Gratitude

In appreciation and gratitude

to The Custodian of the Two Holy Mosques

King Abdullah Bin Abdul Aziz Al Saud

And

H.R.H. Prince Sultan Bin Abdul Aziz Al Saud

Crown Prince, Deputy Premier, Minister of Defence

& Aviation and Inspector General

For their continuous support and gracious consideration,

the Saudi Building Code National Committee (SBCNC)

is honored to present the first issue of

the Saudi Building Code (SBC).

Saudi Building Code Requirements

201	Architectural	
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PREFACE

The Saudi Building Code (SBC) is a set of legal, administrative and technical regulations and requirements that specify the minimum standards of construction for building in order to ensure public safety and health. A Royal Decree dated 11th June 2000 order the formation of a national committee composed of representatives of Saudi universities and governmental and private sectors. In September 2001, the Council of Ministers approved the general plan of the National Committee to develop a national building code for the Kingdom of Saudi Arabia.

To choose a base code for the Saudi Building Code, a number of Codes have been studied. The National Committee has been acquainted with the results of the national researches and the international codes from the U.S.A., Canada and Australia, also, the European Code, and Arab Codes. It has also sought the opinions of specialists in relevant Saudi universities, governmental and private sectors through holding a questionnaire, a symposium and specialized workshops, in the light of which, (ICC) has been chosen to be a base code for the Saudi Building Code.

The International Code Council (ICC) grants permission to the Saudi Building Code National Committee (SBCNC) to include all or any portion of material from the ICC codes, and standards in the SBC and ICC is not responsible or liable in any way to SBCNC or to any other party or entity for any modifications or changes that SBCNC makes to such documents.

Toward expanding the participation of all the specialists in the building and construction industry in the Kingdom through the governmental and private sectors, the universities and research centers, the National Committee took its own decisions related to code content by holding specialized meetings, symposiums and workshops and by the help of experts from inside and outside of Saudi Arabia.

The technical committees and sub-committees started their work in April 2003 to develop the Saudi Building Code that adapts the base code with the social and cultural environment, the natural and climatic conditions, types of soil and properties of materials in the Kingdom

The Saudi Building Code Structural requirements for Loads and Forces (SBC 301) were developed based on the standards of the American Society of Civil Engineers (SEI/ASCE). The American Society of Civil Engineers, through its Structural Engineering Institute (ASCE/SEI), grants permission to the SBCNC to utilize as reference ASCE 7-02 and ASCE 7-05 in the SBC and to include within the SBC provisions and materials from ASCE 7-02 and ASCE 7-05 modified by SBCNC. ASCE/SEI is not responsible for any modifications or changes that SBCNC has made to the provisions to accommodate local conditions.

The development process of SBC 301 followed the methodology approved by the Saudi Building Code National Committee. Many changes and modifications were made and the most important one was adding the seismic contour maps for Saudi Arabia and some parts and items relating to seismic design outside the intensity of the seismic belt of the Kingdom have been deleted. Only SI Units were used through out the Code.

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CHAPTER 1
GENERAL

SECTION 1.1
SCOPE

1.1.0 The Saudi Building Code for Loading referred to as SBC 301 provides minimum load requirements for the design of buildings and other structures. Loads and appropriate load combinations, which have been developed to be used together, are set forth for strength design and allowable stress design. For design strengths and allowable stress limits, design specifications for conventional structural materials used in buildings and modifications contained in SBC 301 shall be followed.

SECTION 1.2
DEFINITIONS

1.2.0 The following definitions apply to the provisions of the entire SBC 301.

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).

Authority Having Jurisdiction. The organization, office, or individual charged with the responsibility of administering and enforcing the provisions of this code.

Buildings. Structures, usually enclosed by walls and a roof, constructed to provide support or shelter for an intended occupancy.

Design Strength. The product of the nominal strength and a resistance factor.

Essential Facilities. Buildings and other structures that are intended to remain operational in the event of extreme environmental loading from wind, or earthquakes.

Factored Load. The product of the nominal load and a load factor.

Hazardous Material. Chemicals or substances classified as a physical or health hazard whether the chemicals or substances are in a usable or waste condition.

Health Hazard. Chemicals or substances classified by the authority having jurisdiction as toxic, highly toxic, or corrosive.

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

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 all 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. All other loads are variable loads. (see also nominal loads.)

Nominal Loads. The magnitudes of the loads specified in Chapter 3 through 13 (dead, live, soil, wind, rain, flood, and earthquake) of SBC 301.

Nominal Strength. The capacity of a structure or member to resist the effects of loads, as determined by computations using specified material strengths and dimensions and formulas 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.

Occupancy. The purpose for which a building or other structure, or part thereof, is used or intended to be used.

Other Structures. Structures, other than buildings, for which loads are specified in this code.

P-Delta Effect. The second-order effect on shears and moments of frame members induced by axial loads on a laterally displaced building frame.

Physical Hazard. Chemicals or substances in a liquid, solid, or gaseous form that are classified by the authority having jurisdiction as combustible, flammable, explosive, oxidizer, pyrophoric, unstable (reactive), or water reactive.

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 Design. A method of proportioning structural members such that the computed forces produced in the members by the factored loads do not exceed the member design strength (also called load and resistance factor design).

Temporary facilities. Buildings or other structures that are to be in service for a limited time and have a limited exposure period for environmental loadings.

SECTION 1.3
CONSTRUCTION DOCUMENTS

1.3.1 **General.** Construction documents shall show the size, section and relative locations of structural members with floor levels, column centers and offsets fully dimensioned. The design loads and other information pertinent to the structural design required by Sections 1.3.1.1 through 1.3.1.7 shall be clearly indicated on the construction documents for parts of the building or structure.

1.3.1.1 **Floor Live Load.** The uniformly distributed, concentrated and impact floor live load used in the design shall be indicated for floor areas. Live load reduction of the uniformly distributed floor live loads, if used in the design, shall be indicated.

1.3.1.2 **Roof Live Load.** The roof live load used in the design shall be indicated for roof areas (Section 4.9).

1.3.1.3 **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), km/hr.
2. Wind importance factor, *I*, and building category.
3. Wind exposure, if more than one wind exposure is utilized, the wind exposure and applicable wind direction shall be indicated.
4. The applicable internal pressure coefficient.
5. Components and cladding. The design wind pressures in terms of kN/m^2 to be used for the design of exterior component and cladding materials not specifically designed by the registered design professional.

1.3.1.4 **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 Occupancy importance factor, *I*.
2. Mapped spectral response accelerations S_S and S_1.
3. Site class.
4. Design spectral response accelerations S_{DS} and S_{D1}.
5. Seismic design category.
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.

1.3.1.5 **Flood Load.** For buildings located in flood hazard areas as established in Section 5.3, the following information shall be shown, regardless of whether flood loads govern the design of the building:

1. In flood hazard areas not subject to high-velocity wave action, the elevation of proposed lowest floor, including basement.
2. In flood hazard areas not subject to high-velocity wave action, the elevation to which any nonresidential building will be dry flood-proofed.
3. In flood hazard areas subject to high-velocity wave action, the proposed elevation of the bottom of the lowest horizontal structural member of the lowest floor, including basement.

1.3.1.6 **Special Loads.** Special loads that are applicable to the design of the building, structure or portions thereof shall be indicated along with the specified section of this code that addresses the special loading condition.

1.3.1.7 **System and Components Requiring Special Inspections for Seismic Resistance.** Construction documents or specifications shall be prepared for those systems and components requiring special inspection for seismic resistance as specified in SBC 302 by the registered design professional responsible for their design and shall be submitted for approval in accordance with SBC administrative code. Reference to seismic provisions in lieu of detailed drawings is acceptable.

1.3.2 **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 these requirements.

1.3.3 **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 2.50 kN/m^2, 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.

1.3.4 **Occupancy Permits for Changed Loads.** Construction documents for other than residential buildings filed with the building official with applications for permits shall show on each drawing the live loads per square meter (m^2) of area covered for which the building is designed. Occupancy permits for buildings hereafter erected shall not be issued until the floor load signs, required by Section 1.3.3, have been installed.

SECTION 1.4
BASIC REQUIREMENTS

1.4.1 **General.** Building, structures 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 chapters.

1.4.2 **Strength.** Buildings and other structures, and all parts thereof, shall be designed and constructed to support safely the factored loads in load combinations defined in this document without exceeding the appropriate strength limit states for the materials of construction. Alternatively, buildings and other structures, and all parts thereof, shall be designed and constructed to support safely the nominal loads in load combinations defined in this document without exceeding the appropriate specified allowable stresses for the materials of construction.
Loads and forces for occupancies or uses not covered in this chapter shall be subject to the approval of the building official.

1.4.3 **Serviceability.** Structural systems, and members thereof, shall be designed to have adequate stiffness to limit deflections, lateral drift, vibration, or any other deformations that adversely affect the intended use and performance of buildings and other structures. See Section 10.12 for drift limits applicable to earthquake loading.

1.4.3.1 **Deflections.** The deflections of structural members shall not exceed the more restrictive of the limitations of Sections 1.4.3.2 through 1.4.3.4 or that permitted by Table 1.4-1.

TABLE 1.4-1: DEFLECTION LIMITS[a, b, c, g,]

Construction	L	W^e	$D+L^f$
Roof members[d]			
Supporting plaster ceiling	l/360	l/360	l/240
Supporting nonplaster 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

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 1.8 m 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 4.11.

c. See SBC 201 for glass supports.

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 Chapter 8 for rain and ponding requirements and SBC 201 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 cantilever members, l shall be taken as twice the length of the cantilever.

1.4.3.2 **Reinforced Concrete.** The deflection of reinforced concrete structural members shall not exceed that permitted by SBC 304.

1.4.3.3 **Steel.** The deflection of steel structural members shall not exceed that permitted by SBC 306.

1.4.3.4 **Masonry.** The deflection of masonry structural members shall not exceed that permitted by SBC 305.

1.4.4 **Analysis.** Load effects on individual structural members 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.

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 that are assumed not to be a part of the lateral-force-resisting system shall be permitted to be incorporated into buildings provided that their effect on the action of the system is considered and provided for in design. Provisions shall be made for the increased forces induced on resisting elements of the structural system resulting from torsion due to eccentricity between the center of application of the lateral forces and the center of rigidity of the lateral-force-resisting system.

1.4.5 **Counteracting Structural Actions.** All structural members and systems, and all components and cladding in a building or other structure, shall be designed to resist forces due to earthquake, wind, soil and hydrostatic pressure and flood loads, with consideration of overturning, sliding, and uplift, and 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. Where all or a portion of the resistance to these forces is provided by dead load, the dead load shall be taken as the minimum dead load likely to be in place during the event causing the considered forces. Consideration shall be given to the effects of vertical and horizontal deflections resulting from such forces.

1.4.6 **Self-straining Forces.** Provision shall be made for anticipated self-straining forces arising from differential settlements of foundations and from restrained dimensional changes due to temperature, moisture, shrinkage, creep, and similar effects.

SECTION 1.5
GENERAL STRUCTURAL INTEGRITY

1.5.0 Buildings and other structures shall be designed to sustain local damage with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage. This shall be achieved through an arrangement of the structural elements that provides stability to the entire structural system by transferring loads from any locally damaged region to adjacent regions capable of resisting those loads without collapse. This shall be accomplished by providing sufficient continuity, redundancy, or energy-dissipating capacity (ductility), or a combination thereof, in the members of the structure.

SECTION 1.6
CLASSIFICATION OF BUILDINGS AND OTHER STRUCTURES

1.6.1 **Nature of Occupancy.** Buildings and other structures shall be classified, based on the nature of occupancy, according to Table 1.6-1 for the purposes of applying flood, wind, and earthquake provisions. The categories range from I to IV, where Category I represents buildings and other structures with a low hazard to human life in the event of failure and Category IV represents essential facilities. Each building or other structure shall be assigned to the highest applicable category or categories. Assignment of the same structure to multiple categories based on use and the type of load condition being evaluated (e.g., wind, seismic, etc.) shall be permissible.

When buildings or other structures have multiple uses (occupancies), the relationship between the uses of various parts of the building or other structure and the independence of the structural systems for those various parts shall be examined. The classification for each independent structural system of a multiple use building or other structure shall be that of the highest usage group in any part of the building or other structure that is dependent on that basic structural system.

1.6.2 **Hazardous Materials and Extremely Hazardous Materials.** Buildings and other structures containing hazardous materials or extremely hazardous materials are permitted to be classified as Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as part of an overall risk management plan (RMP) that a release of the hazardous material or extremely hazardous material does not pose a threat to the public.

In order to qualify for this reduced classification, the owner or operator of the buildings or other structures containing the hazardous materials or extremely hazardous materials shall have a risk management plan that incorporates three elements as a minimum: a hazard assessment, a prevention program, and an emergency response plan.

As a minimum, the hazard assessment shall include the preparation and reporting of worst-case release scenarios for each structure under consideration, showing the potential effect on the public for each. As a minimum, the worst-case event shall include the complete failure (instantaneous release of entire contents) of a vessel, piping system, or other storage structure. A worst-case event includes (but is not limited to) a release during the design wind or design seismic event. In this assessment, the evaluation of the effectiveness of subsequent measures for accident mitigation shall be based on the assumption that the complete failure of the primary storage structure has occurred. The off-site impact must be defined in terms of population within the potentially affected area. In order to qualify for the reduced classification, the hazard assessment shall demonstrate that a release of the hazardous material from a worst-case event does not pose a threat to the public outside the property boundary of the facility.

As a minimum, the prevention program shall consist of the comprehensive elements of process safety management, which is based on accident prevention through the application of management controls in the key areas of design, construction, operation, and maintenance. Secondary containment of the hazardous materials or extremely hazardous materials (including, but not limited to, double wall tank, dike of sufficient size to contain a spill, or other means to contain a release of the hazardous materials or extremely hazardous material within the property boundary of the facility and prevent release of harmful quantities of contaminants to the air, soil, ground water, or surface water) are permitted to be used to mitigate the risk of release. When secondary containment is provided, it shall be designed for all environmental loads and is not eligible for this reduced classification.

As a minimum, the emergency response plan shall address public notification, emergency medical treatment for accidental exposure to humans, and procedures for emergency response to releases that have consequences beyond the property boundary of the facility. The emergency response plan shall address the potential that resources for response could be compromised by the event that has caused the emergency.

SECTION 1.7
ADDITIONS AND ALTERATIONS TO EXISTING STRUCTURES

1.7.0 When an existing building or other structure is enlarged or otherwise altered, structural members affected shall be strengthened if necessary so that the factored loads defined in this document will be supported without exceeding the specified design strength for the materials of construction. When using allowable stress design, strengthening is required when the stresses due to nominal loads exceed the specified allowable stresses for the materials of construction.

SECTION 1.8
LOAD TESTS

1.8.1 **In-situ Load Tests.** The building official is authorized to require an engineering analysis or a load test, or both, of any construction whenever there is reason to question the safety of the construction for the intended occupancy or use. Engineering analysis and load tests shall be conducted in accordance with SBC 302.

1.8.2 **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 specified material design standards or alternative test procedures in accordance with SBC 302, shall be load tested in accordance with SBC 302.

SECTION 1.9
ANCHORAGE

1.9.1 **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.

1.9.2 **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 chapter but not less than a minimum strength design horizontal force of 4.0 kN/m of wall, substituted for "*E*" in the load combinations of Section 2.3 or 2.4. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 1.2 m. 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 7.1 and 10.11 for wind and earthquake design requirements.

TABLE 1.6-1:
CLASSIFICATION OF BUILDINGS AND OTHER STRUCTURES FOR FLOOD, WIND AND EARTHQUAKE LOADS

Nature of Occupancy	Category
1) Buildings and other structures that represent a low hazard to human life in the event of failure including, but not limited to: a) Agricultural facilities b) Certain temporary facilities c) Minor storage facilities	I
All buildings and other structures except those listed in Categories I, III, and IV	II
1) Buildings and other structures that represent a substantial hazard to human life in the event of failure including, but not limited to: a) Buildings and other structures where more than 300 people congregate in one area b) Buildings and other structures with day care facilities with capacity greater than 150 c) Buildings and other structures with elementary school or secondary school facilities with capacity greater than 250 d) Buildings and other structures with a capacity greater than 500 for colleges or adult education facilities e) Health care facilities with a capacity of 50 or more resident patients but not having surgery or emergency treatment facilities f) Jails and detention facilities g) Power generating stations and other public utility facilities not included in Category IV 2) Buildings and other structures not included in Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing sufficient quantities of hazardous materials to be dangerous to the public if released. 3) Buildings and other structures containing hazardous materials shall be eligible for classification as Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.6.2 that a release of the hazardous material does not pose a threat to the public.	III
1) Buildings and other structures designated as essential facilities including, but not limited to: a) Hospitals and other health care facilities having surgery or emergency treatment facilities b) Fire, rescue, ambulance, and police stations and emergency vehicle garages c) Designated earthquake, hurricane, or other emergency shelters d) Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response e) Power generating stations and other public utility facilities required in an emergency f) Ancillary structures (including, but not limited to, communication towers, fuel storage tanks, cooling towers, electrical substation structures, fire water storage tanks or other structures housing or supporting water, or other fire-suppression material or equipment) required for operation of Category IV structures during an emergency g) Aviation control towers, air traffic control centers, and emergency aircraft hangars h) Water storage facilities and pump structures required to maintain water pressure for fire suppression i) Buildings and other structures having critical national defense functions 2) Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing extremely hazardous materials where the quantity of the material exceeds a threshold quantity established by the authority having jurisdiction. 3) Buildings and other structures containing extremely hazardous materials shall be eligible for classification as Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.6.2 that a release of the extremely hazardous material does not pose a threat to the public. This reduced classification shall not be permitted if the buildings or other structures also function as essential facilities.	IV

CHAPTER 2
COMBINATION OF LOADS

SECTION 2.1
GENERAL

Buildings and other structures shall be designed using the provisions of either Section 2.3 or 2.4. Either Section 2.3 or 2.4 shall be used exclusively for proportioning elements of a particular construction material throughout the structure.

SECTION 2.2
SYMBOLS AND NOTATIONS

D = dead load;
E = earthquake load;
F = load due to fluids with well-defined pressures and maximum heights;
F_a = flood load;
H = load due to lateral earth pressure, ground water pressure, or pressure of bulk materials;
L = live load;
L_r = roof live load;
P = ponding load;
R = rain load;
T = self-straining force;
W = wind load;

SECTION 2.3
COMBINING FACTORED LOADS USING STRENGTH DESIGN

2.3.1 **Applicability.** The load combinations and load factors given in Section 2.3.2 shall be used only in those cases in which they are specifically authorized by the applicable material design standard.

2.3.2 **Basic Combinations.** Structures, components, and foundations shall be designed so that their design strength equals or exceeds the effects of the factored loads in the following combinations:

$1.4\ (D + F)$ **(Eq. 2.3.2-1)**

$1.2\ (D + F + T) + 1.6\ (L + H) + 0.5\ (L_r \text{ or } R)$ **(Eq. 2.3.2-2)**

$1.2\ D + 1.6\ (L_r \text{ or } R) + (f_1 L \text{ or } 0.8\ W)$ **(Eq. 2.3.2-3)**

$1.2D + 1.6W + f_1 L + 0.5\ (L_r \text{ or } R)$ **(Eq. 2.3.2-4)**

$1.2D + 1.0\ E + f_1 L$ **(Eq. 2.3.2-5)**

$0.9D + 1.6W + 1.6H$ **(Eq. 2.3.2-6)**

$0.9D + 1.0E + 1.6H$ **(Eq. 2.3.2-7)**

where

f_1 = 1.0 for areas occupied as places of public assembly, for live loads in excess of 5.0 kN/m^2, and for parking garage live load.

f_1 = 0.5 for other live loads.

Exceptions:

1. The load factor on H shall be set equal to zero in (Eq. 2.3.2-6) and (Eq. 2.3.2-7) 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.
2. For Concrete structures and Masonry construction (Eq. 2.3.2-2) shall be

$$1.4\ (D + F + T) + 1.7\ (L + H) + 0.5\ (L_r \text{ or } R)$$

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. Refer to Section 9.3 for specific definition of the earthquake load effect E.

2.3.3 **Load Combinations Including Flood Load.** When a structure is located in a flood zone (Section 5.3.1), the following load combinations shall be considered:

1. In V-Zones or Coastal A-Zones, 1.6W in (Eq. 2.3.2-4) and (Eq. 2.3.2-6) shall be replaced by $1.6W + 2.0\ F_a$.
2. In noncoastal A-Zones, 1.6W in (Eq. 2.3.2-4) and (Eq. 2.3.2-6) shall be replaced by $0.8W + 1.0\ F_a$.

SECTION 2.4
COMBINING NOMINAL LOADS USING ALLOWABLE STRESS DESIGN

2.4.1 **Basic 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.

$$D + F \qquad \textbf{(Eq. 2.4.1-1)}$$

$$D + H + F + L + T \qquad \textbf{(Eq. 2.4.1-2)}$$

$$D + H + F + (L_r \text{ or } R) \qquad \textbf{(Eq. 2.4.1-3)}$$

$$D + H + F + 0.75\ (L + T) + 0.75\ (L_r \text{ or } R) \qquad \textbf{(Eq. 2.4.1-4)}$$

$$D + H + F + (W \text{ or } 0.7E) \qquad \textbf{(Eq. 2.4.1-5)}$$

$$D + H + F + 0.75\ (W \text{ or } 0.7\ E) + 0.75\ L + 0.75\ (L_r \text{ or } R) \qquad \textbf{(Eq. 2.4.1-6)}$$

$$0.6D + W + H \qquad \textbf{(Eq. 2.4.1-7)}$$

$$0.6D + 0.7E + H \qquad \textbf{(Eq. 2.4.1-8)}$$

The most unfavorable effects from both wind and earthquake loads shall be considered, where appropriate, but they need not be assumed to act simultaneously. Refer to Section 9.3 for the specific definition of the earthquake load effect E.

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

2.4.1.1 **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 5.3.1), 1.5 F_a shall be added to other loads in (Eq. 2.4.1-5), (Eq. 2.4.1-6) and (Eq. 2.4.1-7) and E shall be set equal to zero in (Eq. 2.4.1-5) and (Eq. 2.4.1-6).
2. In noncoastal A-Zones, $0.75F_a$ shall be added to combinations (Eq. 2.4.1-5), (Eq. 2.4.1-6) and (Eq. 2.4.1-7) and E shall be set equal to zero in (Eq. 2.4.1-5) and (Eq. 2.4.1-6).

2.4.2 **Alternative basic load combinations.** In lieu of the basic load combinations specified in Section 2.4.1, and as required by SBC 303, structures and portions thereof shall be permitted to be designed for the most critical effects resulting from the following combinations. When using these alternate basic load combinations that include wind or seismic loads, allowable stresses are permitted to be increased or load combinations reduced, where permitted by the material section of this code or referenced standard.

$D + L + (L_r \text{ or } R)$ **(Eq. 2.4.2-1)**

$D + L + 1.3\ W$ **(Eq. 2.4.2-2)**

$D + L + 1.3\ W$ **(Eq. 2.4.2-3)**

$D + L + 1.3\ W/2$ **(Eq. 2.4.2-4)**

$D + L + E/1.4$ **(Eq. 2.4.2-5)**

$0.9D + E/1.4$ **(Eq. 2.4.2-6)**

Exceptions:
Crane hook loads need not be combined with roof live load or one-half of the wind load.

2.4.2.1 **Other loads.** Where *F*, *H*, *P* or *T* are to be considered in design, 1.0 times each applicable load shall be added to the combinations specified in Section 2.4.2.

SECTION 2.5
SPECIAL SEISMIC LOAD COMBINATIONS

For both allowable stress design and strength design methods, where specifically required by Chapters 9 through 16 or by SBC 303, SBC 304, SBC 305, and SBC 306 or Ref. 11.1-1, elements and components shall be designed to resist the forces calculated using Equation 2.5-1 when the effects of the seismic ground motion are additive to gravity forces and those calculated using Equation 2.5-2 when the effects of the seismic ground motion counteract gravity forces.

$1.2D + f_1L + E$ **(Eq. 2.5-1)**

$0.9D + E$ **(Eq. 2.5-2)**

where:

E = The maximum effect of horizontal and vertical forces as per section 10.4.1.

f_1 = 1.0 for areas occupied as places of public assembly, for live loads in excess of 5.0 kN/m^2 and for parking garage live load.

f_1 = 0.5 for other live loads.

SECTION 2.6
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.

CHAPTER 3
DEAD LOADS

SECTION 3.1
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, stairways, built-in partitions, finishes, cladding and other similarly incorporated architectural and structural items, and fixed service equipment including the weight of cranes.

SECTION 3.2
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. The minimum design dead loads are shown in Table 3-1 and Table 3-1(a) and the minimum densities for design loads from materials are shown in Table 3-2.

SECTION 3.3
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, heating, ventilating and air conditioning systems (HVAC) and fire sprinkler systems shall be included.

