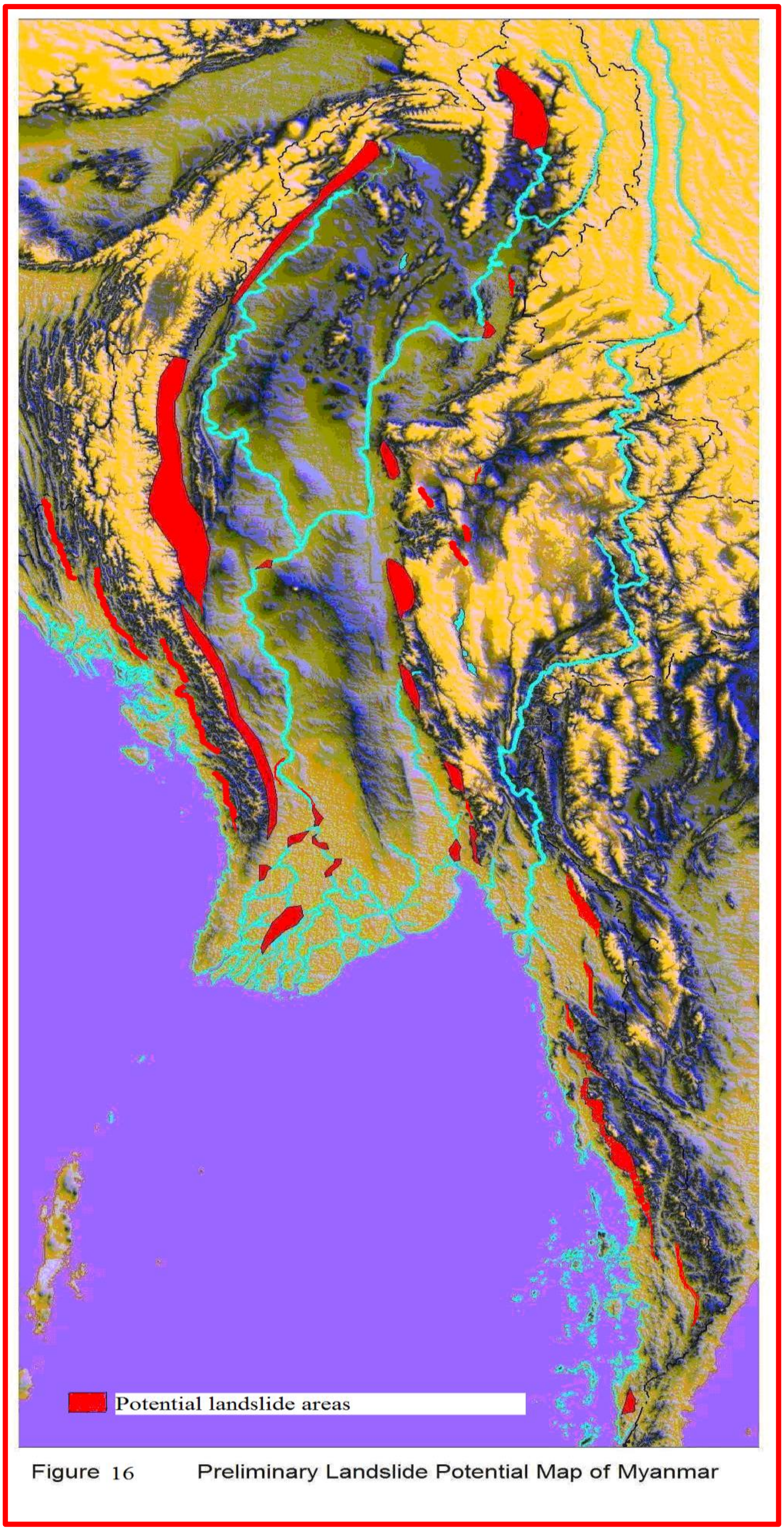


Figure A.3 Potential Landslide Hazard Map of Myanmar (Kyaw Htun, 2011)

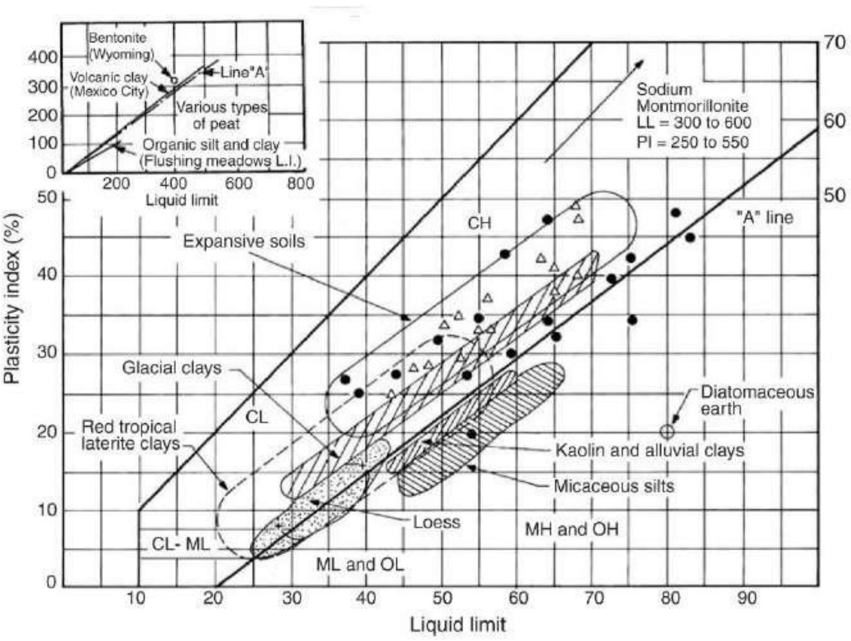


APPENDIX B
UNIFIED SOIL CLASSIFICATION SYSTEM

Table B.1 Unified soil classification system and soil symbols (ASTM D-2487-00)