TABLE 3-1: MINIMUM DESIGN DEAD LOADS

Component	Load (kN/m^2)	Component	Load (kN/m^2)
COVERINGS, ROOF, AND WALL		**FLOORS AND FLOOR FINISHES**	
Asbestos-cement shingles	0.20	Asphalt block (50 mm), 13 mm mortar	1.45
Asphalt shingles	0.10	Cement finish (25 mm) on stone-concrete fill	1.55
Cement tile	0.80	Ceramic or quarry tile (20 mm) on 13 mm mortar bed	0.80
Clay tile (for mortar add 0.50 kN/m^2)			
Book tile, 50 mm	0.60	Ceramic or quarry tile (20 mm) on 25 mm mortar bed	1.10
Book tile, 75 mm	1.00		
Ludowici	0.50	Concrete fill finish (per mm thickness)	0.023
Roman	0.60	Hardwood flooring, 20 mm	0.20
Spanish	0.90	Linoleum or asphalt tile, 6 mm	0.05
Composition:		Marble and mortar on stone-concrete fill	1.60
Three-ply ready roofing	0.05	Slate (per mm thickness)	0.03
Four-ply felt and gravel	0.25	Solid flat tile on 25 mm mortar base	1.10
Five-ply felt and gravel	0.30	Subflooring, 20 mm	0.15
Copper or tin	0.05	Terrazzo (38 mm) directly on slab	0.90
Corrugated asbestos-cement roofing	0.20	Terrazzo (25 mm) on stone-concrete fill	1.55
Deck, metal, 20 gage	0.10	Terrazzo (25 mm), on 50 mm stone concrete	1.55
Deck, metal, 18 gage	0.15	Wood block (75 mm) on mastic, no fill	0.50
Decking, 50 mm wood (Douglas fir)	0.25	Wood block (75 mm) on 13 mm mortar base	0.80
Decking, 75 mm wood (Douglas fir)	0.40	**FRAME PARTITIONS**	
Fiberboard, 13 mm	0.05	Movable steel partitions	0.20
Gypsum sheathing, 13 mm	0.10	Wood or steel studs, 13 mm gypsum board each side	0.40
Insulation, roof boards (per mm thickness)		Wood studs, 50 x 100, unplastered	0.20
Cellular glass	0.0015	Wood studs, 50 x 100, plastered one side	0.60
Fibrous glass	0.002	Wood studs, 50 x 100, plastered two sides	1.00
Fiberboard	0.003	**FRAME WALLS**	
Perlite	0.0015	Exterior stud walls with brick veneer	2.30
Polystyrene foam	0.0005	Windows, glass, frame and sash	0.40
Urethane foam with skin	0.001	Clay brick wythes:	
Plywood (per mm thickness)	0.006	100 mm	1.90
Rigid insulation, 13 mm	0.05	200 mm	3.80
Skylight, metal frame, 10 mm wire glass	0.40	300 mm	5.50
		400 mm	7.50
Slate, 5 mm	0.35	**CEILINGS**	
Slate, 6 mm	0.50	Acoustical fiberboard	0.05
Waterproofing membranes:		Gypsum board (per mm thickness)	0.01
Bituminous, gravel-covered	0.30	Mechanical duct allowance	0.20
Bituminous, smooth surface	0.10	Plaster on tile or concrete	0.25
Liquid applied	0.05	Plaster on wood lath	0.40
Single-ply, sheet	0.03	Suspended steel channel system	0.10
Wood sheathing (per mm thickness)	0.006	Suspended metal lath and cement plaster	0.75
Wood shingles	0.15	Suspended metal lath and gypsum plaster	0.50
FLOOR FILL		Wood furring suspension system	0.15
Cinder concrete, per mm	0.017		
Lightweight concrete, per mm	0.015		
Sand, per mm	0.015		
Stone concrete, per mm	0.023		

TABLE 3-1(a):
MINIMUM DESIGN DEAD LOADS*FOR
DIFFERENT THICKNESS OF MASONRY WALLS, (kN/m^2)

Component thickness	100 mm	150 mm	200 mm	250 mm	300 mm
Hollow concrete masonry unit wythes:					
Density of unit (16.5 kN/m^3)					
No grout	1.05	1.30	1.70	2.00	2.35
1200 mm grout		1.50	1.95	2.35	2.80
1000 mm		1.60	2.05	2.55	3.00
800 mm spacing		1.65	2.15	2.70	3.15
600 mm		1.80	2.35	2.95	3.45
400 mm		2.00	2.70	3.35	4.00
Full grout		2.75	3.70	4.70	5.70
Density of unit (19.5 kN/m^3):					
No grout	1.25	1.35	1.70	2.10	2.40
1200 mm grout		1.60	2.10	2.60	3.00
1000 mm		1.65	2.15	2.70	3.10
800 mm spacing		1.70	2.25	2.80	3.25
600 mm		1.90	2.45	3.00	3.60
400 mm		2.10	2.80	3.50	4.20
Full grout		2.80	3.90	4.90	5.90
Density of unit (21.0 kN/m^3)					
No grout	1.40	1.70	2.15	2.60	3.00
1200 mm grout		1.60	2.40	2.90	3.45
1000 mm		1.70	2.55	3.10	3.70
800 mm spacing		1.80	2.65	3.25	3.85
600 mm		2.00	2.80	3.50	4.10
400 mm		2.25	3.15	3.95	4.70
Full grout		3.05	4.15	5.25	6.40
Solid concrete masonry unit wythes (incl. concrete brick):					
Density of unit (16.5 kN/m^3):	1.55	2.35	3.20	4.00	4.90
Density of unit (19.5 kN/m^3):	1.85	2.85	3.80	4.80	5.80
Density of unit (21.0 kN/m^3):	2.00	3.00	4.15	5.15	6.25

* Weights of masonry include mortar but not plaster. For plaster, add 0.25 kN/m^2 for each face plastered. Values given represent averages. In some cases, there is a considerable range of weight for the same construction.

TABLE 3-2:
MINIMUM DENSITIES FOR DESIGN LOADS FROM MATERIALS

Material	Density (kN/m^3)	Material	Density (kN/m^3)
Aluminum	26.5	Earth (submerged)	
Bituminous products		Clay	12.5
Asphaltum	13.0	Soil	11.0
Graphite	21.0	River mud	14.0
Paraffin	9.0	Sand or gravel	9.5
Petroleum, crude	8.5	Sand or gravel and clay	10.0
Petroleum, refined	8.0	Glass	25.0
Petroleum, benzine	7.0	Gravel, dry	16.5
Petroleum, gasoline	6.5	Gypsum, loose	11.0
Pitch	11.0	Gypsum, wallboard	8.0
Tar	12.0	Ice	9.0
Brass	82.5	Iron	
Bronze	87.0	Cast	71.0
Cast-stone masonry (cement, stone, sand)	23.0	Wrought	75.5
Cement, portland, loose	14.0	Lead	111.5
Ceramic tile	23.5	Lime	
Charcoal	2.0	Hydrated, loose	5.0
Cinder fill	9.0	Hydrated, compacted	7.0
Cinders, dry, in bulk	7.0	Masonry, Ashlar stone	
Coal		Granite	26.0
Anthracite, piled	8.0	Limestone, crystalline	26.0
Bituminous, piled	7.5	Limestone, oolitic	21.0
Lignite, piled	7.5	Marble	27.0
Peat, dry, piled	3.5	Sandstone	23.0
Concrete, plain		Masonry, brick	
Cinder	17.0	Hard (low absorption)	20.5
Expanded-slag aggregate	16.0	Medium (medium absorption)	18.0
Haydite (burned-clay aggregate)	14.0	Soft (high absorption)	16.0
Slag	21.0	Masonry, concrete*	
Stone (including gravel)	23.0	Lightweight units	16.5
Vermiculite and perlite aggregate, non-load-bearing	4.0-8.0	Medium weight units	19.5
Other light aggregate, load-bearing	11.0-16.5	Normal weight units	21.0
Concrete, reinforced		Masonry grout	22.0
Cinder	17.5	Masonry, rubble stone	
Slag	22.0	Granite	24.0
Stone (including gravel)	24.0	Limestone, crystalline	23.0
Copper	87.5	Limestone, oolitic	22.0
Cork, compressed	2.0	Marble	24.5
Earth (not submerged)		Sandstone	21.5
Clay, dry	10.0	Mortar, cement or lime	20.5
Clay, damp	17.5	Particleboard	7.0
Clay and gravel, dry	16.0	Plywood	6.0
Silt, moist, loose	12.5	Riprap (Not submerged)	
Silt, moist, packed	15.0	Limestone	13.0
Silt, flowing	17.0	Sandstone	14.0
Sand and gravel, dry, loose	16.0	Sand	
Sand and gravel, dry, packed	17.5	Clean and dry	14.0
Sand and gravel, wet	19.0	River, dry	17.0

(continued)

TABLE 3-2:
MINIMUM DENSITIES FOR DESIGN LOADS FROM MATERIALS - continued

Material	Density (kN/m^3)	Material	Density (kN/m^3)
Slag		Tin	72.0
Bank	11.0	Water	
Bank screenings	17.0	Fresh	10.0
Machine	15.0	Sea	10.0
Sand	8.0	Wood, seasoned	
Slate	27.0	Ash, commercial white	6.5
Steel, cold-drawn	77.5	Cypress, southern	5.5
Stone, quarried, piled		Fir, Douglas, coast region	5.5
Basalt, granite, gneiss	15.0	Hem fir	4.5
Limestone, marble, quartz	15.0	Oak, commercial reds and whites	7.5
Sandstone	13.0	Pine, southern yellow	6.0
Shale	14.5	Redwood	4.5
Greenstone, hornblende	17.0	Spruce, red, white, and Stika	4.5
Terra cotta, architectural		Western hemlock	5.0
Voids filled	19.0	Zinc, rolled sheet	70.5
Voids unfilled	11.5		

*Tabulated values apply to solid masonry and to the solid portion of hollow masonry.

CHAPTER 4
LIVE LOADS

SECTION 4.1
DEFINITION

Live loads are 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 on a roof are those 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.

SECTION 4.2
UNIFORMLY DISTRIBUTED LOADS

4.2.1 **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 4-1 and Table 4-2.

4.2.2 **Provision for Partitions.** In office buildings or other buildings where partitions will be erected or rearranged partition weight shall be considered, whether or not partitions are shown on the plans, unless the specified live load exceeds (4 kN/m^2).

SECTION 4.3
CONCENTRATED LOADS

Floors and other similar surfaces shall be designed to support safely the uniformly distributed live loads prescribed in Section 4.2 or the concentrated load, in kilonewtons (kN), given in Table 4-1, whichever produces the greater load effects. Unless otherwise specified, the indicated concentration shall be assumed to be uniformly distributed over an area (750 mm x 750 mm) and shall be located so as to produce the maximum load effects in the structural members.

Any single panel point of the lower chord of exposed roof trusses or any point along the primary structural members supporting roofs over manufacturing, commercial storage and warehousing, and commercial garage floors shall be capable of carrying safely a suspended concentrated load of not less than 9.0 kN in addition to dead load. For all other occupancies, a load of 0.9 kN shall be used instead of 9.0 kN.

SECTION 4.4
LOADS ON HANDRAILS, GUARDRAIL SYSTEMS, GRAB BAR SYSTEMS, VEHICLE BARRIER SYSTEMS, AND FIXED LADDERS

4.4.1 **Definitions.**

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

Fixed Ladder. A ladder that is permanently attached to a structure, building, or equipment.

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.

Grab Bar System. A bar provided to support body weight in locations such as toilets, showers, and tub enclosures.

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.

4.4.2 Loads.

(a) Handrail assemblies and guardrail systems shall be designed to resist a load of 0.75 kN/m applied in any direction at the top and to transfer this load through the supports to the structure. For one- and two-family dwellings, the minimum load shall be 0.3 kN/m.

Further, all handrail assemblies and guardrail systems shall be able to resist a single concentrated load of 0.9 kN applied in any direction at any point along the top, and have attachment devices and supporting structure to transfer this loading to appropriate structural elements of the building. This load need not be assumed to act concurrently with the loads specified in the preceding paragraph.

Intermediate rails (all those except the handrail), balusters, and panel fillers shall be designed to withstand a horizontally applied normal load of (0.2 kN) on an area not to exceed (300 mm x 300 mm) including openings and space between rails. Reactions due to this loading are not required to be superimposed with those of either preceding paragraph.

(b) Grab bar systems shall be designed to resist a single concentrated load of (1.1 kN) applied in any direction at any point.

(c) Vehicle barrier systems for passenger cars shall be designed to resist a single load of (27.0 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 (450 mm) above the floor or ramp surface on an area not to exceed (300 mm x 300 mm), and is not required to be assumed to act concurrently with any handrail or guardrail loadings specified in the preceding paragraphs of Section 4.4.2. Garages accommodating trucks and buses shall be designed in accordance with an approved method, which contains provision for traffic railings.

(d) The minimum design live load on fixed ladders with rungs shall be a single concentrated load of 1.5 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 1.5 kN for every 3 m of ladder height.

(e) 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 0.5 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 4-1.

SECTION 4.5
LOADS NOT SPECIFIED

For occupancies or uses not designated in Section 4.2 or 4.3, the live load shall be determined in accordance with a method approved by the building official.

SECTION 4.6
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.

SECTION 4.7
IMPACT LOADS

The live loads specified in Sections 4.2.1 and 4.4.2 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.

4.7.1 **Elevators.** All elevator loads shall be increased by 100% for impact and the structural supports shall be designed within the limits of deflection prescribed by Refs. 4-1 and 4-2.

4.7.2 **Machinery.** For the purpose of design, the weight of machinery and moving loads shall be increased as follows to allow for impact:

(1) elevator machinery, 100%;

(2) light machinery, shaft- or motor-driven, 20%;

(3) reciprocating machinery or power-driven units, 50%;

(4) hangers for floors or balconies, 33%. All percentages shall be increased where specified by the manufacturer.

SECTION 4.8
REDUCTION IN LIVE LOADS

The minimum uniformly distributed live loads, L_o in Table 4-1, may be reduced according to the following provisions.

4.8.1 **General.** Subject to the limitations of Sections 4.8.2 through 4.8.5, members for which a value of $K_{LL}A_T$ is 37.0 m^2 or more are permitted to be designed for a reduced live load in accordance with the following formula:

$$L = L_o \left(0.25 + \frac{4.57}{\sqrt{K_{LL} A_T}} \right) \qquad \textbf{(Eq. 4-1)}$$

where

L = reduced design live load per square m of area supported by the member.

L_o = unreduced design live load per square m of area supported by the member (see Table 4-1)

K_{LL} = live load element factor (see Table 4-3).

A_T = tributary area m^2

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

4.8.2 **Heavy Live Loads.** Live loads that exceed 5 kN/m^2 shall not be reduced except the live loads for members supporting two or more floors may be reduced by 20%.

4.8.3 **Passenger Car Garages.** The live loads shall not be reduced in passenger car garages except the live loads for members supporting two or more floors may be reduced by 20%.

4.8.4 **Special Occupancies.** Live loads of 5 kN/m^2 or less shall not be reduced in public assembly occupancies.

4.8.5 **Limitations on One-Way Slabs.** The tributary area, A_T, for one-way slabs shall not exceed an area defined by the slab span times a width normal to the span of 1.5 times the slab span.

SECTION 4.9
MINIMUM ROOF LIVE LOADS

4.9.1 **Flat, Pitched, and Curved Roofs.** Ordinary flat, pitched, and curved roofs shall be designed for the live loads specified in Eq. 4-2 or other controlling combinations of loads as discussed in Chapter 2, 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. 4-2 shall not be used unless approved by the authority having jurisdiction.

$$L_r = 1.0\ R_1\ R_2 \text{ where } 0.6 \le L_r \le 1.0 \qquad \textbf{(Eq. 4-2)}$$

Where:

L_r = roof live load/m^2 of horizontal projection for a non-accessible roof, kN/m^2.

The reduction factors R_1 and R_2 shall be determined as

$$R_1 = \begin{cases} 1 & \text{for } A_t \le 18.0\ m^2 \\ 1.2 - 0.0111\ A_t & \text{for } 18.0\ m^2 < A_t < 54\ m^2 \\ 0.6 & \text{for } A_t \ge 54\ m^2 \end{cases}$$

where A_t = tributary area m^2 supported by any structural member and

$$R_2 = \begin{cases} 1 & \text{for } F \le 4 \\ 1.2 - 0.05\ F & \text{for } 4 < F < 12 \\ 0.6 & \text{for } F \ge 12 \end{cases}$$

where, for a pitched roof, F = 0.12 x slope, with slope expressed in percentage points and, for an arch or dome, F = rise-to-span ratio multiplied by 32.

4.9.2 **Special-Purpose Roofs.** Roofs used for promenade purposes shall be designed for a minimum live load of 3.0 kN/m^2. Roofs used for roof gardens or assembly purposes shall be designed for a minimum live load of 5 kN/m^2. Roofs used for other special purposes shall be designed for appropriate loads as approved by the authority having jurisdiction.

4.9.3 **Special Structural Elements.** Live loads of 5 kN/m^2 or less shall not be reduced for roof members except as specified in Section 4.9.

SECTION 4.10
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.

4.10.1 **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.

4.10.2 **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

4.10.3 **Lateral Force.** The lateral force on crane runway beams with electrically powered trolleys shall be calculated as 20% 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 with due regard to the lateral stiffness of the runway beam and supporting structure.

4.10.4 **Longitudinal Force.** The longitudinal force on crane runway beams, except for bridge cranes with hand-geared bridges, shall be calculated as 10% 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.

TABLE 4-1:
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o, AND MINIMUM CONCENTRATED LIVE LOADS

Occupancy or Use	Uniform kN/m^2	Conc. kN
Apartments (see residential)		
Access floor systems		
Office use	2.5	9
Computer use	5	9
Armories and drill rooms	7.5	
Assembly areas and theaters		
• Fixed seats (fastened to floor)	3	
• Lobbies	5	
• Movable seats	5	
• Platforms (assembly)	5	
• Stage floors	7.5	
Balconies (exterior)	5	
On one- and two-family residences only, and not exceeding 10 m^2	3	
Bowling alleys, poolrooms, and similar recreational areas	4	
Catwalks for maintenance access	2	1.5
Corridors		
First floor	5	
Other floors, same as occupancy served except as indicated		
Mosques	5	
Decks (patio and roof)		
Same as area served, or for the type of occupancy accommodated		
Dining rooms and restaurants	5	
Dwellings (see residential)		
Elevator machine room grating (on area of 2500 mm^2)		1.5
Finish light floor plate construction (on area of 650 mm^2)		1
Fire escapes	5	
Fixed ladders	See Section 4.4	
Garages (passenger vehicles only)	2	Note (1)
Trucks and buses	Note (2)	Note (2)
Grandstands (see stadium and arena bleachers)		
Gymnasiums, main floors, and balconies	5 Note (4)	
Handrails, guardrails, and grab bars	See Section 4.4	
Hospitals		
• Operating rooms, laboratories	3	4.5
• Private rooms	2	4.5
• Wards	2	4.5
• Corridors above first floor	4	4.5
Hotels (see residential)		
Libraries		
• Reading rooms	3	4.5
• Stack rooms	7.5 Note (3)	4.5
• Corridors above first floor	4	4.5
Manufacturing		
• Light	6	9
• Heavy	12	13.5
Marquees and canopies	4	
Office buildings		
• File and computer rooms shall be designed for heavier loads based on anticipated occupancy:		
• Lobbies and first floor corridors	5	9
• Offices	2.5	9
• Corridors above first floor	4.0	9
Penal institutions		
Cell blocks	2	
Corridors	5	

TABLE 4-1:
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o,
AND MINIMUM CONCENTRATED LIVE LOADS – continued

Occupancy or Use	Uniform kN/m^2	Conc. kN
Residential		
Dwellings (one- and two-family)		
Uninhabitable attics without storage	0.5	
Uninhabitable attics with storage	1.0	
Habitable attics and sleeping areas	1.5	
All other areas except stairs and balconies	2.0	
Hotels and multifamily houses		
Private rooms and corridors serving them	2.0	
Public rooms and corridors serving them	5.0	
Reviewing stands, grandstands, and bleachers	5.0 Note (4)	
Roofs	See Sections 4.3 and 4.9	
Schools		
Classrooms	3	4.5
Corridors above first floor	4	4.5
First floor corridors	5	4.5
Scuttles, skylight ribs, and accessible ceilings		10
Sidewalks, vehicular driveways, and yards subject to trucking	12 Note (5)	36 Note (6)
Stadiums and arenas		
Bleachers	5 Note (4)	
Fixed Seats (fastened to floor)	3 Note (4)	
Stairs and exit-ways	5	Note (7)
One- and two-family residences only	2	
Storage areas above ceilings	1	
Storage warehouses (shall be designed for heavier loads if required for anticipated storage)		
Light	6	
Heavy	12	
Stores		
Retail		
First floor	5	4.5
Upper floors	4	4.5
Wholesale, all floors	6	4.5
Vehicle barriers	See Section 4.4	
Walkways and elevated platforms (other than exit-ways)	3	
Yards and terraces, pedestrians	5	

Notes

(1) Floors in garages or portions of building used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of Table 4-1 or the following concentrated load: (1) for garages restricted to passenger vehicles accommodating not more than nine passengers, 13.0 kN acting on an area of 100 mm by 100 mm, footprint of a jack; (2) for mechanical parking structures without slab or deck which are used for storing passenger car only, 10 kN per wheel.

(2) Garages accommodating trucks and buses shall be designed in accordance with an approved method, which contains provisions for truck and bus loadings.

(3) The loading applies to stack room floors that support nonmobile, double-faced library bookstacks subject to the following limitations:
   a. The nominal bookstack unit height shall not exceed 2300 mm;
   b. The nominal shelf depth shall not exceed 300 mm for each face; and
   c. Parallel rows of double-faced bookstacks shall be separated by aisles not less than 900 mm wide.

(4) In addition to the vertical live loads, the design shall include horizontal swaying forces applied to each row of the seats as follows: 0.4 kN/linear m of seat applied in a direction parallel to each row of seats and 0.15 kN/linear m of seat applied in a direction perpendicular to each row of seats. The parallel and perpendicular horizontal swaying forces need not be applied simultaneously.

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

(6) The concentrated wheel load shall be applied on an area 100 mm by 100 mm, footprint of a jack.

(7) Minimum concentrated load on stair treads on area of 2500 mm^2 is 1.5 kN.

TABLE 4-2: MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS

Occupancy or use	Live Load kN/m^2	Occupancy or use	Live Load kN/m^2
Air-conditioning (machine space)	10*	Laboratories, scientific	5
Amusement park structure	5*	Laundries	7.2*
Attic, Nonresidential		Libraries, corridors	4*
Nonstorage	1.2	Manufacturing, ice	14.5
Storage	4*	Morgue	6.0
Bakery	7.2	Office Buildings	
Exterior	5	Business machine equipment	5*
Interior (fixed seats)	3	Files (see file room)	
Interior (movable seats)	5	Printing Plants	
Boathouse, floors	5*	Composing rooms	5
Boiler room, framed	14.5	Linotype rooms	5
Broadcasting studio	5	Paper storage	**
Catwalks	1.2	Press rooms	7.2*
Ceiling, accessible furred	0.5#	Public rooms	5
Cold Storage		Railroad tracks	++
No overhead system	12+	Ramps	
Overhead system		Driveway (see garages)	
Floor	7.2	Pedestrian (see sidewalks and corridors in Table 4-1)	
Roof	12	Seaplane (see hangars)	
Computer equipment	7.2*	Rest rooms	3
Courtrooms	2.5 – 5.0	Rinks	
Dormitories		Ice skating	12
Nonpartitioned	4	Roller skating	5
Partitioned	2	Storage, hay or grain	14.5*
Elevator machine room	7.2*	Telephone exchange	7.2*
Fan room	7.2*	Theaters	
File room		Dressing rooms	2
Duplicating equipment	7.2*	Grid-iron floor or fly gallery:	
Card	6*	Grating	3
Letter	4*	Well beams, 3.7 kN/m per pair	
Foundries	30*	Header beams, 15 kN/m	
Fuel rooms, framed	20	Pin rail, 3.7 kN/m	
Garages -trucks	℘	Projection room	5
Greenhouses	7.2	Toilet rooms	3
Hangars	7.2 ℘	Transformer rooms	10*
Incinerator charging floor	5	Vaults, in offices	12*
Kitchens, other than domestic	7.2*		

* Use weight of actual equipment or stored material when greater.

\+ Plus 7.2 kN/m^2 for trucks.

℘ Use Ministry of Transportation Lane Load for Highway Bridges. Also subject to not less than 100% maximum axle load.

** Paper storage 2.5 $kN/m^2/m$ of clear story height.

++ As required by Saudi Railway Organization.

\# Accessible ceilings normally are not designed to support persons. The value in this table is intended to account for occasional light storage or suspension of items. If it may be necessary to support the weight of maintenance personnel, this shall be provided for.