Major Divisions (1)	(2)	Symbols Letter (3)	Hatching (4)	Color (5)	Name (6)	Value for Embankments (7)	Permeability cm per sec (8)
Coarse-Grained Soils	Gravel and Gravelly Soils	GW		Red	Well-graded gravels or gravel-sand mixtures, little or no fines	Very stable, pervious shells of dikes and dams	$k > 10^{-2}$
		GP		Red	Poorly graded gravels or gravel-sand mixtures, little or no fines	Reasonably stable, pervious shells of dikes and dams	$k > 10^{-2}$
		GM		Yellow	Silty gravels, gravel-sand-silt mixtures	Reasonably stable, not particularly suited to shells, but may be used for impervious cores or blankets	$k = 10^{-3}$ to 10^{-6}
		GC		Yellow	Clayey gravels, gravel-sand-clay mixtures	Fairly stable, may be used for impervious core	$k = 10^{-6}$ to 10^{-8}
	Sand and Sandy Soils	SW		Red	Well-graded sands or gravelly sands, little or no fines	Very stable, pervious sections, slope protection required	$k > 10^{-3}$
		SP		Red	Poorly graded sands or gravelly sands, little or no fines	Reasonably stable, may be used in dike section with flat slopes	$k > 10^{-3}$
		SM		Yellow	Silty sands, sand-silt mixtures	Fairly stable, not particularly suited to shells, but may be used for impervious cores or dikes	$k = 10^{-3}$ to 10^{-6}
		SC		Yellow	Clayey sands, sand-silt mixtures	Fairly stable, use for impervious core or flood-control structures	$k = 10^{-6}$ to 10^{-8}
Fine-Grained Soils	Silts and Clays LL < 50	ML		Green	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	Poor stability, may be used for embankments with proper control	$k = 10^{-3}$ to 10^{-6}
		CL		Green	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	Stable, impervious cores and blankets	$k = 10^{-6}$ to 10^{-8}
		OL		Green	Organic silts and organic silt-clays of low plasticity	Not suitable for embankments	$k = 10^{-4}$ to 10^{-6}
	Silts and Clays LL ≥ 50	MH		Blue	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	Poor stability, core of hydraulic-fill dam, not desirable in rolled-fill construction	$k = 10^{-4}$ to 10^{-6}
		CH		Blue	Inorganic clays of high plasticity, fat clays	Fair stability with flat slopes, thin cores, blankets and dike sections	$k = 10^{-6}$ to 10^{-8}
		OH		Blue	Organic clays of medium to high plasticity, organic silts	Not suitable for embankments	$k = 10^{-6}$ to 10^{-8}
Highly Organic Soils		Pt		Orange	Peat and other highly organic soils	Not used for construction	

Figure B.1 Plasticity Chart for Soil Classification



MNBC -2025 TECHNICAL WORKING GROUP (TWG-4)
Participants List

1.	Dr. Nyan Myint Kyaw	Group Leader
2.	U Kyaw San Win	Coordinator
3.	U Nyunt Oo	Member
4.	U Khin Maung Tint	Member
5.	U Saw Htwe Zaw	Member
6.	U Thaung Sein	Member
7.	U Maung Maung Cho	Member
8.	U Hla Shwe	Member
9.	Dr. Tun Naing	Member
10.	U Aung Myat Kyaw	Member
11.	U Maung Maung Htwe	Member
12.	U Thein Paing	Member
13.	U Khin Maung Lwin	Member
14.	U Hla Phone Maw	Member
15.	Dr. Kay Thwe Tun	Member
16.	U Linn Khine	Member
17.	U Kyaw Tun	Member
18.	U Khin Maung Aye	Member
19.	U Htay Win	Member
20.	Daw Mya Mya Win	Member
21.	Daw Soe Soe Tin	Member
22.	Dr. Zin Zin Htike	Member
23.	U Nay Win	Member
24.	Dr. Min Maung Maung	Member
25.	U Shwe Kyaw Hla	Member
26.	U Thein Htay	Member
27.	U Zaw Naing Oo	Member
28.	U Zarli Chan	Member
29.	Dr. Tin Tin Win	Member
30.	Daw Khine Khine Oo	Member
31.	Daw Mar Mar Swe	Member
32.	U Hla Naing	Member
33.	U Thura Naing	Member
34.	U Thet Soe Oo	Member
35.	Daw San San Thwin	Member

36. Daw May Moe Moe Khine	Member
37. Daw May Sandar Soe	Member
38. Daw Myat Thu Zar Maw	Member