SECTION 4.11
INTERIOR WALLS AND PARTITIONS

4.11.1 Interior walls and partitions that exceed 1.8 m 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 $0.25 kN/m^2$.

TABLE 4-3:
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 One-way Slabs Two-way Slabs Members without provisions for continuous shear transfer normal to their span	1

CHAPTER 5
SOIL AND HYDROSTATIC PRESSURE AND FLOOD LOADS

SECTION 5.1
PRESSURE ON BASEMENT WALLS

In the design of basement walls and similar approximately vertical structures below grade, provision shall be made for the lateral pressure of adjacent soil. 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 on the weight of the soil diminished by buoyancy, plus full hydrostatic pressure.

Basement walls shall be designed to resist lateral soil loads. Soil loads specified in Table 5-1 shall be used as the minimum design lateral soil loads unless specified otherwise in a soil investigation report approved by the authority having jurisdiction. The lateral pressure from surcharge loads shall be added to the lateral earth pressure load. The lateral pressure shall be increased if soils with expansion potential are present at the site as determined by a geotechnical investigation.

SECTION 5.2
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 head 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 pressures caused by the expansive soils, or the expansive soil shall be removed or stabilized around and beneath the structure according to SBC 303.

SECTION 5.3
FLOOD LOADS

The provisions of this section apply to buildings and other structures located in areas prone to flooding as defined on a flood hazard map as shown in Figure 5-1.

5.3.1 **Definitions.** The following definitions apply to the provisions of Section 5.3.

Approved. Acceptable to the authority having jurisdiction.

Base Flood. The flood having a 1% chance of being equaled or exceeded in any given year.

Base Flood Elevation (BFE). The elevation of flooding, including wave height, having a 1% 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.

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. The greater of the following two flood events: (1) the Base Flood, affecting those areas identified as Special Flood Hazard Areas or (2) the flood corresponding to the area designated as a Flood Hazard Area or otherwise legally designated.

Design Flood Elevation (DFE). The elevation of the Design Flood, including wave height, relative to the datum.

Flood Hazard Area. The area subject to flooding during the Design Flood.

Flood Hazard Map. The map delineating flood hazard areas adopted by the authority having jurisdiction.

Special Flood Hazard Area (Area of Special Flood Hazard). The land in the floodplain subject to a 1% or greater chance of flooding in any given year. These areas are designated as A-Zones or V-Zones.

5.3.2 Design Requirements.

5.3.2.1 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 5.3.3) and other loads in accordance with the load combinations of Chapter 2.

5.3.2.2 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.

5.3.2.3 Loads on Breakaway Walls. Walls and partitions required by Ref. 5-1, 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 Chapters 6 and 7.
2. The earthquake load specified in Chapters 9 through 16.
3. 0.5 kN/m^2.

The loading at which breakaway walls are intended to collapse shall not exceed 1.0 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; and

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 Chapter 2.

5.3.3 Loads during Flooding.

5.3.3.1 Load Basis. In flood hazard areas, the structural design shall be based on the design flood. The local flood elevation shall be determined by the authority having jurisdiction.

5.3.3.2 Hydrostatic Loads. Hydrostatic loads caused by a depth of water to the level of the design flood elevation 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 0.30 m.

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

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

Regression equations for flood magnitude are presented below for the three hydrologic regions of Saudi Arabia, as shown in Figure 5-2. The average velocity of water V can be obtained by dividing the flood magnitude by the vertical cross sectional area at where the flood is passing.

For Hydrologic Region 1:

$$Q_{50} = 14.40\ A^{0.49472} \qquad \textbf{(Eq. 5-1)}$$

For Hydrologic Region 2:

$$Q_{50} = 0.0594\ A^{0.617}\ E^{-1.22}\ P^{0.933} \qquad \textbf{(Eq. 5-2)}$$

For Hydrologic Region 3:

$$Q_{50} = 0.499\ A^{0.383}\ E^{-5.60} \qquad \textbf{(Eq. 5-3)}$$

where

Q_{50} = Flood magnitude in cubic metres per second for 50 years recurrence interval.

A = Drainage area in square kilometers.

E = Mean basin elevation in thousands of metres above mean sea level.

P = Mean annual precipitation in millimetres as per Table 5-2.

Exception: Where water velocities do not exceed 3.0 m/s, dynamic effects of moving water shall be permitted to be converted into equivalent hydrostatic loads by increasing the design flood elevation for design purposes by an equivalent surcharge depth, d_h, on the headwater side and above the ground level only, equal to:

$$d_h = \frac{aV^2}{2g} \qquad \textbf{(Eq. 5-4)}$$

where

V = average velocity of water in m/s

g = acceleration due to gravity, 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 design flood elevation design depth and the resultant hydrostatic pressures applied to, and uniformly distributed across, the vertical projected area of the building or structure which is perpendicular to the flow. Surfaces parallel to the flow or surfaces wetted by the tailwater shall be subject to the hydro-static pressures for depths to the design flood elevation only.

5.3.3.4 **Wave Loads.** Wave loads shall be determined by one of the following three methods: (1) using the analytical procedures outlined in this section, (2) by more advanced numerical modeling procedures or, (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 run-up striking any portion of the building or structure; wave-induced drag and inertia forces; 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 (0.90 m) high, or higher; in coastal floodplains landward of the V-Zone, waves are less than high (0.90 m).

Nonbreaking and broken wave loads shall be calculated using the procedures described in Sections 5.3.3.2 and 5.3.3.3 to calculate hydrostatic and hydrodynamic loads.

Breaking waves loads shall be calculated using the procedures described in Sections 5.3.3.4.1 through 5.3.3.4.4. Breaking wave heights used in the procedures described in Sections 5.3.3.4.1 through 5.3.3.4.4 shall be calculated for V Zones and Coastal A Zones using Eqs. 5-5 and 5-6.

$$H_b = 0.78\ d_s \qquad \textbf{(Eq. 5-5)}$$

where

H_b = breaking wave height in m

d_s = local stillwater depth in m

The local stillwater depth shall be calculated using Eq. 5-5, unless more advanced procedures or laboratory tests permitted by this section are used.

$$d_s = 0.65(BFE\text{-}G) \qquad \textbf{(Eq. 5-6)}$$

where
BFE = Base Flood Elevation in m
G = Ground elevation in m

5.3.3.4.1 **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 stillwater elevation and shall be calculated by the following:

$$F_D = 0.5\ \gamma_w C_D D H_b^2 \qquad \textbf{(Eq. 5-7)}$$

where

F_D = net wave force, in kN.

γ_w = unit weight of water, in kN/m^3, = 9.80 kN/m^3 for fresh water and 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 m for circular sections, or for a square pile or column, 1.4 times the width of the pile or column in m.

H_b = breaking wave height, in m.

5.3.3.4.2 **Breaking Wave Loads on Vertical Walls.** Maximum pressures and net forces resulting from a normally incident breaking wave (depth-limited in size, with $H_b = 0.78\ d_s$ acting on a rigid vertical wall shall be calculated by the following:

$$P_{max} = C_p\ \gamma_w\ d_s + 1.2\ \gamma_w\ d_s \qquad \textbf{(Eq. 5-8)}$$

and

$$F_t = 1.1 C_p \gamma_w\ d_s^2 + 2.4\ \gamma_w\ d_s^2 \qquad \textbf{(Eq. 5-9)}$$

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 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 kN/m, acting near the still-water elevation

C_p = dynamic pressure coefficient ($1.6 < C_p < 3.5$) (see Table 5-3)

γ_w = unit weight of water, in kN/m^3, = 9.80 kN/m^3 for fresh water and 10.05 kN/m^3 for salt water

d_s = stillwater depth in 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 1.2 d_s above the stillwater level. Thus, the dynamic static and total pressure distributions against the wall are as shown in Figure 5-3.

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 Figure 5-4) and the net force shall be computed by Eq. 5-10 (the maximum combined wave pressure is still computed with Eq. 5-8).

$$F_t = 1.1\ C_p\ \gamma_w\ d_s^2 + 1.9\gamma_w\ d_s^2 \qquad \textbf{(Eq. 5-10)}$$

where

F_t = net breaking wave force per unit length of structure, also referred to as shock, impulse, of wave impact force in kN/m, acting near the stillwater elevation

C_p = dynamic pressure coefficient ($1.6 < C_p < 3.5$) (see Table 5-3)

γ_w = unit weight of water, in kN/m^3, = 9.80 kN/m^3 for fresh water and 10.00 kN/m^3 for salt water

d_s = stillwater depth in m at base of building or other structure where the wave breaks

5.3.3.4.3 **Breaking Wave Loads on Nonvertical Walls.** Breaking wave forces given by Eq. 5-9 and Eq. 5-10 shall be modified in instances where the walls or surfaces upon which the breaking waves act are nonvertical. The horizontal component of breaking wave force shall be given by:

$$F_{nv} = F_t \sin^2\ \alpha \qquad \textbf{(Eq. 5-11)}$$

where

F_{nv} = horizontal component of breaking wave force in kN/m

F_t = net breaking wave force acting on a vertical surface in kN/m

α = vertical angle between nonvertical surface and the horizontal

5.3.3.4.4 **Breaking Wave Loads from Obliquely Incident Waves.** Breaking wave forces given by Eq. 5-9 and Eq. 5-10 shall be modified in instances where waves are obliquely incident. Breaking wave forces from non-normally incident waves shall be given by:

$$F_{oi} = F_t\ \sin^2\ \alpha \qquad \textbf{(Eq. 5-12)}$$

where

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

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

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

5.3.3.4.5 **Impact Loads.** Impact loads are those that result from debris, 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 design flood elevation.

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(Map will be provided when available)

FIGURE 5-1
FLOOD HAZARD MAP

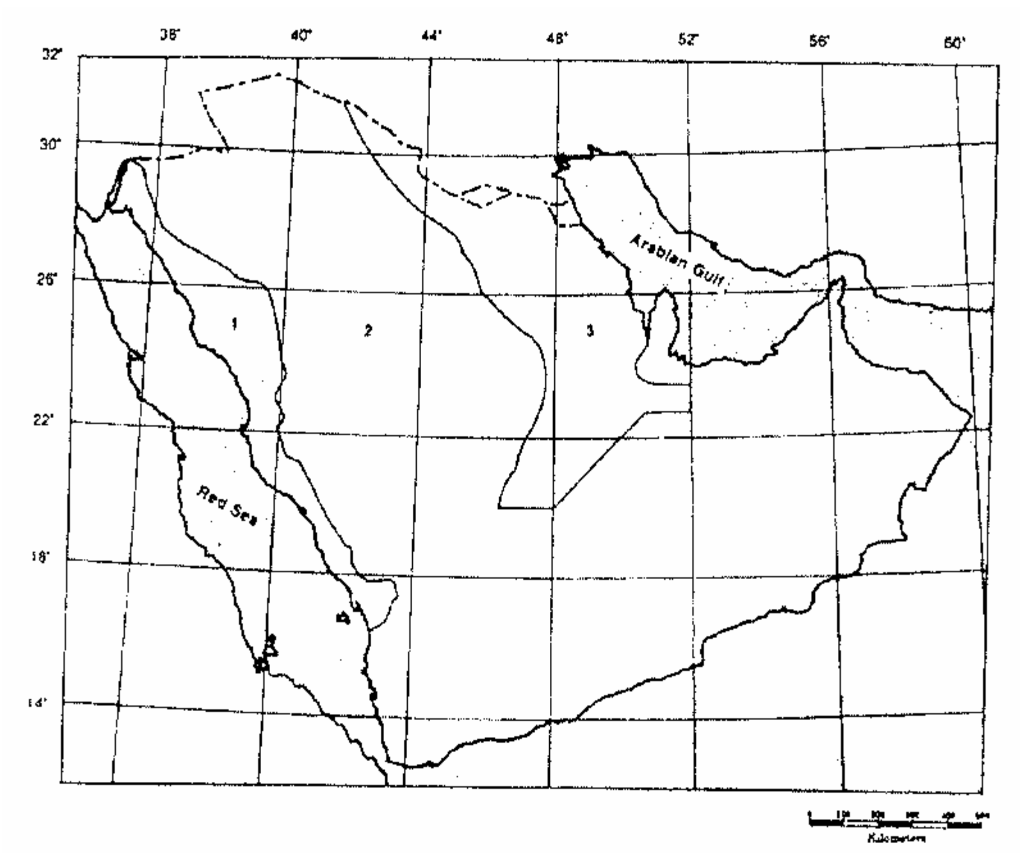


FIGURE 5-2
HYDROLOGIC REGIONS IN SAUDI ARABIA

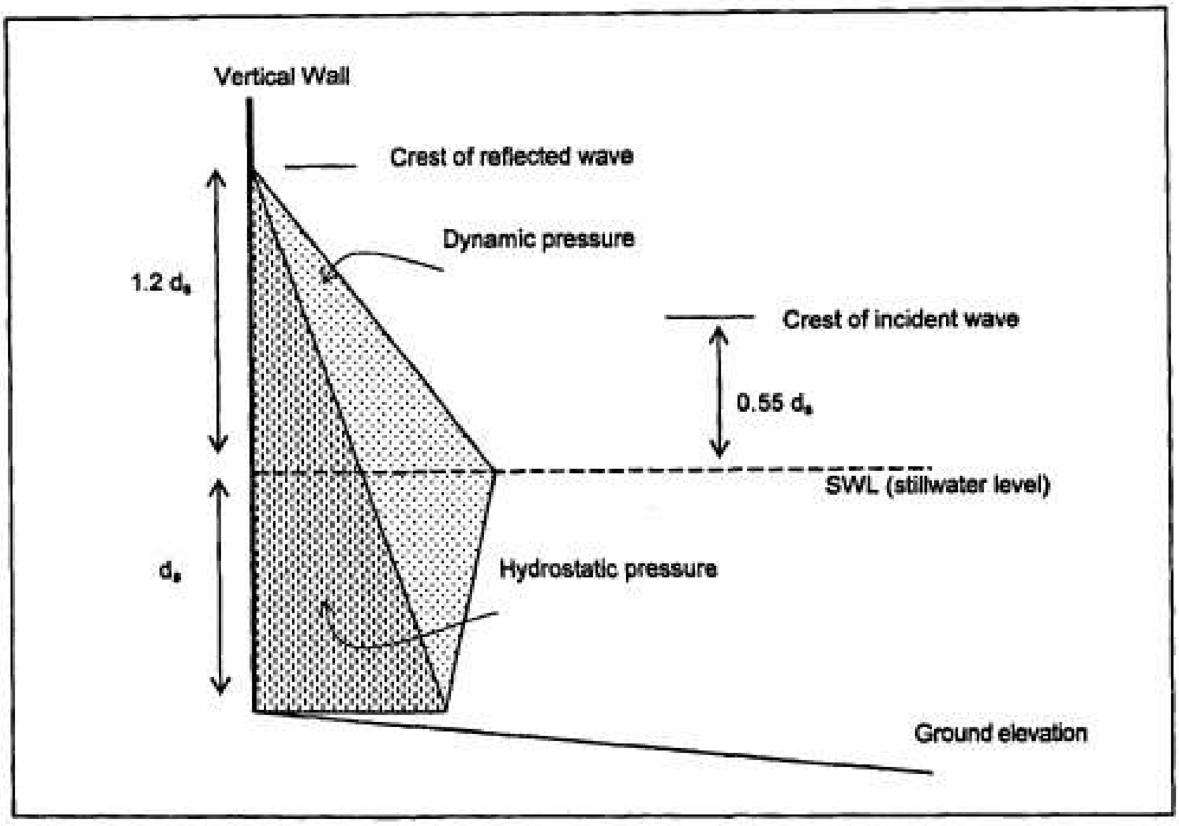


FIGURE 5-3
NORMALLY INCIDENT BREAKING WAVE PRESSURES AGAINST A VERTICAL WALL (SPACE BEHIND VERTICAL WALL IS DRY)

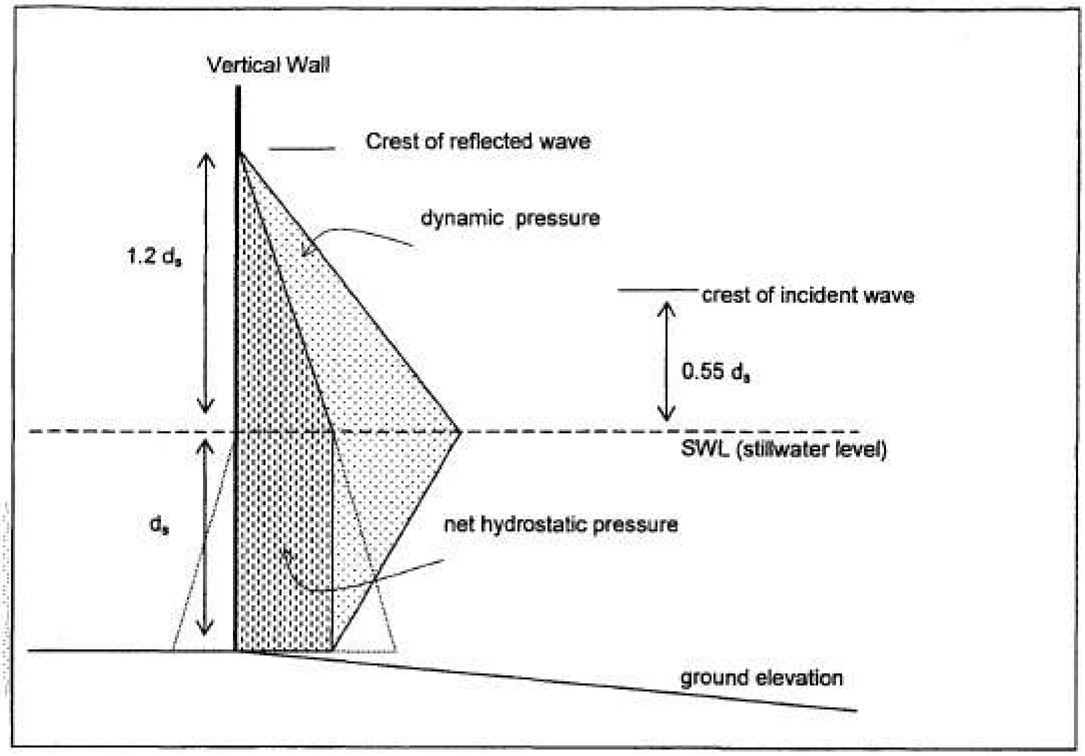


FIGURE 5-4
NORMALLY INCIDENT BREAKING WAVE PRESSURES AGAINST A VERTICAL WALL (STILLWATER LEVEL EQUAL ON BOTH SIDES OF WALL)

TABLE 5-1:
DESIGN LATERAL SOIL LOAD

Description of Backfill Material	Unified Soil Classification	Design Lateral Soil Load (Note A) $kN/m^2/m$ of depth
Well-graded, clean gravels; gravel-sand mixes	GW	5.50 Note C
Poorly graded clean gravels; gravel-sand mixes	GP	5.50 Note C
Silty gravels, poorly graded gravel-sand mixes	GM	5.50 Note C
Clayey gravels, poorly graded gravel-and-clay mixes	GC	7.0 Note C
Well-graded, clean sands; gravelly-sand mixes	SW	5.50 Note C
Poorly graded clean sands; sand-gravel mixes	SP	5.50 Note C
Silty sands, poorly graded sand-silt mixes	SM	7.0 Note C
Sand-silt clay mix with plastic fines	SM-SC	13.5 Note D
Clayey sands, poorly graded sand-clay mixes	SC	13.5 Note D
Inorganic silts and clayey silts	ML	13.5 Note D
Mixture of inorganic silt and clay	ML-CL	13.5 Note D
Inorganic clays of low to medium plasticity	CL	16
Organic silts and silt-clays, low plasticity	OL	Note B
Inorganic clayey silts, elastic silts	MH	Note B
Inorganic clays of high plasticity	CH	Note B
Organic clays and silty clays	OH	Note B

Note 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.

Note B. Unsuitable as backfill material.

Note C. For relatively rigid walls, as when braced by floors, the design lateral soil load shall be increased for sand and gravel type soils to 9.5 kN/m^2 per metre of depth. Basement walls extending not more than 2.5 m below grade and supporting light floor systems are not considered as being relatively rigid walls.

Note D. For relatively rigid walls, as when braced by floors, the design lateral load shall be increased for silt and clay type soils to 16 kN/m^2 per metre of depth. Basement walls extending not more than 2.5 m below grade and supporting light floor systems are not considered as being relatively rigid walls.

TABLE 5-2:
RAINFALL PRECIPITATION

City (Region)	Mean Annual Precipitation 'P' (mm)
Abha	330
Abqaiq	81
Abu Sa'fah	**
Al-Baha	154
Al-Jauf	59
Ar'Ar' (Badana)	73
Berri	90
Dhahran	83
Duba (as Al-Wajih)	28
Hail	130
Rarad	40
Hawta	112
Hofuf	92
Jeddah	54
Jizan	120
Ju'aymah	89
Khamis Mushayt	220
Khurais	81
Medina	**
Marjan	**
Najran	64
Qasim	135
Qaisumah	121
Qatif	87
Rabigh	60
Ras Tanura	89
Riyadh	116
Safaniya	98
Shaybah	24
Shedgum	81
Tanajib	98
Tabuk	48
Turaif	78
Udhailiyah	**
Uthmaniyah	**
Yanbu	36

** Data Not Available

TABLE 5-3:
VALUE OF DYNAMIC PRESSURE COEFFICIENT, C_p

Building Category	C_p
I	1.6
II	2.8
III	3.2
IV	3.5

CHAPTER 6
WIND LOADS

SECTION 6.1
GENERAL

6.1.1 **Scope.** Buildings and other structures, including the main wind force-resisting system and all components and cladding thereof, shall be designed and constructed to resist wind loads as specified herein.

6.1.2 **Allowed Procedures.** The design wind loads for buildings and other structures, including the main wind force-resisting system and component and cladding elements thereof, shall be determined using one of the following procedures: (1) Method 1 – Simplified Procedure as specified in Section 7.1 for buildings meeting the requirements specified therein; (2) Method 2 – Analytical Procedure as specified in Section 7.2 for buildings meeting the requirements specified therein; (3) Method 3 – Wind Tunnel Procedure according to Section 7.3.

6.1.3 **Wind Pressures Acting on Opposite Faces of Each Building Surface.** In the calculation of design wind loads for the main wind force-resisting system and for components and cladding for buildings, the algebraic sum of the pressures acting on opposite faces of each building surface shall be taken into account.

6.1.4 **Minimum Design Wind Loading.** The design wind load, determined by any one of the procedures specified in Section 6.1.2, shall be not less than specified in this Section.

6.1.4.1 Main Wind Force-Resisting System

The wind load to be used in the design of the main wind force-resisting system for an enclosed or partially enclosed building or other structure shall not be less than 0.5 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 0.5 kN/m^2 multiplied by the area A_f, as defined in section 6.3.

6.1.4.2 Components and Cladding.

The design wind pressure for components and cladding of buildings shall not be less than a net pressure of 0.5 kN/m^2 acting in either direction normal to the surface.

SECTION 6.2
DEFINITIONS

The following definitions apply only to the provisions of Chapters 6 and 7:

Approved. Acceptable to the authority having jurisdiction.

Basic Wind Speed, V. 3-second gust speed at 10 m above the ground in Exposure C (see Section 6.4.2) as determined in accordance with Section 6.4.1.

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 and other structures that have a fundamental natural frequency less than 1 Hz.

Building, Low-Rise. Enclosed or partially enclosed buildings that comply with the following conditions:

1. Mean roof height h is less than or equal to 18 m; and
2. Mean roof height h does not exceed least horizontal dimension.

Building, Open. A building having each wall at least 80% open. This condition is expressed for each wall by the equation $A_o > 0.8A_g$ where

A_o = total area of openings in a wall that receives positive external pressure, in m^2

A_g = the gross area of that wall in which A_o is identified, in m^2

Building, Partially Enclosed. A building that complies with 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%, and
2. The total area of openings in a wall that receives positive external pressure exceeds 0.4 m^2 or 1% 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%.

These conditions are expressed by the following equations:

1. $A_o > 1.10\ A_{oi}$
2. $A_o > 0.4\ m^2$ or $> 0.01\ A_g$, whichever is smaller, and $A_{oi} / A_{gi} \leq 0.20$

where

A_o, A_g are as defined for Open Building

A_{oi} = the sum of the areas of openings in the building envelope (walls and roof) not including A_o, in m^2.

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

Building or Other Structure, Regular Shaped. A building or other structure having no unusual geometrical irregularity in spatial form.

Building or Other Structures, Rigid. A building or other structure whose fundamental frequency is greater than or equal to 1 Hz.

Building, Simple Diaphragm. An enclosed or partially enclosed building in which wind loads are transmitted through floor and roof diaphragms to the vertical main wind force-resisting system.

Components and Cladding. Elements of the building envelope that do not qualify as part of the main wind force-resisting system.

Design Force, F. Equivalent static force to be used in the determination of wind loads for open buildings and other structures.

Design Pressure, p. Equivalent static pressure to be used in the determination of wind loads for buildings.

Effective Wind Area. The area used to determine GC_p. For component and cladding elements, the effective wind area in Figures 7.2-7 through 7.2-13 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 6.4.3, a cliff or steep slope generally separating two levels or gently sloping areas (see Figure 6.4-2).

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

Hill. With respect to topographic effects in Section 6.4.3, a land surface characterized by strong relief in any horizontal direction (see Figure 6.4-2).

Importance Factor, I. A factor that accounts for the degree of hazard to human life and damage to property.

Main Wind Force-Resisting System. 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 degrees, 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 6.4.3, an elongated crest of a hill characterized by strong relief in two directions (see Figure 6.4-2).

SECTION 6.3
SYMBOLS AND NOTATIONS

The following symbols and notations apply only to the provisions of Chapters 6 and 7:

A	=	effective wind area, in m^2;
A_f	=	area of open buildings and other structures either normal to the wind direction or projected on a plane normal to the wind direction, in m^2;
A_g	=	the gross area of that wall in which A_o is identified, in m^2;
A_{gi}	=	the sum of the gross surface areas of the building envelope (walls and roof) not including A_g, in m^2;
A_o	=	total area of openings in a wall that receives positive external pressure, in m^2;
A_{oi}	=	the sum of the areas of openings in the building envelope (walls and roof) not including A_o, in m^2;
A_{og}	=	total area of openings in the building envelope in m^2;
a	=	width of pressure coefficient zone, in m;
B	=	horizontal dimension of building measured normal to wind direction, in m;
$\bar{b}$	=	mean hourly wind speed factor in Eq. 7.2-11 from Table 7.2-1;
$\hat{b}$	=	3-second gust speed factor from Table 7.2-1;
C_f	=	force coefficient to be used in determination of wind loads for other structures;
C_p	=	external pressure coefficient to be used in determination of wind loads for buildings;
c	=	turbulence intensity factor in Eq. 7.2-2 from Table 7.2-1;
D	=	diameter of a circular structure or member, in m;
D'	=	depth of protruding elements such as ribs and spoilers, in m;
G	=	gust effect factor;
G_f	=	gust effect factor for main wind force-resisting systems of flexible buildings and other structures;
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 main wind force-resisting system 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 in Eqs. 7.2-1 and 7.2-5;
g_R	=	peak factor for resonant response in Eq. 7.2-5;
g_v	=	peak factor for wind response in Eqs. 7.2-1 and 7.2-5;
H	=	height of hill or escarpment in Figure 6.4-2, in m;
h	=	mean roof height of a building or height of other structure, except that eave height shall be used for roof angle θ of less than or equal to 10 degrees, in m;
I	=	importance factor;
$I_{\bar{z}}$	=	intensity of turbulence from Eq. 7.2-2;
K_1, K_2, K_3	=	multipliers in Figure 6.4-2 to obtain K_{zt};
K_d	=	wind directionality factor in Table 6.4-1;
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;
L	=	horizontal dimension of a building measured parallel to the wind direction, in m;
L_h	=	distance upwind of crest of hill or escarpment in Figure 6.4-2 to where the difference in ground elevation is half the height of hill or escarpment, in m;
$L_{\bar{z}}$	=	integral length scale of turbulence, in m;
ℓ	=	integral length scale factor from Table 7.2-1, m;
M	=	larger dimension of sign, in m;
N	=	smaller dimension of sign, in m;
N_I	=	reduced frequency from Eq. 7.2-9;
n_1	=	building natural frequency, Hz;
p	=	design pressure to be used in determination of wind loads for buildings, in kN/m^2;
p_L	=	wind pressure acting on leeward face in Figure 7.2-5;
p_{net10}	=	net design wind pressure for exposure B at h = 10 m and I = 1.0 from Figure 7.1-2;
p_p	=	combined net pressure on a parapet from Eq. 7.2-17;
p_{s10}	=	simplified design wind pressure for exposure B at h = 10 m and I = 1.0 from Figure 7.1-1;
p_w	=	wind pressure acting on windward face in Figure 7.2-5;
Q	=	background response factor from Eq. 7.2-3;
q	=	velocity pressure, in kN/m^2;
q_h	=	velocity pressure evaluated at height z = h, kN/m^2;
q_i	=	velocity pressure for internal pressure determination;
q_p	=	velocity pressure at top of parapet;
q_z	=	velocity pressure evaluated at height z above ground, kN/m^2;
R	=	resonant response from Eq. 7.2-7;
R_B, R_h, R_L	=	values from Eq. 7.2-10a and 7.2-10b;
R_i	=	reduction factor from Eq. 7.2-13;
R_n	=	value from Eq. 7.2-8;
r	=	rise-to-span ratio for arched roofs;
V	=	basic wind speed in km/h. The basic wind speed corresponds to a 3-second gust speed at 10 m above ground in Exposure Category C;
V_i	=	unpartitioned internal volume m^3;
$\overline{V}_{\bar{z}}$	=	mean hourly wind speed at height $\bar{z}$, m/s;
W	=	width of building in Figures 7.2-8, and 7.2-10 A and B and width of span in Figures 7.2-9 and 7.2-11, in m;
X	=	distance to center of pressure from windward edge in Figure 7.2-14, in m;
x	=	distance upwind or downwind of crest in Figure 6.4-2, in m;
z	=	height above ground level, in m;
$\bar{z}$	=	equivalent height of structure, in m;
z_g	=	nominal height of the atmospheric boundary layer used in this standard. Values appear in Table 7.2-1;
z_{min}	=	exposure constant from Table 7.2-1;
α	=	3-sec gust speed power law exponent from Table 7.2-1
$\hat{\alpha}$	=	reciprocal of α from Table 7.2-1;
$\bar{\alpha}$	=	mean hourly wind speed power law exponent in Eq. 7.2-11 from Table 7.2-1;

β	=	damping ratio, percent critical for buildings or other structures;
ε	=	ratio of solid area to gross area for open sign, face of a trussed tower, or lattice structure;
λ	=	adjustment factor for building height and exposure from Figures 7.1-1 and 7.1-2;
$\bar{\varepsilon}$	=	integral length scale power law exponent in Eq. 7.2-4 from Table 7.2-1;
η	=	value used in Eq. 7.2-10a and 7.2-10b (see Section 7.2.7.2);
θ	=	angle of plane of roof from horizontal, in degrees;
υ	=	height-to-width ratio for solid sign.

SECTION 6.4
BASIC WIND PARAMETERS

6.4.1 **Basic Wind Speed.** The basic wind speed, V, used in the determination of design wind loads on buildings and other structures shall be taken as shown in Figure 6.4-1 except as provided in Sections 6.4.1.1 and 6.4.1.2. The wind shall be assumed to come from any horizontal direction.

6.4.1.1 **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 Figure 6.4-1. Mountainous terrain, gorges, and special regions shall be examined for unusual wind conditions. The authority having jurisdiction shall adjust basic wind speed 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 6.4.1.2.

6.4.1.2 **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 Figure 6.4-1 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.

6.4.1.3 **Wind Directionality Factor.** The wind directionality factor, K_d, shall be determined from Table 6.4-1. This factor shall only be applied when used in conjunction with load combinations specified in Sections 2.3 and 2.4.

6.4.2 **Exposure.** For each wind direction considered, an exposure category that adequately reflects the characteristics of ground roughness and surface irregularities shall be determined for the site at which the building or structure is to be constructed. Account shall be taken of variations in ground surface roughness that arises from natural topography and vegetation as well as constructed features.

6.4.2.1 **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 degrees either side of the selected wind direction. The exposures in these two sectors shall be determined in accordance with Sections 6.4.2.2 and 6.4.2.3 and the exposure resulting in the highest wind loads shall be used to represent the winds from that direction.

TABLE 6.4-1:
WIND DIRECTIONALITY FACTOR, K_d

Structure Type	Directionality Factor K_d*
Buildings Main Wind Force Resisting System Components and Cladding	 0.85 0.85
Arched Roofs	0.85
Chimneys, Tanks, and Similar Structures Square Hexagonal Round	 0.90 0.95 0.95
Solid Signs	0.85
Open Signs and Lattice Framework	0.85
Trussed Towers Triangular, square, rectangular All other cross sections	 0.85 0.95

* Directionality Factor K_d has been calibrated with combinations of loads specified in Chapter 2. This factor shall only be applied when used in conjunction with load combinations specified in Sections 2.3 and 2.4.

6.4.2.2 **Surface Roughness Categories.** A ground surface roughness within each 45-degree sector shall be determined for a distance upwind of the site as defined in Section 6.4.2.3 from the categories defined below, for the purpose of assigning an exposure category as defined in Section 6.4.2.3.

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 10 m.

Surface Roughness D: Flat, unobstructed areas and water surfaces.

6.4.2.3 **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 800 m or 10 times the height of the building, whichever is greater.

Exception: For buildings whose mean roof height is less than or equal to 10 m, the upwind distance may be reduced to 450 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 at least 1500 m or 10 times the building height, whichever is greater. Exposure D shall extend inland from the shoreline for a distance of 200 m or 10 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.

6.4.3 Topographic Effects.

6.4.3.1 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, 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 (100 H) or 3 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 3 km radius in any quadrant by a factor of two or more;
3. The structure is located as shown in Figure 6.4-2 in the upper half of a hill or ridge or near the crest of an escarpment;
4. $H/L_h \geq 0.2$; and
5. H is greater than or equal to 4.5m for Exposures C and D and 18 m for Exposure B.

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

$$K_{zt} = (1 + K_1K_2K_3)^2 \qquad \textbf{(Eq. 6.4-1)}$$

where K_1, K_2, and K_3 are given in Figure 6.4-2.

SECTION 6.5
IMPORTANCE FACTOR

An importance factor, I, for the building or other structure shall be determined from Table 6.5-1 based on building and structure categories defined in Table 1.6-1.

SECTION 6.6
ENCLOSURE CLASSIFICATIONS

6.6.1 General. For the purpose of determining internal pressure coefficients, all buildings shall be classified as enclosed, partially enclosed, or open as defined in Section 6.2.

6.6.2 Openings.

A determination shall be made of the amount of openings in the building envelope in order to determine the enclosure classification as defined in Section 6.6.1.

6.6.3 Multiple Classifications.

If a building by definition complies with both the "open" and "partially enclosed" definitions, it shall be classified as an "open" building. A building that does not comply with either the "open" or "partially enclosed" definitions shall be classified as an "enclosed" building.

TABLE 6.5-1: IMPORTANCE FACTOR, I

Category[1]	Regions with V = 136-160 km/h	Regions with V > 160 km/h
I	0.87	0.77
II	1.00	1.00
III	1.15	1.15
IV	1.15	1.15

1. The building and structure classification categories are defined in Table 1.6-1.

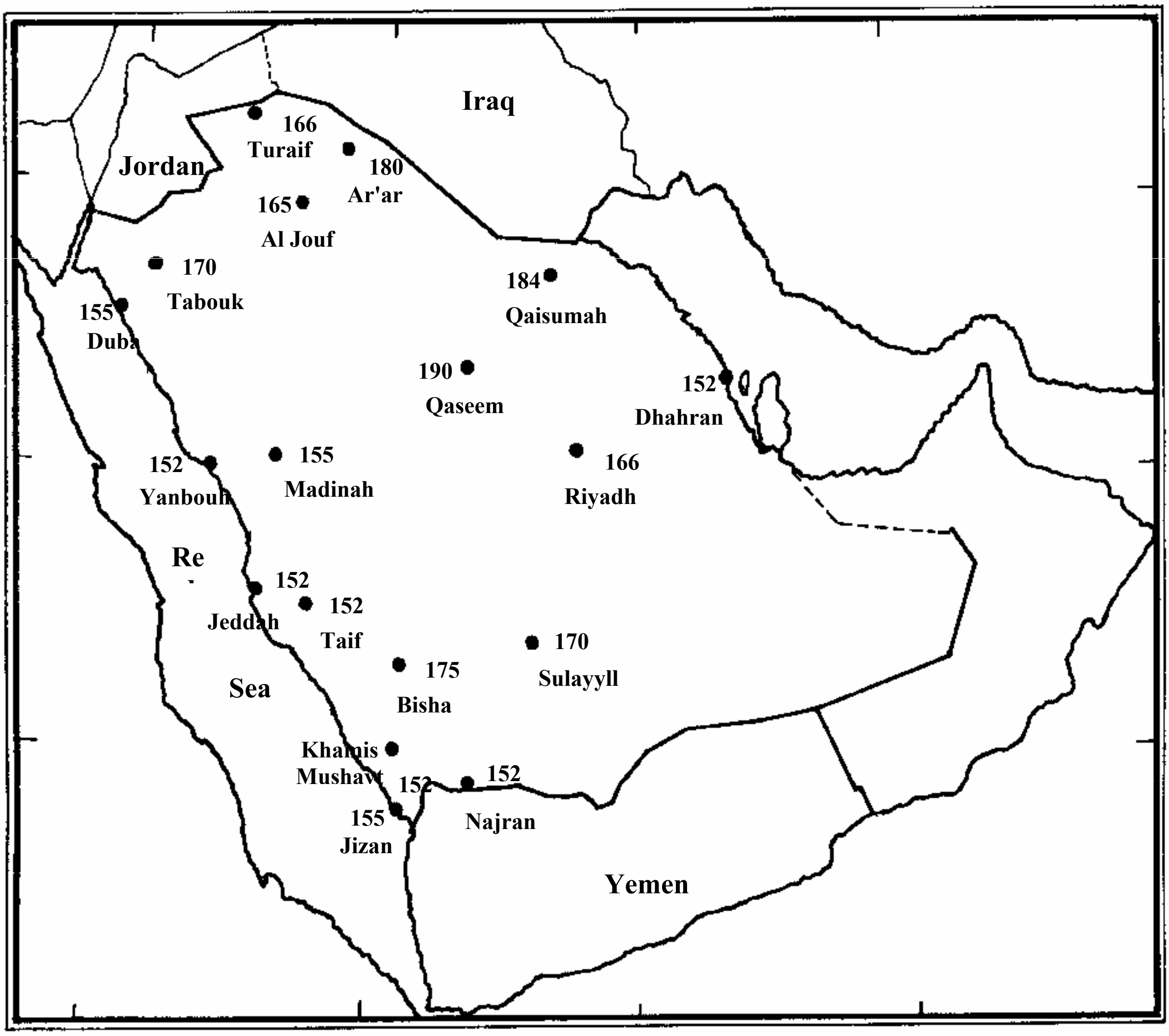
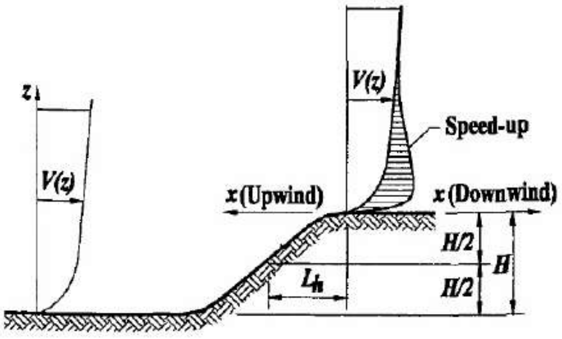
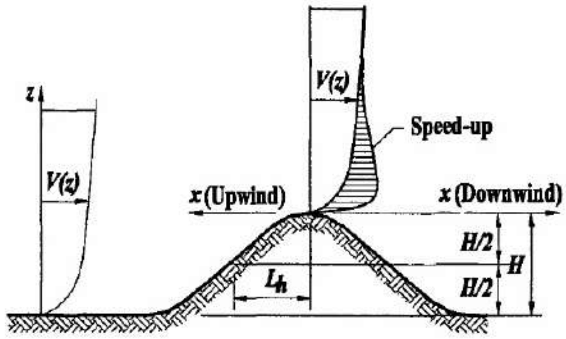


FIGURE 6.4-1
BASIC 3-SECOND GUST WIND SPEED IN km/h FOR SELECTED CITIES OF SAUDI ARABIA. ADOPTED FROM SAUDI ARAMCO DATA SAES A-112.





ESCARPMENT **2-D RIDGE OR 3-D AXISYMMETRICAL HILL**

Topographic Multipliers for Exposure C										
H/L_h	K_1 Multiplier			x/L_h	K_2 Multiplier		z/L_h	K_3 Multiplier		
	2-D Ridge	2-D Escarp.	3-D Axisym. Hill		2-D Escarp.	All Other Cases		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 2H for 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 metres.
   - L_h: Distance upwind of crest to where the difference in ground elevation is half the height of hill or escarpment, in metres.
   - 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 metres.
   - z: Height above local ground level, in metres.
   - μ : Horizontal attenuation factor.
   - γ : Height attenuation factor.

FIGURE 6.4-2
TOPOGRAPHIC FACTOR, K_{zt}

Equations:

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

K_1 is determined from table below

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

$$K_3 = e^{-\gamma z / L_h}$$

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

FIGURE 6.4-2 – cont'd
TOPOGRAPHIC FACTOR, K_{zt}

CHAPTER 7
DESIGN WIND LOAD PROCEDURES

SECTION 7.1
METHOD 1 - SIMPLIFIED PROCEDURE

7.1.1 **Scope.** A building whose design wind loads are determined in accordance with this Section shall meet all the conditions of Section 7.1.1.1 or 7.1.1.2. If a building qualifies only under Section 7.1.1.2 for design of its components and cladding, then its main wind force-resisting system shall be designed by Method 2.

7.1.1.1 **Main Wind Force-Resisting Systems.**

For the design of main wind force-resisting systems, the building must meet all of the following conditions:

1. The building is a simple diaphragm building as defined in Section 6.2,

2. The building is a low-rise building as defined in Section 6.2,

3. The building is enclosed as defined in Section 6.2,

4. The building is a regular shaped building or structure as defined in Section 6.2, and has an approximately symmetrical cross section in each direction with either a flat roof, or a gable or hip roof with $\theta \leq 45$ degrees,

5. The building is not classified as a flexible building as defined in Section 6.2,

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 structure has no expansion joints or separations,

8. The building is not subject to the topographic effects of Section 6.4.3 (i.e., $K_{zt} = 1.0$).

7.1.1.2 **Components and Cladding.** For the design of components and cladding the building must meet all the following conditions:

1. The mean roof height $h < 18$ m,

2. The building is enclosed as defined in Section 6.2,

3. The building is a regular shaped building or structure as defined in Section 6.2,

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 is not subject to the topographic effects of Section 6.4.3 (i.e., $K_{zt} = 1.0$),

6. The building has either a flat roof, or a gable roof with $\theta \leq 45$ degrees, or a hip roof with $\theta \leq 27$ degrees.

7.1.2 Design Procedure.

1. The basic wind speed V shall be determined in accordance with Section 6.4.1. The wind shall be assumed to come from any horizontal direction.
2. An exposure category shall be determined in accordance with Section 6.4.2.
3. An importance factor I shall be determined in accordance with Section 6.5.
4. A height and exposure adjustment coefficient, λ, shall be determined from Figure 7.1-1.

7.1.2.1 Main Wind Force-Resisting System. Simplified design wind pressures, p_s, for the main wind force-resisting systems 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 Figure 7.1-1. 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

$$p_s = \lambda\ I\ p_{s10} \qquad \textbf{(Eq. 7.1-1)}$$

Where

λ = adjustment factor for building height and exposure from Figure 7.1-1.

I = importance factor as defined in Section 6.5.

p_{s10} = simplified design wind pressure for exposure B, at h = 10 m, and for I = 1.0, from Figure 7.1-1.

7.1.2.1.1 Minimum Pressures. The load effects of the design wind pressures from Section 7.1.2.1 shall not be less than the minimum load case from Section 6.1.4.1 assuming the pressures, p_s, for Zones A, B, C, and D all equal to +0.5 kN/m^2, while assuming Zones E, F, G, and H all equal to 0 kN/m^2.

7.1.2.1.2 Pressures in Concrete Buildings. In reinforced concrete buildings the additional edge pressure in Zones A, B, E, and F may be neglected and a uniform pressure as computed for Zones C, D, G, and H can be taken across the entire width.

7.1.2.2 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 Figure 7.1-2. p_{net} shall be determined by the following equation:

$$p_{net} = \lambda\ I\ p_{net10} \qquad \textbf{(Eq. 7.1-2)}$$

Where

λ = adjustment factor for building height and exposure from Figure 7.1-2.

I = importance factor as defined in Section 6.5.

p_{net10} = net design wind pressure for exposure B, at h = 10 m and for I = 1.0, from Figure 7.1-2.

7.1.2.2.1 Minimum Pressures. The positive design wind pressures, p_{net}, from Section 7.1.2.2 shall not be less than +0.5 kN/m^2, and the negative design wind pressures, p_{net}, from 7.1.2.2 shall not be less than -0.5 kN/m^2.

7.1.3 Air-Permeable Cladding. Design wind loads determined from Figure 7.1-2 shall be used for all air-permeable cladding unless approved test data or recognized literature demonstrate lower loads for the type of air-permeable cladding being considered.

SECTION 7.2
METHOD 2 – ANALYTICAL PROCEDURE

7.2.1 **Scope.** A building or other structure whose design wind loads are determined in accordance with this Section shall meet all of the following conditions:

1. The building or other structure is a regular shaped building or structure as defined in Section 6.2, and
2. The building or other structure does not have response characteristics making it subject to across-wind loading, vortex shedding, instability due to galloping or flutter; or does not have a site location for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.

7.2.2 **Limitations.** The provisions of Section 7.2 take into consideration the load magnification effect caused by gusts in resonance with along-wind vibrations of flexible buildings or other structures. Buildings or other structures not meeting the requirements of Section 7.2.1, or having unusual shapes or response characteristics shall be designed using recognized literature documenting such wind load effects or shall use the wind tunnel procedure according to Section 7.3.

7.2.2.1 **Shielding.** There shall be no reductions in velocity pressure due to apparent shielding afforded by buildings and other structures or terrain features.

7.2.2.2 **Air-Permeable Cladding.** Design wind loads determined from Section 7.2 shall be used for air-permeable cladding.

7.2.3 **Design Procedure.**

1. The basic wind speed V and wind directionality factor K_d shall be determined in accordance with Section 6.4.1.
2. An exposure category or exposure categories shall be determined for each wind direction in accordance with Section 6.4.2.
3. A topographic factor K_{zt} shall be determined in accordance with Section 6.4.3.
4. An importance factor I shall be determined in accordance with Section 6.5.
5. An enclosure classification shall be determined in accordance with Section 6.6.
6. Velocity pressure exposure coefficient K_z or K_h, as applicable, shall be determined for each wind direction in accordance with Section 7.2.6.
7. A gust effect factor G or G_f, as applicable, shall be determined in accordance with Section 7.2.7.
8. Velocity pressure q_z or q_h, as applicable, shall be determined in accordance with Section 7.2.8.

9. Internal pressure coefficient GC_{pi} shall be determined in accordance with Section 7.2.9.1.

10. External pressure coefficients C_p or GC_{pf}, or force coefficients C_f, as applicable, shall be determined in accordance with Section 7.2.9.2 or 7.2.9.3, respectively.

11. Design wind load p or F shall be determined in accordance with Sections 7.2.10 and 7.2.11, as applicable.

7.2.4 Exposure Category for Main Wind Force-Resisting Systems.

7.2.4.1 Buildings and Other Structures. For each wind direction considered, wind loads for the design of the main wind force-resisting system (MWFRS) determined from Figure 7.2-2 shall be based on the exposure categories defined in Section 6.4.2.3.

7.2.4.2 Low-Rise Buildings. Wind loads for the design of the main wind force-resisting systems for low-rise buildings shall be determined using a velocity pressure q_h based on the exposure resulting in the highest wind loads for any wind direction at the site when external pressure coefficients GC_{pf} given in Figure 7.2-6 are used.

7.2.5 Exposure Category for Components and Cladding.

7.2.5.1 Buildings with Mean Roof Height h Less Than or Equal to 18 m. Components and cladding for buildings with a mean roof height h of 18 m or less shall be designed using a velocity pressure q_h based on the exposure resulting in the highest wind loads for any wind direction at the site.

7.2.5.2 Buildings with Mean Roof Height h Greater Than 18 m and Other Structures. Components and cladding for buildings with a mean roof height h in excess of 18 m and for other structures shall be designed using the exposure resulting in the highest wind loads for any wind direction at the site.

7.2.6 Velocity Pressure Exposure Coefficient. Based on the exposure category determined in Section 6.4.2.3, a velocity pressure exposure coefficient K_z or K_h, as applicable, shall be determined from Table 7.2-2.

7.2.7 Gust Effect Factor.

7.2.7.1 Rigid Structures. For rigid structures as defined in Section 6.2, the gust effect factor shall be taken as 0.85 or calculated by the formula:

$$G = 0.925\left(\frac{\left(1+1.7g_Q I_{\bar{z}} Q\right)}{1+1.7g_v I_{\bar{z}}}\right) \qquad \textbf{(Eq. 7.2-1)}$$

$$I_{\bar{z}} = c\left(10/\bar{z}\right)^{1/6} \qquad \textbf{(Eq. 7.2-2)}$$

where $I_{\bar{z}}$ = the intensity of turbulence at height $\bar{z}$ where $\bar{z}$ = the equivalent height of the structure defined as 0.6 h but not less than z_{min} for all building heights h. z_{min} and c are listed for each exposure in Table 7.2-1; g_Q and g_v shall be taken as 3.4. The background response Q is given by

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} \qquad \textbf{(Eq. 7.2-3)}$$

where *B, h* are defined in Section 6.3; and $L_{\bar{z}}$ = the integral length scale of turbulence at the equivalent height given by

$$L_{\bar{z}} = \ell(\bar{z}/10)^{\bar{\varepsilon}} \qquad \textbf{(Eq. 7.2-4)}$$

in which ℓ and $\bar{\varepsilon}$ are constants listed in Table 7.2-1.

7.2.7.2 Flexible or Dynamically Sensitive Structures. For flexible or dynamically sensitive structures as defined in Section 6.2, the gust effect factor shall be calculated by:

$$G_f = 0.925\left(\frac{\left(1+1.7\,I_{\bar{z}}\,\sqrt{g_Q^2 Q^2 + g_R^2 R^2}\right)}{1+1.7 g_v I_{\bar{z}}}\right) \qquad \textbf{(Eq. 7.2-5)}$$

g_Q and g_v shall be taken as 3.4 and g_R is given by

$$g_R = \sqrt{2\,ln\,(3600\,n_1)} + \frac{0.577}{\sqrt{2\,ln\,(3600\,n_1)}} \qquad \textbf{(Eq. 7.2-6)}$$

R, the resonant response factor, is given by

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B\,(0.53 + 0.47\,R_\ell)} \qquad \textbf{(Eq. 7.2-7)}$$

$$R_n = \frac{7.47\,N_1}{(1+10.3\,N_1)^{5/3}} \qquad \textbf{(Eq. 7.2-8)}$$

$$N_1 = \frac{n_1\,L_{\bar{z}}}{\bar{V}_{\bar{z}}} \qquad \textbf{(Eq. 7.2-9)}$$

$$R_\ell = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1-e^{-2\eta}\right) \quad for \;\; \eta > 0 \qquad \textbf{(Eq. 7.2-10a)}$$

$$R_\ell = 1 \quad for \;\; \eta = 0 \qquad \textbf{(Eq. 7.2-10b)}$$

where the subscript ℓ in Eq. 7.2-10a and Eq. 7.2-10b shall be taken as h, B, and L respectively.

n_1 = building natural frequency;

R_ℓ = R_h setting $\eta = 4.6\, n_1\, h/\bar{V}_{\bar{z}}$;

R_ℓ = R_B setting $\eta = 4.6\, n_1\, B/\bar{V}_{\bar{z}}$;

R_ℓ = R_L setting $\eta = 15.4\, n_1 L/\bar{V}_{\bar{z}}$;

β = damping ratio, percent of critical h, B, L are defined in Section 6.3; and

$\bar{V}_{\bar{z}}$ = mean hourly wind speed (m/sec) at height $\bar{z}$ determined from Eq.7.2-11.

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{10}\right)^{\bar{\alpha}} V\left(\frac{1}{3.6}\right) \qquad \textbf{(Eq. 7.2-11)}$$

where $\bar{b}$ and $\bar{\alpha}$ are constants listed in Table 7.2-1 and V is the basic wind speed in km/h.

7.2.7.3 **Limitations** Where combined gust effect factors and pressure coefficients (GC_p, GC_{pi}, GC_{pf}) are given in figures and tables, the gust effect factor shall not be determined separately.

7.2.8 Velocity Pressure.

Velocity pressure, q_z evaluated at height z shall be calculated by the following equation:

$$q_z = 0.0473 \times 10^{-3}\, K_z K_{zt} K_d V^2 I \quad (\text{kN/m}^2);\ \text{V in km/h} \qquad \textbf{(Eq. 7.2-12)}$$

where
K_d is the wind directionality factor defined in Section 6.4.1, K_z is the velocity pressure exposure coefficient defined in Section 7.2.6 and K_{zt} is the topographic factor defined in Section 6.4.3, and q_h is the velocity pressure calculated using Eq. 7.2-12 at mean roof height h.

7.2.9 Pressure and Force Coefficients.

7.2.9.1 **Internal Pressure Coefficient.** Internal pressure coefficients, GC_{pi} shall be determined from Figure 7.2-1 based on building enclosure classifications determined from Section 6.6.

7.2.9.1.1 **Reduction Factor for Large Volume Buildings.** For a partially enclosed building containing a single, unpartitioned large volume, the internal pressure coefficient, GC_{pi}, shall be multiplied by the following reduction factor, R_i:

$R_i = 1.0$
or

$$R_i = 0.5\left(1 + \frac{1}{\sqrt{1 + \frac{V_i}{6950 A_{og}}}}\right) \le 1.0 \qquad \textbf{(Eq. 7.2-13)}$$

where
A_{og} = total area of openings in the building envelope (walls and roof, in m^2)
V_i = unpartitioned internal volume, in m^3

7.2.9.2 External Pressure Coefficients.

7.2.9.2.1 Main Wind Force-Resisting Systems. External pressure coefficients for main wind force resisting systems C_p, are given in Figures 7.2-2, 7.2-3, and 7.2-4. Combined gust effect factor and external pressure coefficients, GC_{pf}, are given in Figure 7.2-6 for low-rise buildings. The pressure coefficient values and gust effect factor in Figure 7.2-6 shall not be separated.

7.2.9.2.2 Components and Cladding. Combined gust effect factor and external pressure coefficients for components and cladding GC_p, are given in Figures 7.2-7 through 7.2-13. The pressure coefficient values and gust effect factor shall not be separated.

7.2.9.3 Force Coefficients.

Force coefficients C_f are given in Figures 7.2-14 through 7.2-18.

7.2.9.4 Roof Overhangs.

7.2.9.4.1 Main Wind Force-Resisting System. Roof overhangs shall be designed for a positive pressure on the bottom surface of windward roof overhangs corresponding to C_p = 0.8 in combination with the pressures determined from using Figures 7.2-2 and 7.2-6.

7.2.9.4.2 Components and Cladding. For all buildings, roof overhangs shall be designed for pressures determined from pressure coefficients given in Figure 7.2-7 B, C, and D.

7.2.9.5 Parapets.

7.2.9.5.1 Main Wind Force-Resisting System. The pressure coefficients for the effect of parapets on the MWFRS loads are given in Section 7.2.10.2.4

7.2.9.5.2 Components and Cladding. The pressure coefficients for the design of parapet component and cladding elements are taken from the wall and roof pressure coefficients as specified in Section 7.2.10.4.4.

7.2.10 Design Wind Loads on Enclosed and Partially Enclosed Buildings.

7.2.10.1 General.

7.2.10.1.1 Sign Convention. Positive pressure acts toward the surface and negative pressure acts away from the surface.

7.2.10.1.2 Critical Load Condition. Values of external and internal pressures shall be combined algebraically to determine the most critical load.

7.2.10.1.3 Tributary Areas Greater than 65 m^2. Component and cladding elements with tributary areas greater than 65 m^2 shall be permitted to be designed using the provisions for main wind force resisting systems.

7.2.10.2 Main Wind Force-Resisting Systems.

7.2.10.2.1 Rigid Buildings of All Height. Design wind pressures for the main wind force-resisting system of buildings of all heights shall be determined by the following equation:

$$p = qGC_p - q_i(GC_{pi}) \quad (\text{kN/m}^2) \qquad \textbf{(Eq. 7.2-14)}$$

where

q = q_z for windward walls evaluated at height z above the ground;

q = q_h for leeward walls, side walls, and roofs, evaluated at height h;

q_i = q_h for windward walls, side walls, leeward walls, and roofs of enclosed buildings and for negative internal pressure evaluation in partially enclosed buildings;

q_i = q_z for positive internal pressure evaluation in partially enclosed buildings where height z is defined as the level of the highest opening in the building that could affect the positive internal pressure. For positive internal pressure evaluation, q_i may conservatively be evaluated at height h ($q_i = q_h$);

G = gust effect factor from Section 7.2.7;

C_p = external pressure coefficient from Figure 7.2-2 or 7.2-4;

(GC_{pi}) = internal pressure coefficient from Figure 7.2-1

q and q_i = shall be evaluated using exposure defined in Section 6.4.2.3. Pressure shall be applied simultaneously on windward and leeward walls and on roof surfaces as defined in Figures 7.2-2 and 7.2-5.

7.2.10.2.2 Low-Rise Building. Alternatively, design wind pressures for the main wind force-resisting system of low-rise buildings shall be determined by the following equation:

$$p = q_h\,[(GC_{pf}) - (GCp_i)] \quad (\text{kN/m}^2) \qquad \textbf{(Eq. 7.2-15)}$$

where

q_h = velocity pressure evaluated at mean roof height h using exposure defined in Section 6.4.2.3;

(GC_{pf}) = external pressure coefficient from Figure 7.2-6;

(GC_{pi}) = internal pressure coefficient from Figure 7.2-1.

7.2.10.2.3 Flexible Buildings. Design wind pressures for the main wind force-resisting system of flexible buildings shall be determined from the following equation:

$$p = qG_f\,C_p - q_i\,(GC_{pi}) \quad (\text{kN/m}^2) \qquad \textbf{(Eq. 7.2-16)}$$

where q, q_i, C_p and (GC_{pi}) are as defined in Section 7.2.10.2.1 and, G_f = gust effect factor defined in Section 7.2.7.2

7.2.10.2.4 Parapets. The design wind pressure for the effect of parapets on main wind force-resisting systems of rigid, low-rise or flexible buildings with flat, gable, or hip roofs shall be determined by the following equation:

$$p_p = q_pGC_{pn} \quad (\text{kN/m}^2) \qquad \textbf{(Eq. 7.2-17)}$$

where

p_p = combined net pressure on the parapet due to the combination of the net pressures from the front and back parapet surfaces. Plus (and minus) signs signify net pressure acting toward (and away from) the front (exterior) side of the parapet.

q_p = velocity pressure evaluated at the top of the parapet.

GC_{pn} = combined net pressure coefficient

= +1.8 for windward parapet

= -1.1 for leeward parapet

7.2.10.3 Design Wind Load Cases. The main wind force-resisting system of buildings of all heights, whose wind loads have been determined under the provisions of Sections 7.2.10.2.1 and 7.2.10.2.3, shall be designed for the wind load cases as defined in Figure 7.2-5. The eccentricity e for rigid structures shall be measured from the geometric center of the building face and shall be considered for each principal axis (e_x, e_y). The eccentricity e for flexible structures shall be determined from the following equation and shall be considered for each principal axis (e_x, e_y):

$$e = \frac{e_Q + 1.7\, I_{\bar{z}} \sqrt{(g_Q Q e_Q)^2 + (g_R R e_R)^2}}{1.7\, I_{\bar{z}} \sqrt{(g_Q Q)^2 + (g_R R)^2}} \qquad \textbf{(Eq. 7.2-18)}$$

where

e_Q = eccentricity e as determined for rigid structures in Figure 7.2-5

e_R = distance between the elastic shear center and center of mass of each floor

$I_{\bar{z}}$, g_Q, Q, g_R, R shall be as defined in Section 7.2.7

The sign of the eccentricity e shall be plus or minus, whichever causes the more severe load effect.

Exception: One-story buildings with h less than or equal to 10 m, buildings two stories or less framed with light-framed construction and buildings two stories or less designed with flexible diaphragms need only be designed for Load Case 1 and Load Case 3 in Figure 7.2-5.

7.2.10.4 Components and Cladding.

7.2.10.4.1 Low-Rise Buildings and Buildings with h < 18 m. Design wind pressures on component and cladding elements of low-rise buildings and buildings with h ≤ 18 m shall be determined from the following equation:

$$p = q_h[(GC_p) - (GC_{pi})] \quad (kN/m^2) \qquad \textbf{(Eq. 7.2-19)}$$

where

q_h = velocity pressure evaluated at mean roof height h using exposure defined in Section 6.4.2.3;

(GC_p) = external pressure coefficients given in Figures 7.2-7 through 7.2-12; and

(GC_{pi}) = internal pressure coefficient given in Figure 7.2-1.

7.2.10.4.2 Buildings with h > 18 m. Design wind pressures on components and cladding for all buildings with h > 18 m shall be determined from the following equation:

$$p = q(GC_p) - q_i(GC_{pi}) \ \ (kN/m^2) \qquad \textbf{(Eq. 7.2-20)}$$

where

q = q_z for windward walls calculated at height z above the ground;

q = q_h for leeward walls, side walls, and roofs, evaluated at height h;

q_i = q_h for windward walls, side walls, leeward walls, and roofs of enclosed buildings and for negative internal pressure evaluation in partially enclosed buildings; and

q_i = q_z for positive internal pressure evaluation in partially enclosed buildings where height z is defined as the level of the highest opening in the building that could affect the positive internal pressure. For positive internal pressure evaluation, q_i may conservatively be evaluated at height h ($q_i = q_h$);

(GCp) = external pressure coefficient from Figure 7.2-13;

(GCpi) = internal pressure coefficient from Figure 7.2-1.

q and q_i = shall be evaluated using exposure defined in Section 6.4.2.3.

7.2.10.4.3 Alternative Design Wind Pressure for Components and Cladding in Buildings with 18 m < h < 27 m.

Alternative to the requirements of Section 7.2.10.4.2, the design of components and claddings for building with a mean roof height greater than 18 m and less than 27 m, values from Figures 7.2-7 through 7.2-12 shall be used only if the height to width ratio is one or less (except as permitted by Note 6 of Figure 7.2-13) and Eq. 7.2-19 is used.

7.2.10.4.4 Parapets. The design wind pressure on the components and cladding elements of parapets shall be designed by the following equation:

$$p = q_p(GC_p - GC_{pi}) \qquad \textbf{(Eq. 7.2-21)}$$

where

q_p = velocity pressure evaluated at the top of the parapet;

GC_p = external pressure coefficient from Figures 7.2-7 through 7.2-13; and

GC_{pi} = internal pressure coefficient from Figure 7.2-1, based on the porosity of the parapet envelope

Two load cases shall be considered. Load Case A shall consist of applying the applicable positive wall pressure from Figure 7.2-7A or 7.2-13 to the front surface of the parapet while applying the applicable negative edge or corner zone roof pressure from Figure 7.2-7B through 7.2-13 to the back surface. Load Case B shall consist of applying the applicable positive wall pressure from Figure 7.2-7A or 7.2-13 to the back of the parapet surface, and applying the applicable negative wall pressure from Figure 7.2-7A or 7.2-13 to the front surface. Edge and corner zones shall be arranged as shown in Figures 7.2-7 through 7.2-13. GC_p shall be determined for appropriate roof angle and effective wind area from Figures 7.2-7 through 7.2-13. If internal pressure is present, both load cases should be evaluated under positive and negative internal pressure.

7.2.11 **Design Wind Loads on Open Buildings and Other Structures.** The design wind force for open buildings and other structures shall be determined by the following formula:

$$F = q_z G C_f A_f \quad \text{(kN)} \qquad \textbf{(Eq. 7.2-22)}$$

where

q_z = velocity pressure evaluated at height z of the centroid of area A_f using exposure defined in Section 6.4.2.3;

G = gust effect factor from Section 7.2.7;

C_f = net force coefficients from Figures 7.2-14 through 7.2-18; and

A_f = projected area normal to the wind except where C_f is specified for the actual surface area, m^2.

SECTION 7.3
METHOD 3 – WIND-TUNNEL PROCEDURE

7.3.1 **Scope.** Wind-tunnel tests shall be used where required by Section 7.2.2. Wind-tunnel testing shall be permitted in lieu of Methods 1 and 2 for any building or structure.

7.3.2 **Test Conditions.** Wind-tunnel tests, or similar tests employing fluids other than air, used for the determination of design wind loads for any building or other structure, shall be conducted in accordance with this section. Tests for the determination of mean and fluctuating forces and pressures shall meet all of the following conditions:

1. the natural atmospheric boundary layer has been modeled to account for the variation of wind speed with height;

2. the relevant macro (integral) length and micro length scales of the longitudinal component of atmospheric turbulence are modeled to approximately the same scale as that used to model the building or structure;

3. the modeled building or other structure and surrounding structures and topography are geometrically similar to their full-scale counterparts, except that, for low-rise buildings meeting the requirements of Section 7.2.1, tests shall be permitted for the modeled building in a single exposure site as defined in Section 6.4.2.1;

4. the projected area of the modeled building or other structure and surroundings is less than 8% of the test section cross-sectional area unless correction is made for blockage;

5. the longitudinal pressure gradient in the wind-tunnel test section is accounted for;

6. Reynolds number effects on pressures and forces are minimized; and

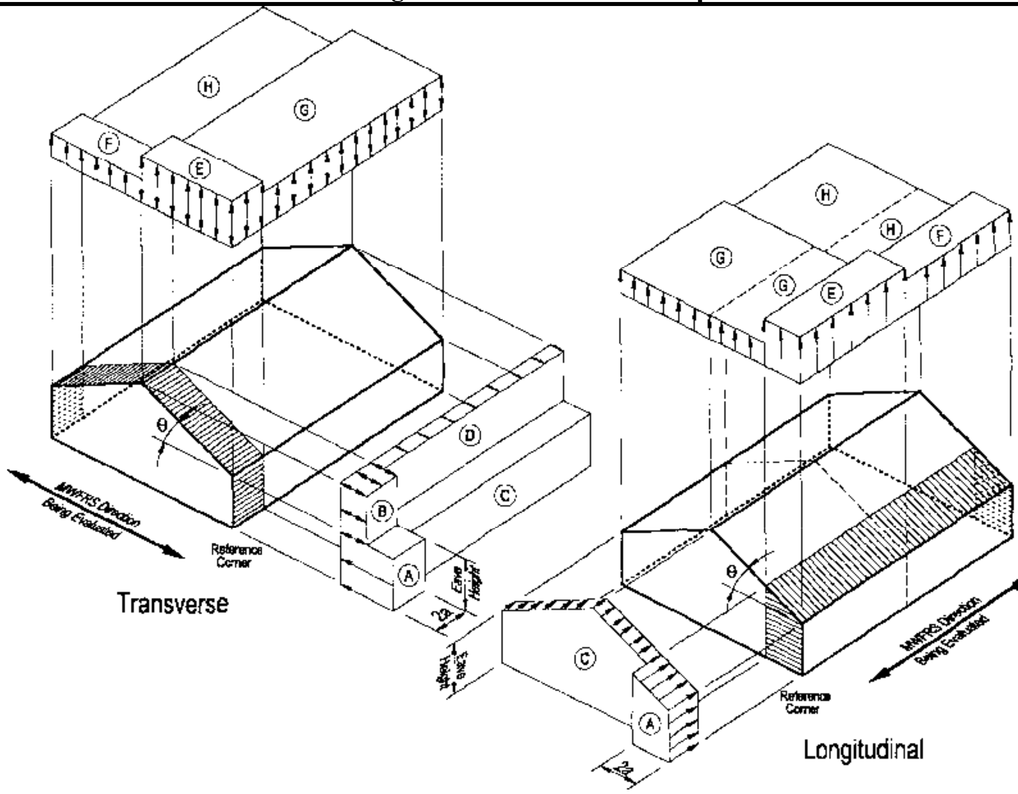
7. response characteristics of the wind-tunnel instrumentation are consistent with the required measurements.

7.3.3 **Dynamic Response.** Tests for the purpose of determining the dynamic response of a building or other structure shall be in accordance with Section 7.3.2. The structural model and associated analysis shall account for mass distribution, stiffness, and damping.

7.3.4 **Limitations.**

7.3.4.1 **Limitations on Wind Speeds.** Variation of basic wind speeds with direction shall not be permitted unless the analysis for wind speeds conforms to the requirements of Section 6.4.1.2.

Main Wind Force Resisting System - Method 1		$h \leq 18m$
Figure 7.1-1	**Design Wind Pressures**	**Walls & Roofs**
Enclosed Buildings		



Notes:

1. Pressures shown are applied to the horizontal and vertical projections, for exposure B, at h= 10 m, for I=1.0. Adjust to other exposures and heights with adjustment factor λ .
2. The load patterns shown shall be applied to each corner of the building in turn as the reference corner. (See Figure 7.2-6)
3. For the design of the longitudinal MWFRS use $\theta = 0°$, 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°< $\theta \leq$ 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:
   Horizontal pressure zones - Sum of the windward and leeward net (sum *of* internal and external) pressures on vertical projection of:
   - A - End zone of wall — C - Interior zone of wall
   - B - End zone of roof — D - Interior zone of roof

   Vertical pressure zones - Net (sum of internal and external) pressures on horizontal projection of:
   - E - End zone of windward roof — G - Interior zone of windward roof
   - F - End zone of leeward roof — H - Interior zone of leeward roof
9. 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.
10. 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 0.9 m.
    - h: Mean roof height, in metres, except that eave height shall be used for roof angles <10°.
    - θ : Angle of plane of roof from horizontal, in degrees.

Main Wind Force Resisting System - Method 1		$h \leq 18m$
Figure 7.1-1 (Cont'd)	Design Wind Pressures	**Walls & Roofs**
Enclosed Buildings		

Simplified Design Wind Pressure , p_{s10} (kN/m^2) (Exposure B at h = 10 m with I = 1.0)

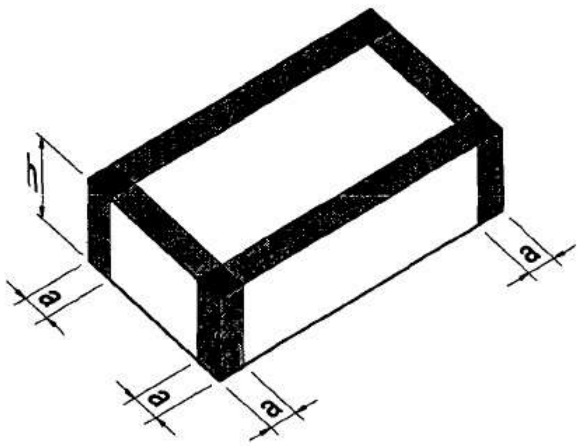
Basic Wind Speed (km/h)	Roof Angle (deg)	Load Case	Zones									
			Horizontal Pressures				Vertical Pressures				Overhangs	
			A	B	C	D	E	F	G	H	E_{OH}	G_{OH}
135	0 to 5°	1	0.55	-0.28	0.36	-0.17	-0.66	-0.37	-0.46	-0.29	-0.92	-0.72
	10°	1	0.62	-0.26	0.41	-0.15	-0.66	-0.40	-0.46	-0.31	-0.92	-0.72
	15°	1	0.69	-0.23	0.46	-0.13	-0.66	-0.43	-0.46	-0.33	-0.92	-0.72
	20°	1	0.76	-0.20	0.51	-0.11	-0.66	-0.46	-0.46	-0.35	-0.92	-0.72
	25°	1	0.69	0.11	0.50	0.11	-0.31	-0.42	-0.22	-0.34	-0.57	-0.48
		2	—	—	—	—	-0.11	-0.23	-0.03	-0.14	—	—
	30° to 45°	1	0.62	0.42	0.49	0.34	0.05	-0.37	0.01	-0.32	-0.22	-0.25
		2	0.62	0.42	0.49	0.34	0.24	-0.19	0.21	-0.13	-0.22	-0.25
145	0 to 5°	1	0.61	-0.32	0.41	-0.19	-0.74	-0.42	-0.51	-0.33	-1.03	-0.81
	10°	1	0.69	-0.29	0.46	-0.17	-0.74	-0.45	-0.51	-0.34	-1.03	-0.81
	15°	1	0.77	-0.26	0.51	-0.14	-0.74	-0.48	-0.51	-0.37	-1.03	-0.81
	20°	1	0.85	-0.23	0.57	-0.12	-0.74	-0.51	-0.51	-0.39	-1.03	-0.81
	25°	1	0.77	0.12	0.56	0.13	-0.34	-0.47	-0.25	-0.37	-0.64	-0.55
		2	—	—	—	—	-0.13	-0.25	-0.03	-0.16	—	—
	30° to 45°	1	0.69	0.47	0.55	0.38	0.05	-0.42	0.02	-0.36	-0.24	-0.28
		2	0.69	0.47	0.55	0.38	0.27	-0.21	0.23	-0.15	-0.24	-0.28
160	0 to 5°	1	0.76	-0.39	0.50	-0.23	-0.91	-0.52	-0.64	-0.40	-1.28	-1.00
	10°	1	0.86	-0.35	0.57	-0.21	-0.91	-0.56	-0.64	-0.43	-1.28	-1.00
	15°	1	0.95	-0.32	0.64	-0.18	-0.91	-0.59	-0.64	-0.45	-1.28	-1.00
	20°	1	1.05	-0.28	0.70	-0.15	-0.91	-0.64	-0.64	-0.48	-1.28	-1.00
	25°	1	0.95	0.15	0.69	0.16	-0.42	-0.57	-0.31	-0.46	-0.79	-0.67
		2	—	—	—	—	-0.16	-0.32	-0.04	-0.20	—	—
	30° to 45°	1	0.85	0.58	0.68	0.47	0.07	-0.52	0.02	-0.45	-0.30	-0.34
		2	0.85	0.58	0.68	0.47	0.33	-0.25	0.28	-0.18	-0.30	-0.34
175	0 to 5°	1	0.92	-0.48	0.61	-0.28	-1.11	-0.63	-0.77	-0.48	-1.55	-1.21
	10°	1	1.03	-0.43	0.69	-0.25	-1.11	-0.68	-0.77	-0.52	-1.55	-1.21
	15°	1	1.15	-0.38	0.77	-0.22	-1.11	-0.72	-0.77	-0.55	-1.55	-1.21
	20°	1	1.27	-0.34	0.85	-0.19	-1.11	-0.77	-0.77	-0.58	-1.55	-1.21
	25°	1	1.15	0.19	0.83	0.19	-0.51	-0.70	-0.37	-0.56	-0.95	-0.81
		2	—	—	—	—	-0.20	-0.38	-0.05	-0.24	—	—
	30° to 45°	1	1.03	0.71	0.82	0.56	0.08	-0.63	0.03	-0.54	-0.36	-0.42
		2	1.03	0.71	0.82	0.56	0.40	-0.31	0.34	-0.22	-0.36	-0.42
190	0 to 5°	1	1.09	-0.57	0.72	-0.34	-1.31	-0.75	-0.91	-0.58	-1.84	-1.44
	10°	1	1.24	-0.51	0.82	-0.30	-1.31	-0.80	-0.91	-0.62	-1.84	-1.44
	15°	1	1.37	-0.45	0.91	-0.26	-1.31	-0.86	-0.91	-0.66	-1.84	-1.44
	20°	1	1.51	-0.40	1.01	-0.22	-1.31	-0.91	-0.91	-0.69	-1.84	-1.44
	25°	1	1.37	0.22	0.99	0.23	-0.61	-0.83	-0.44	-0.67	-1.13	-0.97
		2	—	—	—	—	-0.23	-0.45	-0.06	-0.29	—	—
	30° to 45°	1	1.23	0.84	0.98	0.67	0.10	-0.75	0.03	-0.64	-0.43	-0.49
		2	1.23	0.84	0.98	0.67	0.47	-0.37	0.41	-0.26	-0.43	-0.49
205	0 to 5°	1	1.28	-0.67	0.85	-0.39	-1.54	-0.88	-1.07	-0.68	-2.16	-1.69
	10°	1	1.45	-0.60	0.96	-0.35	-1.54	-0.94	-1.07	-0.72	-2.16	-1.69
	15°	1	1.61	-0.54	1.07	-0.31	-1.54	-1.01	-1.07	-0.77	-2.16	-1.69
	20°	1	1.78	-0.47	1.18	-0.26	-1.54	-1.07	-1.07	-0.81	-2.16	-1.69
	25°	1	1.61	0.26	1.16	0.26	-0.71	-0.98	-0.52	-0.79	-1.33	-1.13
		2	—	—	—	—	-0.27	-0.53	-0.07	-0.34	—	—
	30° to 45°	1	1.44		1.15	0.79	0.11	-0.88	0.04	-0.75	-0.51	-0.58
		2	1.44	0.99	1.15	0.79	0.56	-0.43	0.48	-0.31	-0.51	-0.58

Main Wind Force Resisting System - Method 1		$h \le 18m$
Figure 7.1-1 (Cont'd)	Design Wind Pressures	Walls & Roofs
Enclosed Buildings		

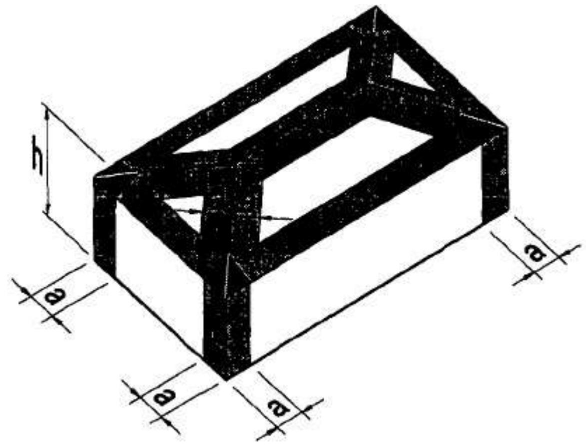
ADJUSTMENT FACTOR
FOR BUILDING HEIGHT AND EXPOSURE, λ

Mean roof height (m)	Exposure		
	B	C	D
4.5	1	1.21	1.47
6.0	1	1.29	1.55
7.5	1	1.35	1.61
9.0	1	1.4	1.66
10.5	1.05	1.45	1.7
12.0	1.09	1.49	1.74
13.5	1.12	1.53	1.78
15.0	1.16	1.56	1.81
16.5	1.19	1.59	1.84
18.0	1.22	1.62	1.87

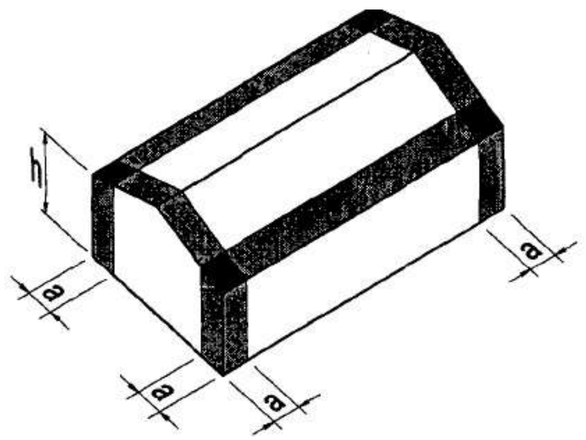
Components and Cladding - Method 1		$h \le 18m$
Figure 7.1-2	**Design Wind Pressures**	**Walls & Roofs**
Enclosed Buildings		



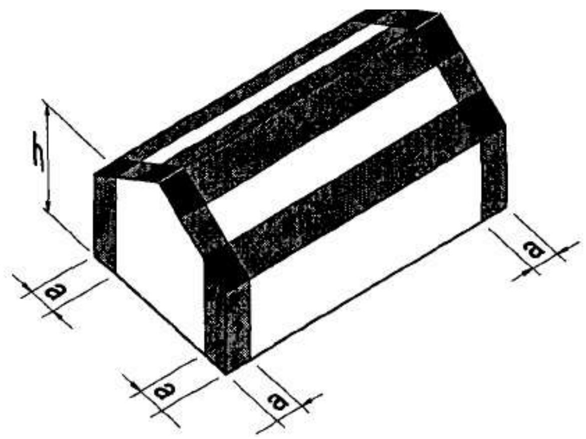
Flat Roof



Hip Roof ($7^o < \theta \le 27^o$)



Gable Roof ($\theta \le 7^o$)



Gable Roof ($7^o < \theta \le 45^o$)

☐ **Interior Zones** — **Roofs-Zone 1 / Walls-Zone 4**

■ **End Zones** — **Roofs –Zone 2/ Walls – Zone 5**

■ **Corner Zones** — **Roofs - Zone 3**

Notes:

1. Pressures shown are applied normal to the surface, for exposure B, at h = 10 m, for I=1.0. Adjust to other exposures and heights with adjustment factor λ
2. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
3. For hip roofs with $\theta \le 25°$, 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 0.9 m.
   - h: Mean roof height, in metres, 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		$h \leq 18m$
Figure 7.1-2 (Cont'd)	Design Wind Pressures	**Walls & Roofs**
Enclosed Buildings		

Net Design Wind Pressure , p_{net10} (kN/m^2) (Exposure B at h = 10 m with I = 1.0)

	Zone	Effective Wind area (m^2)	Basic Wind Speed V (km/h)											
			135		145		160		175		190		205	
Roof 0 to 7 degrees	1	1	0.25	-0.62	0.28	-0.70	0.35	-0.86	0.43	-1.04	0.50	-1.24	0.59	-1.46
	1	2	0.24	-0.61	0.27	-0.68	0.33	-0.84	0.40	-1.02	0.47	-1.21	0.56	-1.42
	1	5	0.22	-0.58	0.24	-0.66	0.30	-0.81	0.36	-0.98	0.43	-1.17	0.51	-1.37
	1	10	0.20	-0.57	0.23	-0.64	0.28	-0.79	0.34	-0.95	0.40	-1.13	0.47	-1.33
	2	1	0.25	-1.04	0.28	-1.17	0.35	-1.45	0.43	-1.75	0.50	-2.08	0.59	-2.44
	2	2	0.24	-0.93	0.27	-1.04	0.33	-1.29	0.40	-1.56	0.47	-1.86	0.56	-2.18
	2	5	0.22	-0.79	0.24	-0.88	0.30	-1.09	0.36	-1.32	0.43	-1.57	0.51	-1.84
	2	10	0.20	-0.68	0.23	-0.76	0.28	-0.93	0.34	-1.13	0.40	-1.35	0.47	-1.58
	3	1	0.25	-1.57	0.28	-1.76	0.35	-2.17	0.43	-2.63	0.50	-3.13	0.59	-3.68
	3	2	0.24	-1.30	0.27	-1.46	0.33	-1.80	0.40	-2.18	0.47	-2.60	0.56	-3.05
	3	5	0.22	-0.94	0.24	-1.06	0.30	-1.31	0.36	-1.58	0.43	-1.88	0.51	-2.21
	3	10	0.20	-0.68	0.23	-0.76	0.28	-0.93	0.34	-1.13	0.40	-1.35	0.47	-1.58
Roof > 7 to 27 degrees	1	1	0.36	-0.57	0.40	-0.64	0.50	-0.79	0.60	-0.95	0.71	-1.13	0.84	-1.33
	1	2	0.33	-0.56	0.37	-0.62	0.45	-0.77	0.55	-0.93	0.65	-1.10	0.77	-1.29
	1	5	0.29	-0.53	0.32	-0.60	0.39	-0.74	0.48	-0.89	0.57	-1.06	0.67	-1.24
	1	10	0.25	-0.52	0.28	-0.58	0.35	-0.71	0.43	-0.87	0.50	-1.03	0.59	-1.21
	2	1	0.36	-0.99	0.40	-1.11	0.50	-1.37	0.60	-1.66	0.71	-1.98	0.84	-2.32
	2	2	0.33	-0.91	0.37	-1.02	0.45	-1.26	0.55	-1.53	0.65	-1.82	0.77	-2.14
	2	5	0.29	-0.81	0.32	-0.90	0.39	-1.12	0.48	-1.35	0.57	-1.61	0.67	-1.89
	2	10	0.25	-0.73	0.28	-0.81	0.35	-1.01	0.43	-1.22	0.50	-1.45	0.59	-1.70
	3	1	0.36	-1.47	0.40	-1.64	0.50	-2.03	0.60	-2.46	0.71	-2.92	0.84	-3.43
	3	2	0.33	-1.37	0.37	-1.54	0.45	-1.90	0.55	-2.29	0.65	-2.73	0.77	-3.21
	3	5	0.29	-1.24	0.32	-1.39	0.39	-1.72	0.48	-2.08	0.57	-2.48	0.67	-2.91
	3	10	0.25	-1.15	0.28	-1.29	0.35	-1.59	0.43	-1.92	0.50	-2.29	0.59	-2.69
Roof > 27 to 45 degrees	1	1	0.57	-0.62	0.64	-0.70	0.79	-0.86	0.95	-1.04	1.13	-1.24	1.33	-1.46
	1	2	0.56	-0.59	0.62	-0.66	0.77	-0.82	0.93	-0.99	1.10	-1.18	1.29	-1.38
	1	5	0.53	-0.55	0.60	-0.61	0.74	-0.76	0.89	-0.92	1.06	-1.09	1.24	-1.28
	1	10	0.52	-0.52	0.58	-0.58	0.71	-0.71	0.87	-0.87	1.03	-1.03	1.21	-1.21
	2	1	0.57	-0.73	0.64	-0.81	0.79	-1.01	0.95	-1.22	1.13	-1.45	1.33	-1.70
	2	2	0.56	-0.69	0.62	-0.78	0.77	-0.96	0.93	-1.16	1.10	-1.39	1.29	-1.63
	2	5	0.53	-0.66	0.60	-0.73	0.74	-0.90	0.89	-1.10	1.06	-1.30	1.24	-1.53
	2	10	0.52	-0.62	0.58	-0.70	0.71	-0.86	0.87	-1.04	1.03	-1.24	1.21	-1.46
	3	1	0.57	-0.73	0.64	-0.81	0.79	-1.01	0.95	-1.22	1.13	-1.45	1.33	-1.70
	3	2	0.56	-0.69	0.62	-0.78	0.77	-0.96	0.93	-1.16	1.10	-1.39	1.29	-1.63
	3	5	0.53	-0.66	0.60	-0.73	0.74	-0.90	0.89	-1.10	1.06	-1.30	1.24	-1.53
	3	10	0.52	-0.62	0.58	-0.70	0.71	-0.86	0.87	-1.04	1.03	-1.24	1.21	-1.46
Wall	4	1	0.62	-0.68	0.70	-0.76	0.86	-0.93	1.04	-1.13	1.24	-1.35	1.46	-1.58
	4	2	0.59	-0.65	0.67	-0.72	0.82	-0.90	1.00	-1.08	1.18	-1.29	1.39	-1.51
	4	5	0.56	-0.61	0.62	-0.68	0.77	-0.84	0.93	-1.02	1.11	-1.22	1.30	-1.43
	4	10	0.53	-0.58	0.59	-0.65	0.73	-0.80	0.89	-0.98	1.05	-1.16	1.24	-1.36
	4	50	0.46	-0.52	0.52	-0.58	0.64	-0.71	0.78	-0.87	0.92	-1.03	1.09	-1.21
	5	1	0.62	-0.83	0.70	-0.93	0.86	-1.15	1.04	-1.39	1.24	-1.66	1.46	-1.95
	5	2	0.59	-0.78	0.67	-0.87	0.82	-1.08	1.00	-1.30	1.18	-1.55	1.39	-1.82
	5	5	0.56	-0.70	0.62	-0.79	0.77	-0.97	0.93	-1.18	1.11	-1.40	1.30	-1.64
	5	10	0.53	-0.65	0.59	-0.72	0.73	-0.90	0.89	-1.08	1.05	-1.29	1.24	-1.51
	5	50	0.46	-0.52	0.52	-0.58	0.64	-0.71	0.78	-0.87	0.92	-1.03	1.09	-1.21

Components and Cladding - Method 1		$h \le 18m$
Figure 7.1-2 (Cont'd)	Design Wind Pressures	**Walls & Roofs**
Enclosed Buildings		

Roof Overhang Net Design Wind Pressure , p_{net10} (kN/m^2) (Exposure B at h = 10 m with I = 1.0)

	Zone	Effective Wind area (m^2)	Basic Wind Speed V (km/h)				
			145	160	175	190	205
Roof 0 to 7 degrees	2	1	-1.01	-1.24	-1.50	-1.79	-2.10
	2	2	-0.99	-1.22	-1.47	-1.76	-2.06
	2	5	-0.96	-1.19	-1.44	-1.71	-2.01
	2	10	-0.95	-1.17	-1.41	-1.68	-1.97
	3	1	-1.66	-2.04	-2.47	-2.94	-3.45
	3	2	-1.30	-1.60	-1.94	-2.31	-2.71
	3	5	-0.83	-1.02	-1.24	-1.47	-1.73
	3	10	-0.48	-0.58	-0.71	-0.84	-0.99
Roof > 7 to 27 degrees	2	1	-1.30	-1.60	-1.94	-2.31	-2.71
	2	2	-1.30	-1.60	-1.94	-2.31	-2.71
	2	5	-1.30	-1.60	-1.94	-2.31	-2.71
	2	10	-1.30	-1.60	-1.94	-2.31	-2.71
	3	1	-2.19	-2.70	-3.27	-3.89	-4.56
	3	2	-1.97	-2.44	-2.95	-3.51	-4.12
	3	5	-1.69	-2.09	-2.53	-3.01	-3.53
	3	10	-1.48	-1.82	-2.21	-2.63	-3.08
Roof > 27 to 45 degrees	2	1	-1.18	-1.46	-1.77	-2.10	-2.47
	2	2	-1.15	-1.42	-1.71	-2.04	-2.39
	2	5	-1.10	-1.36	-1.64	-1.95	-2.29
	2	10	-1.06	-1.31	-1.59	-1.89	-2.22
	3	1	-1.18	-1.46	-1.77	-2.10	-2.47
	3	2	-1.15	-1.42	-1.71	-2.04	-2.39
	3	5	-1.10	-1.36	-1.64	-1.95	-2.29
	3	10	-1.06	-1.31	-1.59	-1.89	-2.22

ADJUSTMENT FACTOR
FOR BUILDING HEIGHT AND EXPOSURE, λ

Mean roof height (m)	Exposure		
	B	C	D
4.5	1	1.21	1.47
6.0	1	1.29	1.55
7.5	1	1.35	1.61
9.0	1	1.4	1.66
10.5	1.05	1.45	1.7
12.0	1.09	1.49	1.74
13.5	1.12	1.53	1.78
15.0	1.16	1.56	1.81
16.5	1.19	1.59	1.84
18.0	1.22	1.62	1.87

Main Wind Force Resisting System / Components and Cladding - Method 2		All Heights
Figure 7.2-1	**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:

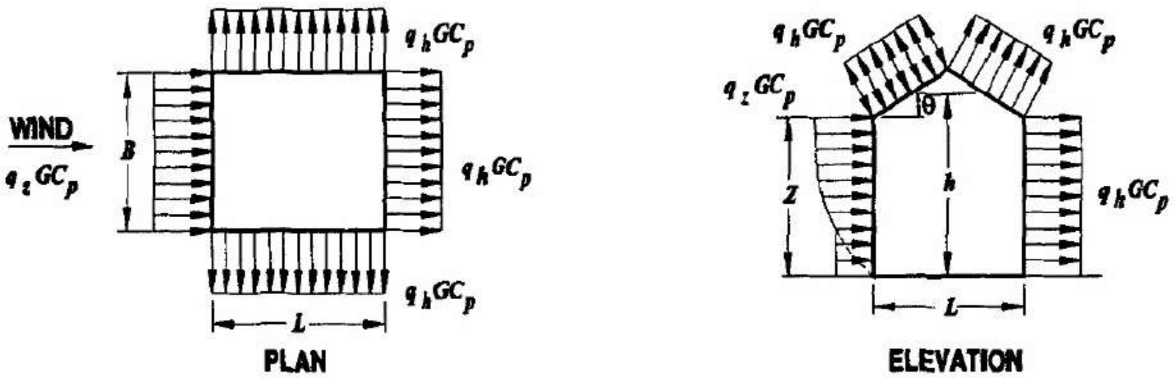
Plus and minus signs signify pressures acting toward and away from the internal surfaces, respectively.

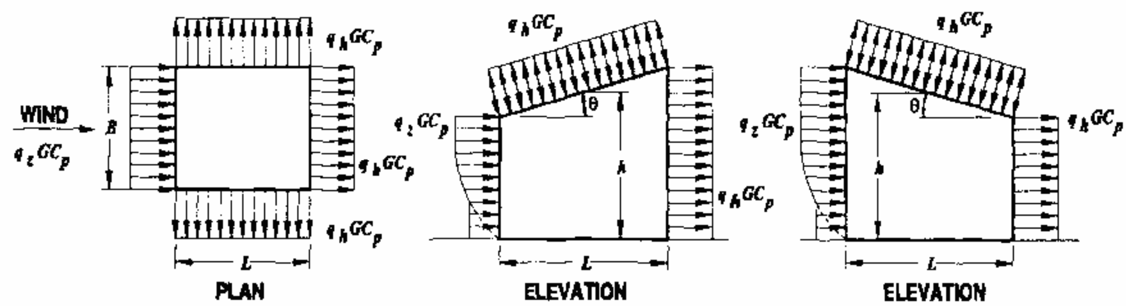
Values of GC_{pi} shall be used with q_Z or q_h, as specified in 7.2.10.

Two cases shall be considered to determine the critical load requirements for the appropriate condition:

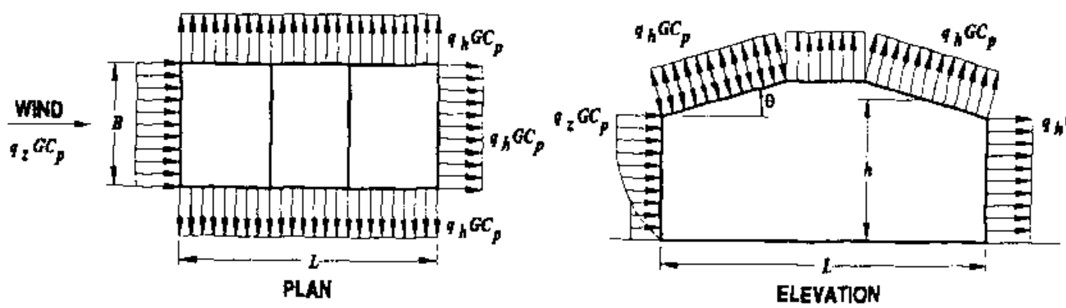
(i) a positive value of GC_{pi} applied to all internal surfaces
(ii) a negative value of GC_{pi} applied to all internal surfaces

Main Wind Force Resisting System - Method 2		All Heights
Figure 7.2-2	External Pressure Coefficient, C_p	Walls & Roofs
Enclosed, Partially Enclosed Buildings		





MONOSLOPE ROOF (NOTE 4)



MANSARD ROOF (NOTE 8)

Main Wind Force Resisting System Method 2		All Heights
Figure 7.2-2 (Cont'd)	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 $\theta \geq 10^0$	≤ 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	Horizontal distance from windward edge	Cp
Normal to ridge for $\theta < 10^o$ 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

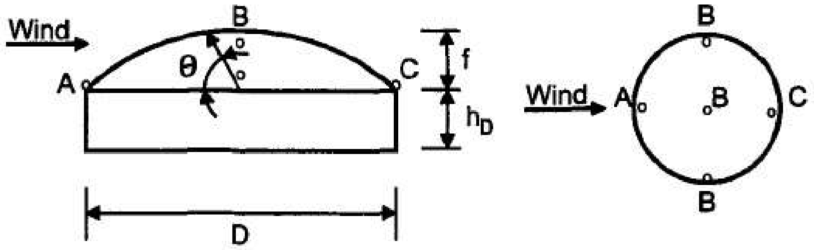
* Value is provided for interpolation purposes.
**Value can be reduced linearly with area over which it is applicable as follows

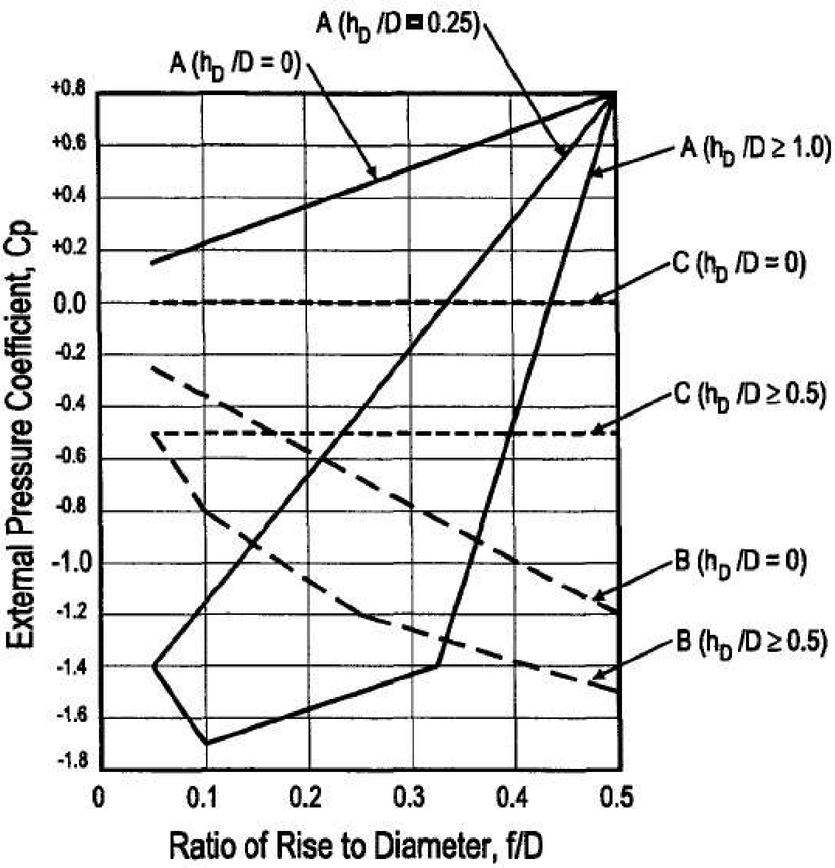
Area (sq m)	Reduction Factor
≤ 10 sq m	1.0
20 sq m	0.9
≥ 100 sq m	0.8

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 7.2.7.2.
6. Refer to Figure 7.2-3 for domes and Figure 7.2-4 for arched roofs.
7. Notation:
   - B: Horizontal dimension of building, in metre, measured normal to wind direction.
   - L: Horizontal dimension of building, in metre, measured parallel to wind direction.
   - h: Mean roof height in metres, except that eave height shall be used for θ ≤ 10 degrees.
   - z: Height above ground, in metres.
   - G: Gust effect factor.
   - q_z, q_h: Velocity pressure, in kN/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 7.2-3	External Pressure Coefficient, C_p	Domed Roofs
Enclosed, Partially Enclosed Buildings and Structures		





External Pressure Coefficients for Domes with a Circular Base.

(Adapted from Eurocode, 1995)

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 $\theta \le 25$ 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. θ = 0 degrees on dome spring line, θ = 90 degrees at dome center top point. f is measured from spring line 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 7.2-2.

Main Wind Force Resisting System - Method 2		All Heights
Figure 7.2-4	External Pressure Coefficients, C_p	**Arched Roofs**
Enclosed, Partially Enclosed Buildings and Structures		

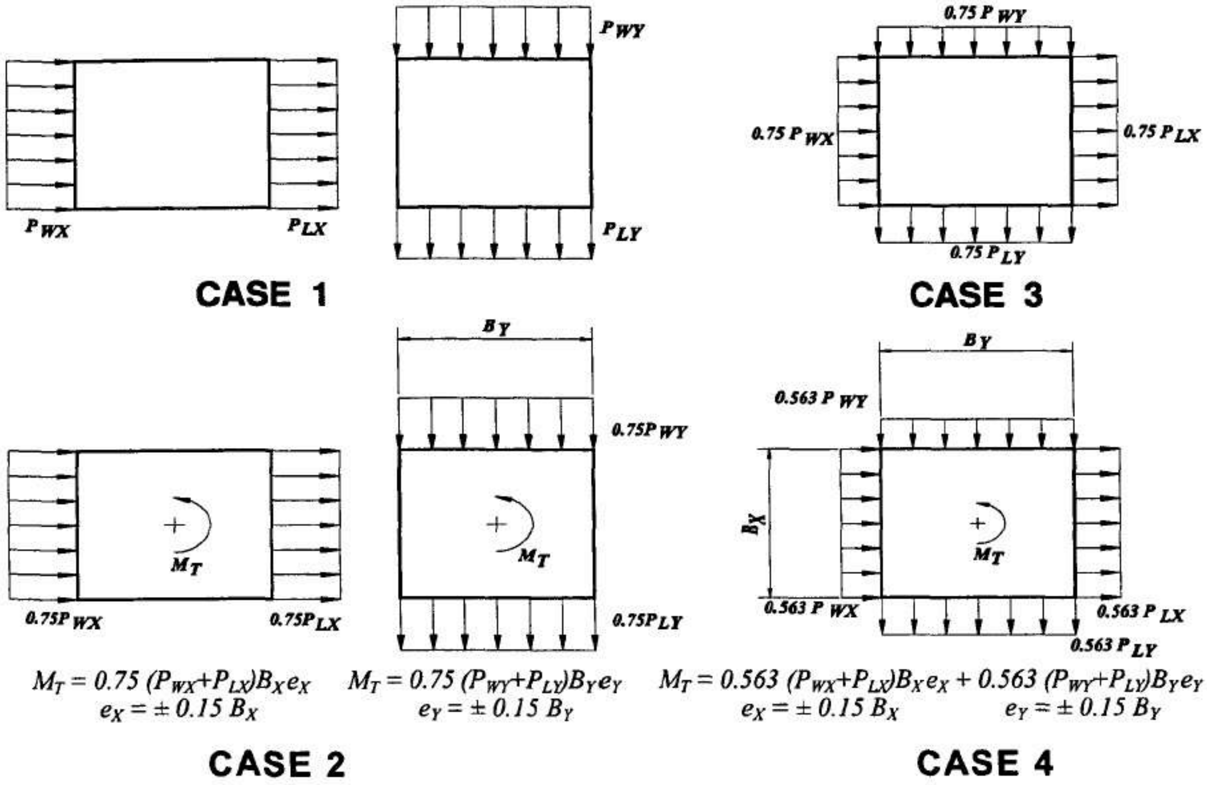
Conditions	Rise-to-span ratio, r	C_P		
		Windward quarter	Center half	Leeward quarter
Roof on elevated structure	0 < r < 0.2	-0.9	-0.7 - *r*	-0.5
	0.2 ≤ r < 0.3*	1.5r - 0.3	-0.7 - r	-0.5
	0.3 ≤ r ≤ 0.6	2.75r - 0.7	-0.7 - *r*	-0.5
Roof springing from ground level	0 < *r* ≤ 0.6	1.4r	-0.7 - r	-0.5

* When the rise-to-span ratio is 0.2 ≤ r ≤ 0.3, alternate coefficients given by 6r - 2.1 shall also be used for the windward quarter.

Notes:

1. Values listed are for the determination of average loads on main wind force resisting systems.
2. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
3. For wind directed parallel to the axis of the arch, use pressure coefficients from Fig. 7.2-2 with wind directed parallel to ridge.
4. For components and cladding: (1) At roof perimeter, use the external pressure coefficients in Fig. 7.2-7 with θ based on spring-line slope and (2) for remaining roof areas, use external pressure coefficients of this table multiplied by 0.87.

Main Wind Force Resisting System - Method 2		All Heights
Figure 7.2-5	**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.

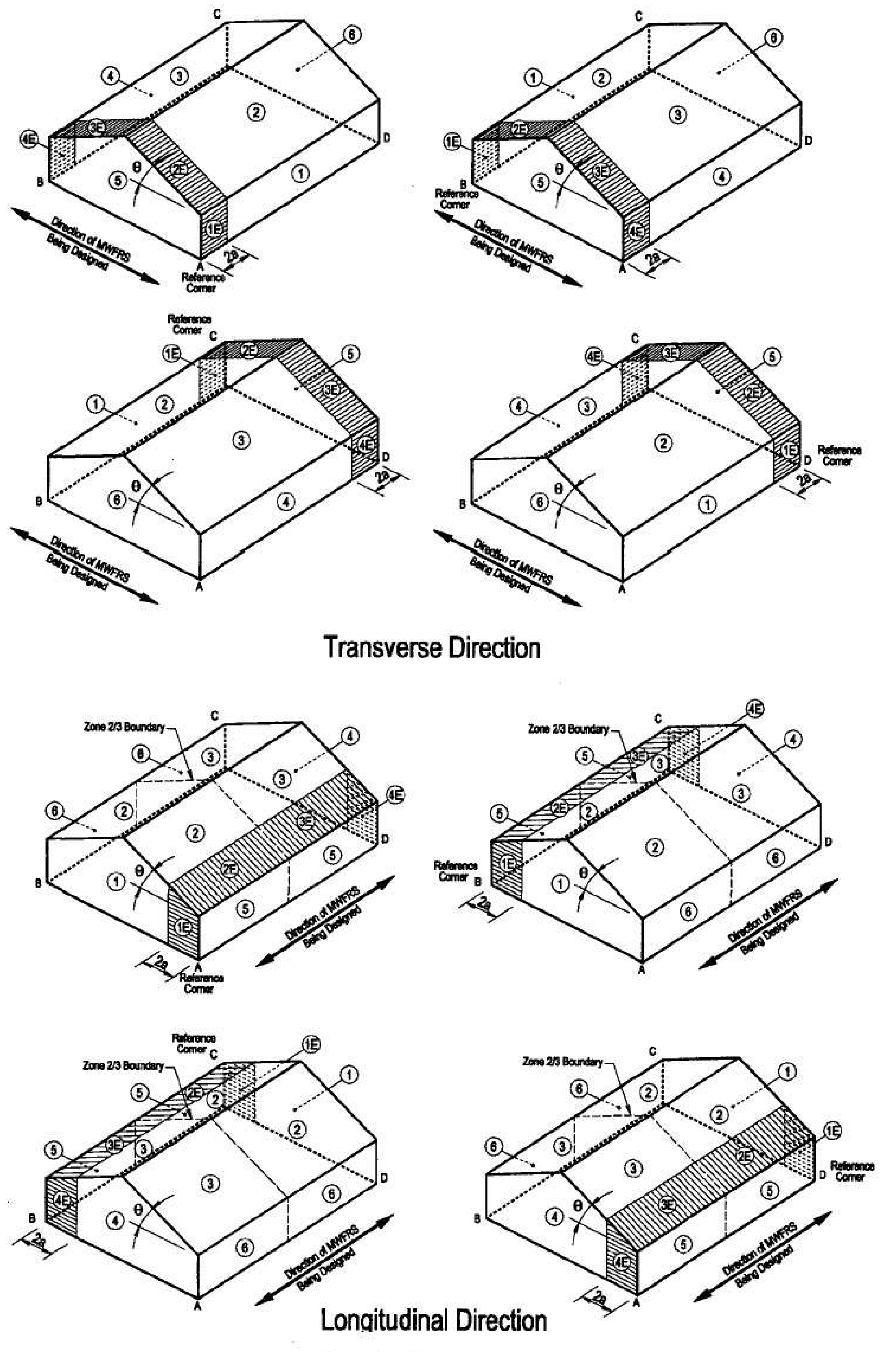
Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 7.2.10.2.1 and 7.2.10.2.3 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		$h \le 18m$
Figure 7.2-6	External Pressure Coefficients, GC_{pf}	Low-rise Walls & Roofs
Enclosed, Partially Enclosed Buildings		



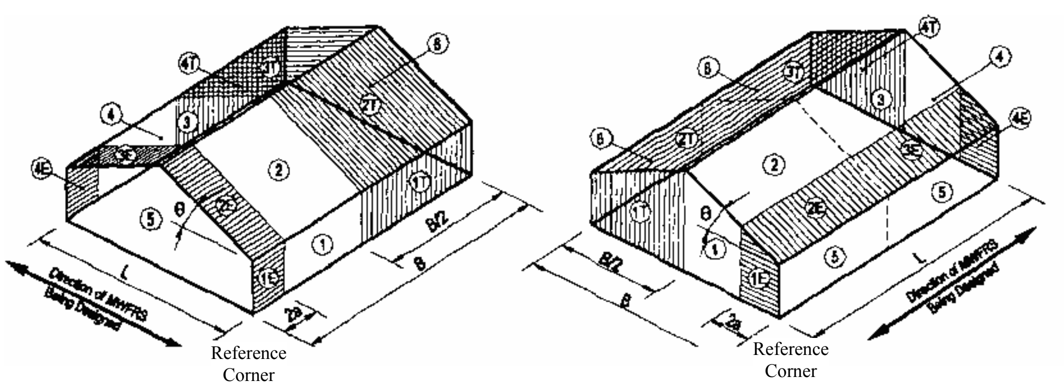
Basic Load Cases

Main Wind Force Resisting System - Method 2		$h \leq 18m$
Figure 7.2-6 (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:

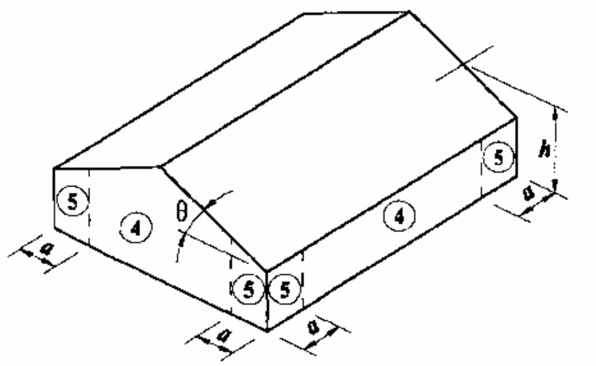
1. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
2. For values of θ other than those shown, linear interpolation is permitted.
3. The building must be designed for all wind directions using the 8 loading patterns shown. The load patterns are applied to each building corner in turn as the Reference Corner.
4. Combinations of external and internal pressures (see Figure 7.2-1) 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 9m, 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, shall be applied in Zone 2 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 2.5h, whichever is less; the remainder of Zone 2 extending shall use the pressure coefficient GC_{pf} for Zone 3.
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 0.9 m.
   h: Mean roof height, in meters, except that eave height shall be used for θ ≤ 10°.
   θ : Angle of plane of roof from horizontal, in degrees.

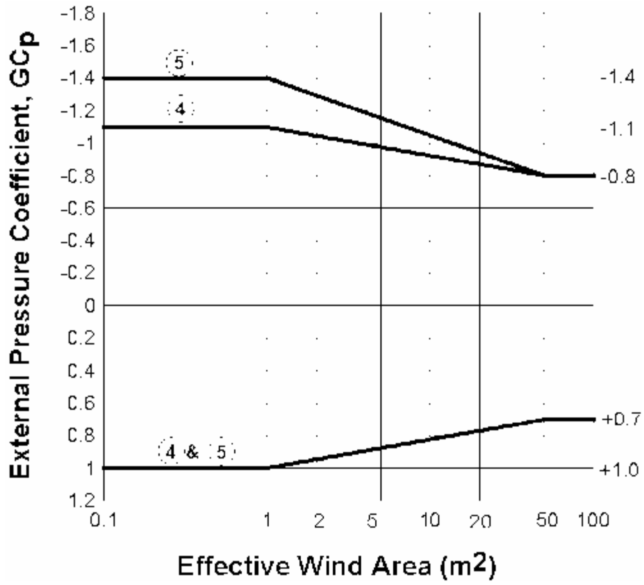


Transverse Direction **Longitudinal Direction**

Torsional Load Cases

Components and Cladding - Method 2		$h \le 18m$
Figure 7.2-7A	**External Pressure Coefficients, GC_p**	**Walls**
Enclosed, Partially Enclosed Buildings		

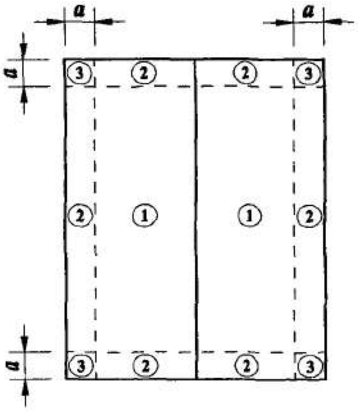


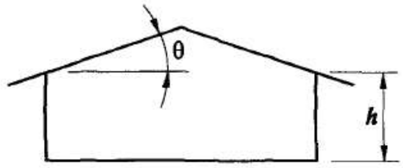


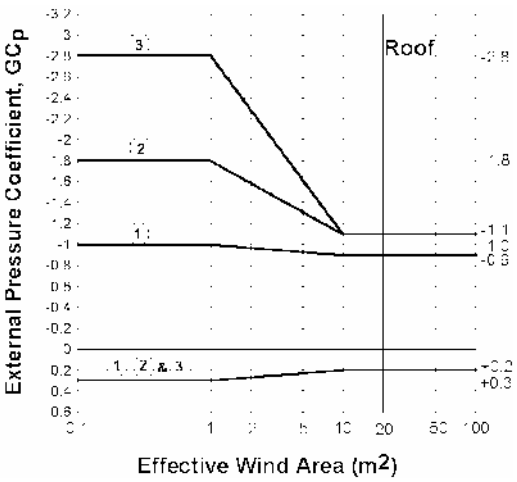
Notes:

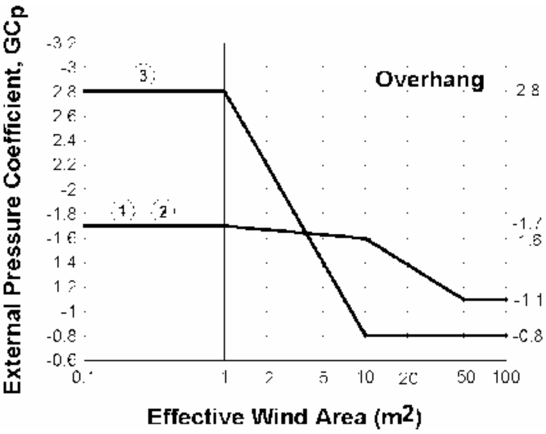
1. Vertical scale denotes GC_P to be used with q_h.
2. Horizontal scale denotes effective wind area, in square metres.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of GC_P for walls shall be reduced by 10% when $\theta \le 10^0$.
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 0.9 m.
   - h: Mean roof height, in metres, except that eave height shall be used for $\theta \le 10^0$.
   - θ: Angle of plane of roof from horizontal, in degrees.

Components and Cladding - Method 2		$h \leq 18m$
Figure 7.2-7B	**External Pressure Coefficients, GC_p**	**Gable Roof $\theta \leq 7^o$**
Enclosed, Partially Enclosed Buildings		





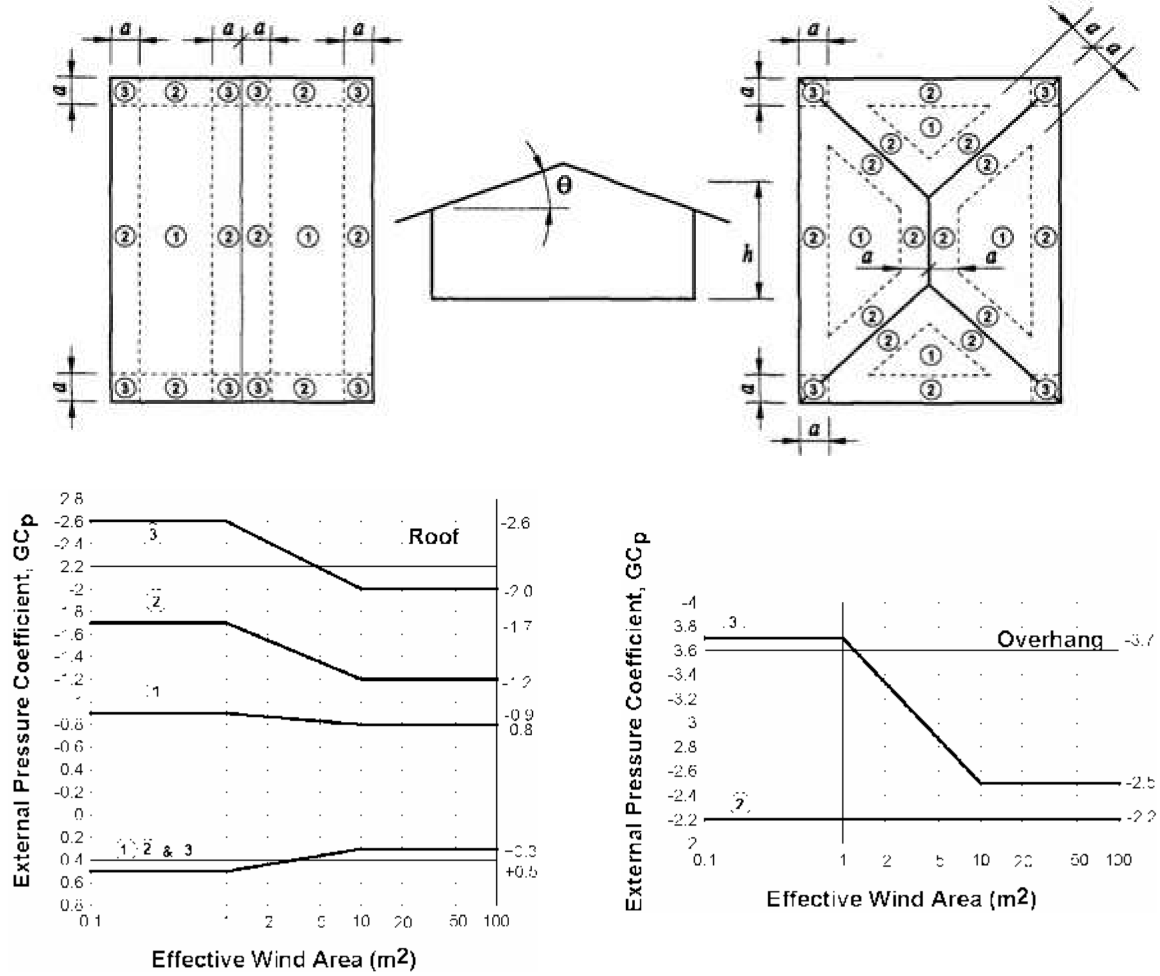




Notes:

1. Vertical scale denotes GC_p to be used with q_h.
2. Horizontal scale denotes effective wind area, in square metres.
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 0.9m is provided around the perimeter of the roof with $\theta \leq 7^0$, Zone 3 shall be treated as Zone 2.
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 0.9 m.
   h: Eave height shall be used for $\theta \leq 10^0$
   θ : Angle of plane of roof from horizontal, in degrees.

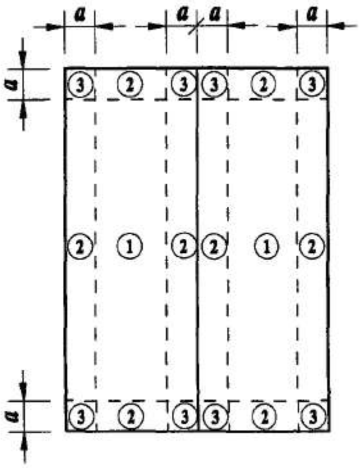
Components and Cladding - Method 2		$h \le 18m$
Figure 7.2-7C	**External Pressure Coefficients, GC_p**	**Gable/Hip Roof** $7^o < \theta \le 27^o$
Enclosed, Partially Enclosed Buildings		

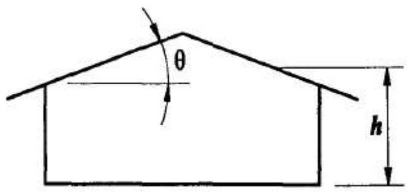


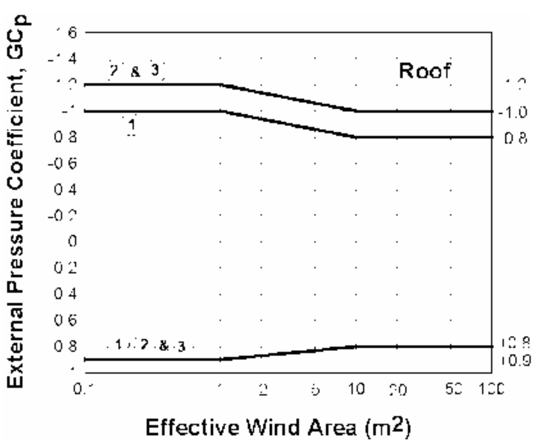
Notes:

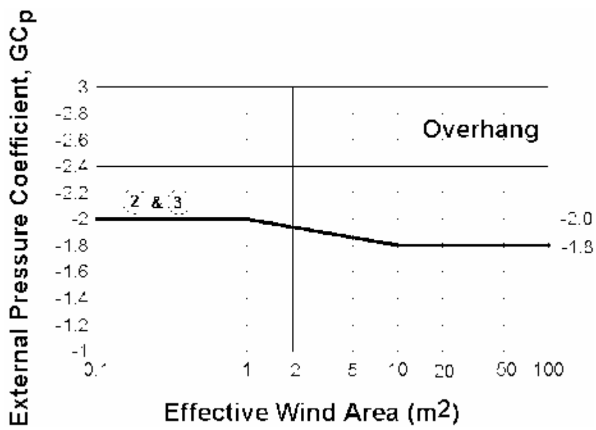
1. Vertical scale denotes GC_p to be used with q_h.
2. Horizontal scale denotes effective wind area, in square metres.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of GC_p for roof overhangs include pressure contributions from both upper and lower surfaces.
6. For hip roofs with $7^o < \theta \le 27^o$ edge/ridge strips and pressure coefficients for ridges of gabled roofs shall apply on each hip.
7. For hip roofs with $\theta \le 25^o$ 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 0.9 m.
   h: Mean roof height, in metres, except that eave height shall be used for $\theta \le 10^o$
   θ : Angle of plane of roof from horizontal, in degrees.

Components and Cladding - Method 2		$h \leq 18m$
Figure 7.2-7D	**External Pressure Coefficients, GC_p**	**Gable Roofs** $27^o < \theta \leq 45^o$
Enclosed, Partially Enclosed Buildings		





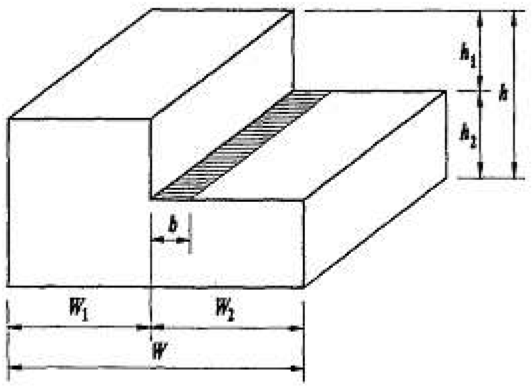




Notes:

1. Vertical scale denotes GC_p to be used with q_h.
2. Horizontal scale denotes effective wind area, in square metres.
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 0.9 m.
   h: Mean roof height, in metres.
   θ: Angle of plane of roof from horizontal, in degrees.

Components and Cladding - Method 2		$h \leq 18m$
Figure 7.2-8	**External Pressure Coefficients, GC_p**	**Stepped Roofs**
Enclosed, Partially Enclosed Buildings		



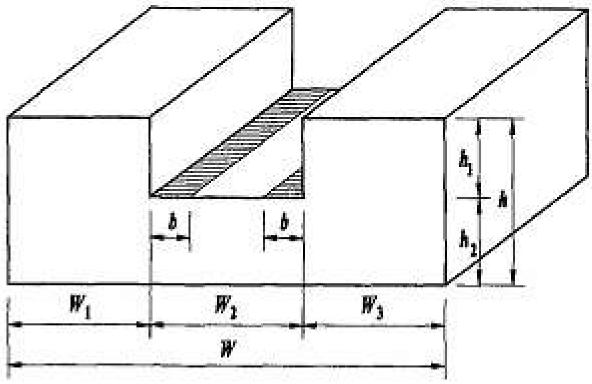
$h_1 \geq 3\,\text{m}$

$b = 1.5\,h_1$

$b < 30.5\,\text{m}$

$\dfrac{h_i}{h} = 0.3 \text{ to } 0.7$

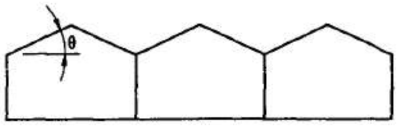
$\dfrac{W_i}{W} = 0.25 \text{ to } 0.75$



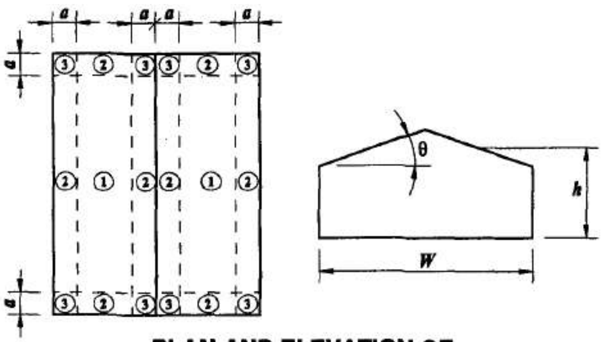
Notes:

1. On the lower level of flat, stepped roofs shown in Fig. 7.2-8, the zone designations and pressure coefficients shown in Fig. 7.2-7 B 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. 7.2-7 A shall apply on the cross-hatched areas shown in Fig. 7.2-8.
2. Notation:
   - *b:* $1.5h_1$ in Fig. 7.2-8, but not greater than *30.5 m.*
   - *h:* Mean roof height, in metres.
   - h_i: h_1 or h_2 in Fig. 7.2-8; $h = h_1 + h_2$; $h_1 \geq 3.1$ *m*; $h_i/h = 0.3$ to 0.7.
   - *W:* Building width in Fig. 7.2-8.
   - W_i: W_1 or W_2 or W_3 in Fig. 7.2-8. $W = W_1 + W_2$ 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.

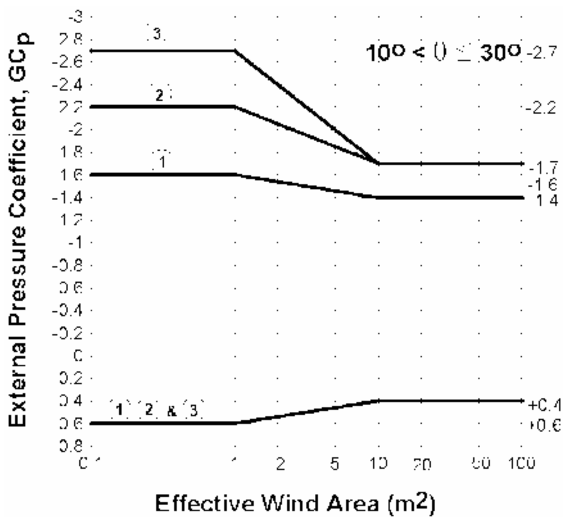
Components and Cladding - Method 2		$h \le 18m$
Figure 7.2-9	External Pressure Coefficients, GC_p	**Multispan Gable Roofs**
Enclosed, Partially Enclosed Buildings		

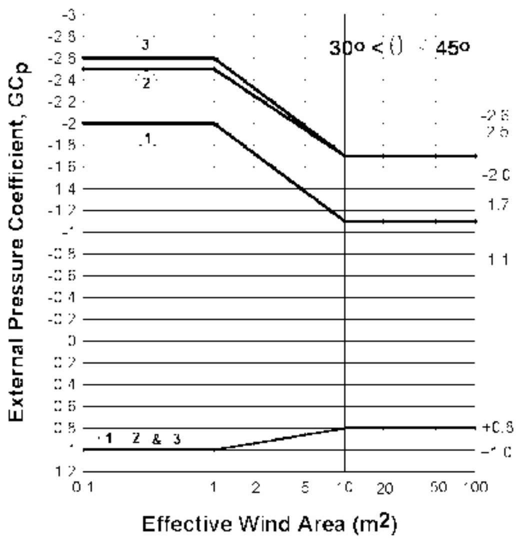


ELEVATION OF BUILDING
(2 or More Spans)



PLAN AND ELEVATION OF A SINGLE SPAN MODULE

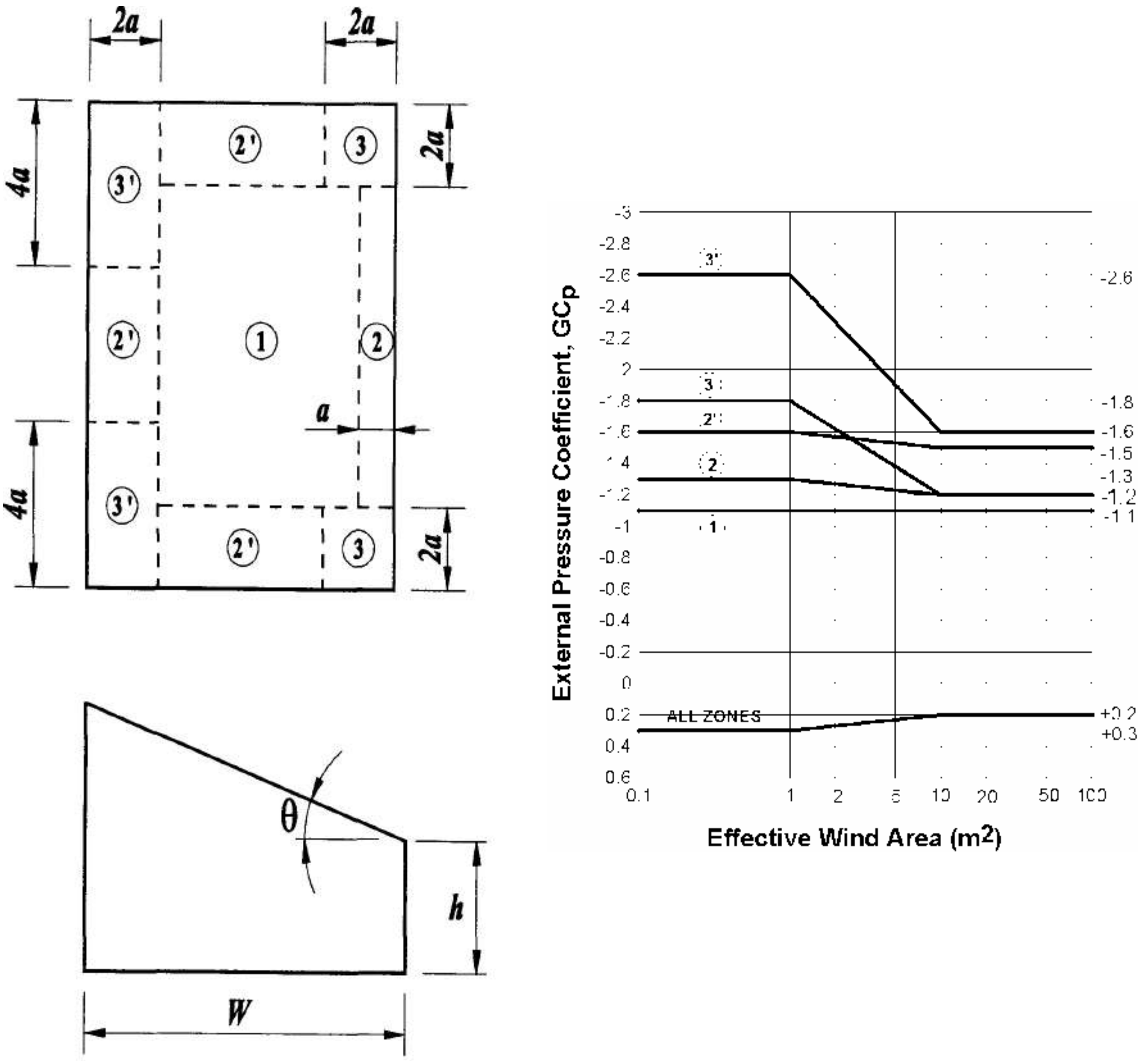




Notes:

1. Vertical scale denotes GC_P to be used with q_h.
2. Horizontal scale denotes effective wind area A, in square metres.
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 $\theta \le 10^o$, values of GC_P from Fig. 7.2-7 shall be used.
6. Notation:
   - a: 10 percent of least horizontal dimension of a single-span module or 0.4h, whichever is smaller, but not less than either 4 percent of least horizontal dimension of a single-span module or 0.9 m.
   - h: Mean roof height, in metres, except that eave height shall be used for $\theta \le 10^o$.
   - W: Building module width, in metres.
   - θ: Angle of plane of roof from horizontal, in degrees.

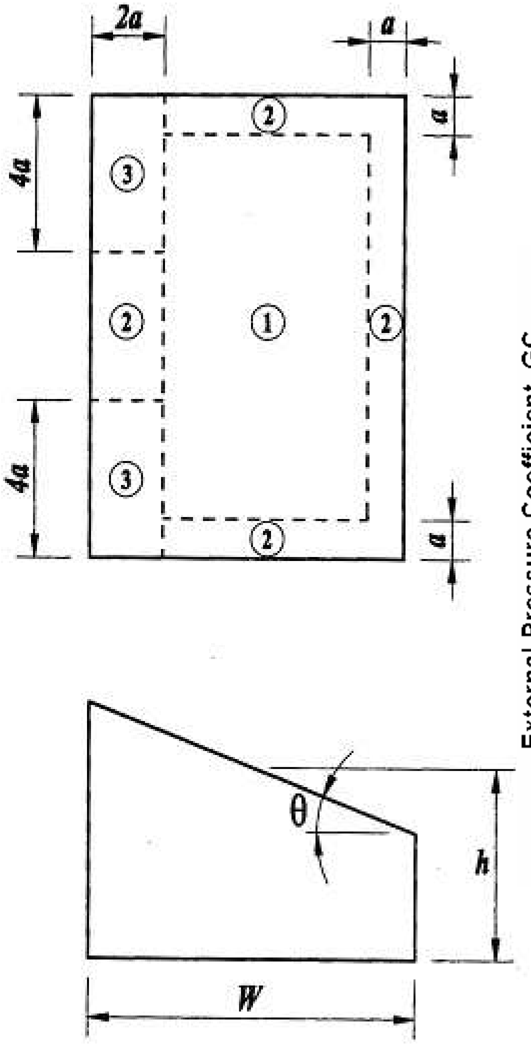
Components and Cladding - Method 2		$h \le 18m$
Figure 7.2-10A	**External Pressure Coefficients, GC_p**	**Monoslope Roofs**
Enclosed, Partially Enclosed Buildings		$3^o < \theta \le 10^o$

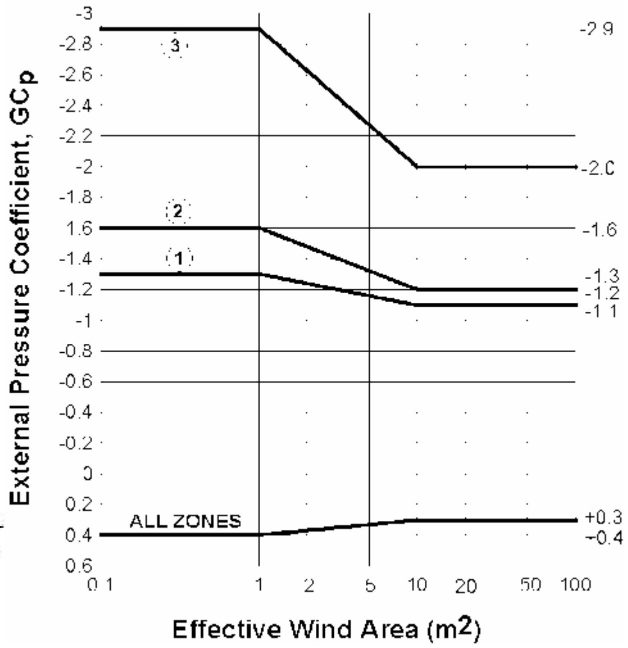


Notes:

1. Vertical scale denotes GC_P to be used with q_h.
2. Horizontal scale denotes effective wind area A, in square metres.
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 $\theta \le 3^o$, values of GC_P from Fig. 7.2-7B shall be used.
6. Notation:
   - a : 10 percent of least horizontal dimension or 0.4 h, whichever is smaller, but not less than either 4 percent of least horizontal dimension or 0.9 m.
   - h: Eave height shall be used for $\theta \le 10^o$.
   - W: Building width, in metres.
   - θ : Angle of plane of roof from horizontal, in degrees.

Components and Cladding - Method 2		$h \leq 18m$
Figure 7.2-10B	**External Pressure Coefficients, GC_p**	**Monoslope Roofs**
Enclosed, Partially Enclosed Buildings		$10^o < \theta \leq 30^o$

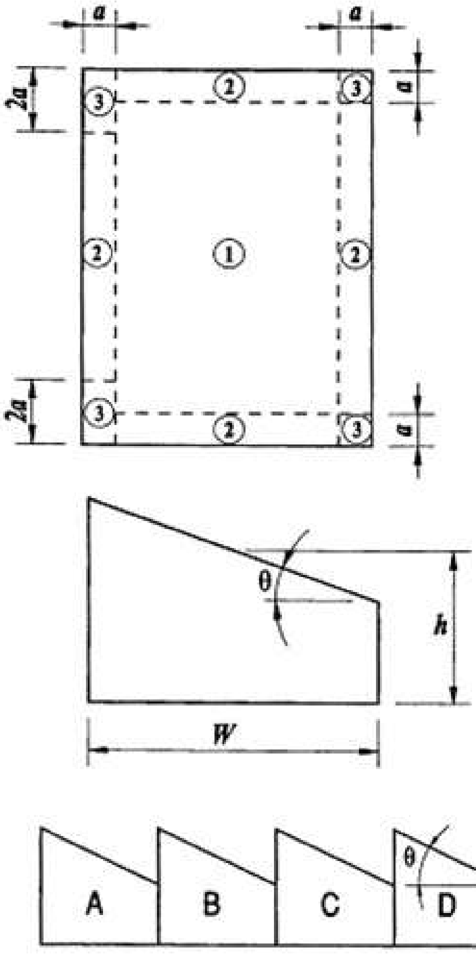




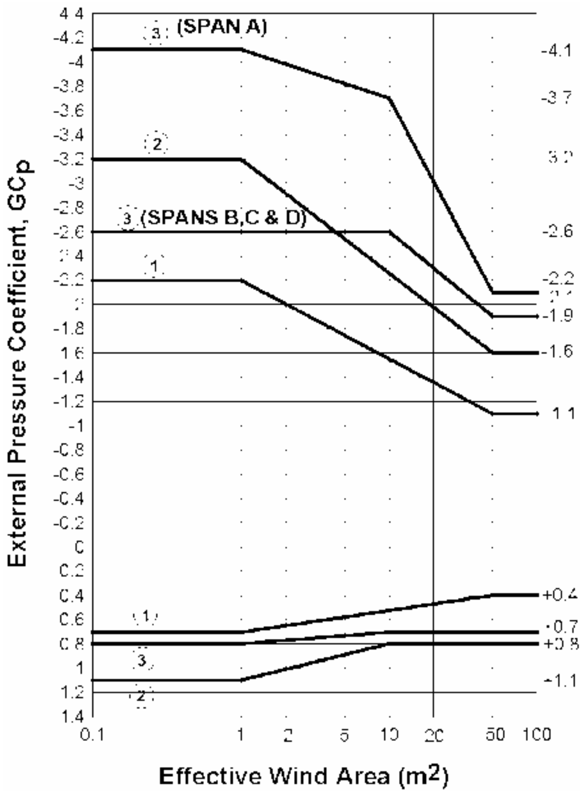
Notes:

1. Vertical scale denotes GC_P to be used with q_h.
2. Horizontal scale denotes effective wind area A, in square metres.
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 0.4 h, whichever is smaller, but not less than either 4 percent of least horizontal dimension or 0.9 m.
   - h: Mean roof height, in metres.
   - W: Building width, in metres.
   - θ : Angle of plane of roof from horizontal, in degrees.

Components and Cladding - Method 2		$h \leq 18m$
Figure 7.2-11	**External Pressure Coefficients, GC_p**	**Sawtooth Roofs**
Enclosed, Partially Enclosed Buildings		



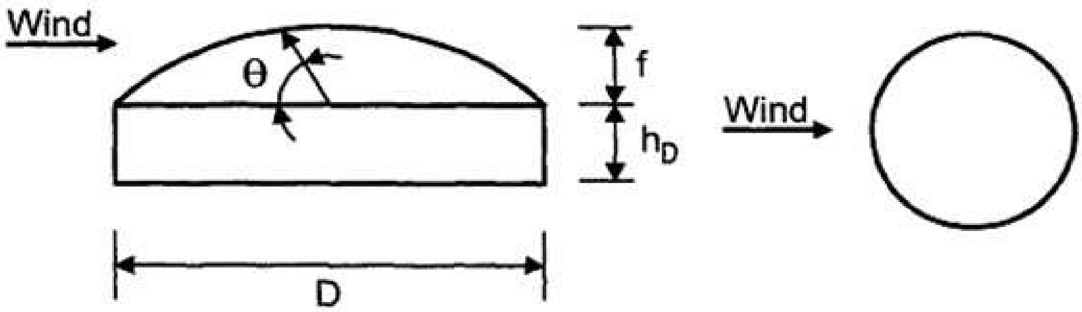
Elevation of Building
(2 or More Spans)



Notes:

1. Vertical scale denotes GC_P to be used with q_h.
2. Horizontal scale denotes effective wind area A, in square metres.
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 $\theta \leq 10^o$ values of GC_P from Fig. 7.2-7 shall be used.
6. Notation:
   - a: 10 percent of least horizontal dimension or 0.4 h, whichever is smaller, but not less than either 4 percent of least horizontal dimension or 0.9 m.
   - h: Mean roof height, in metres, except that eave height shall be used for $\theta \leq 10^o$
   - W: Building width, in metres.
   - θ: Angle of plane of roof from horizontal, in degrees.

Components and Cladding - Method 2		All Heights
Figure 7.2-12	External Pressure Coefficients, GC_p	Domed Roofs
Enclosed, Partially Enclosed Buildings and Structures		

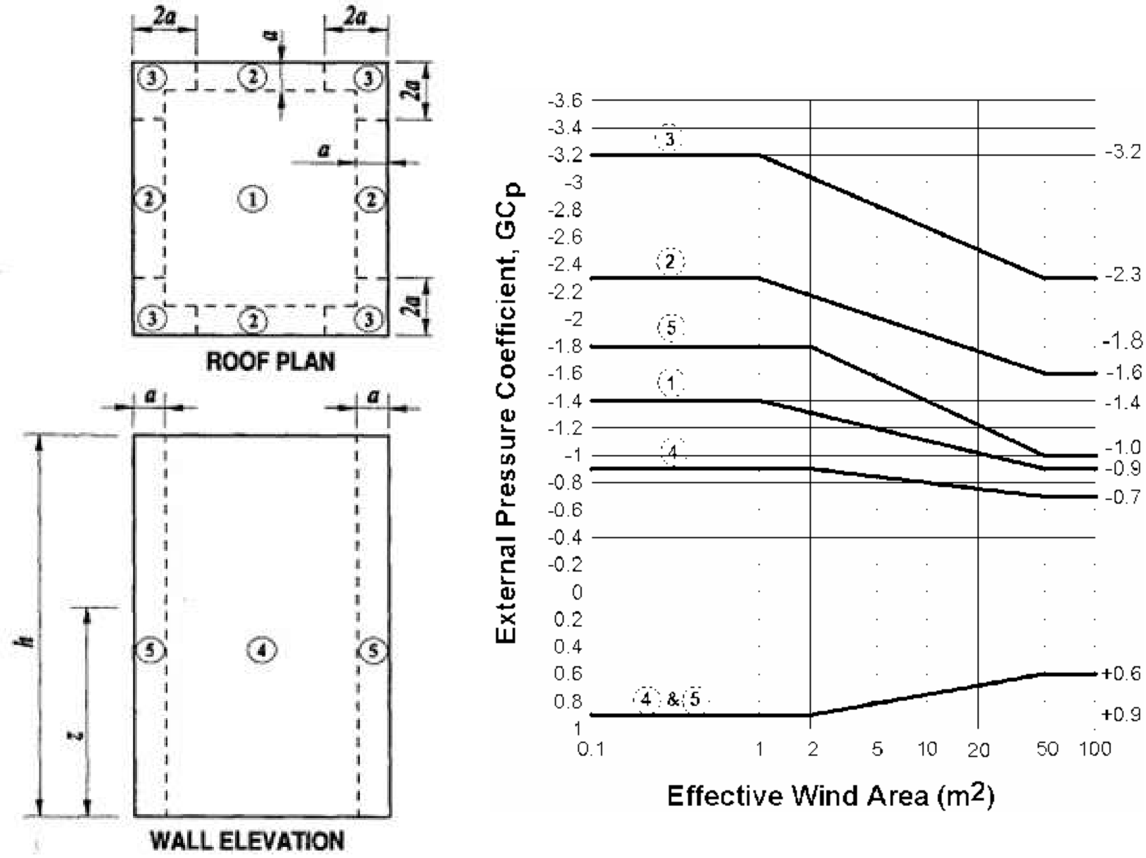


External Pressure Coefficients for Domes with a Circular Base			
	Negative Pressures	Positive Pressures	Positive Pressures
θ, degrees	0 – 90	0 – 60	61 – 90
GC_P	-0.9	+0.9	+0.5

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 $\theta \leq h_D/D \leq 0.5, 0.2 \leq f/D \leq 0.5$.
5. θ = 0 degrees on dome springline, θ = 90 degrees at dome center top point. f is measured from springline to top.

Components and Cladding - Method 2		$h \le 18m$
Figure 7.2-13	**External Pressure Coefficients, GC_p**	**Wall & 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 metres.
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 $\theta \le 10^o$ **.** For other roof angles and geometry, use GC_P values from Fig. 7.2-7 and attendant q_h based on exposure defined in 6.4.2.
7. If a parapet equal to or higher than 0.9 m is provided around the perimeter of the roof with $\theta \le 10^o$ Zone 3 shall be treated as Zone 2.
8. Notation:
   a: 10 percent of least horizontal dimension, but not less than 0.9 m.
   h: Mean roof height, in metres, except that eave height shall be used for $\theta \le 10^o$.
   z: height above ground, in metres.
   θ: Angle of plane of roof from horizontal, in degrees.

Main Wind Force Resisting System - Method 2		All Heights
Figure 7.2-14	Force Coefficients, C_f	Monoslope Roofs
Open Buildings		

Roof angle θ degrees	L/B						
	5	3	2	1	1/2	1/3	1/5
10	0.2	0.25	0.3	0.45	0.55	0.7	0.75
15	0.35	0.45	0.5	0.7	0.85	0.9	0.85
20	0.5	0.6	0.75	0.9	1.0	0.95	0.9
25	0.7	0.8	0.95	1.15	1.1	1.05	0.95
30	0.9	1.0	1.2	1.3	1.2	1.1	1.0

Roof angle θ degrees	Center of Pressure X/L		
	L/B		
	2 to 5	1	1/5 to 1/2
10 to 20	0.35	0.3	0.3
25	0.35	0.35	0.4
30	0.35	0.4	0.45

Notes:

1. Wind forces act normal to the surface. Two cases shall be considered: (1) wind forces directed inward; and (2) wind forces directed outward.
2. The roof angle shall be assumed to vary ± 10^0 from the actual angle and the angle resulting in the greatest force coefficient shall be used.
3. Notation:
   B: Dimension of roof measured normal to wind direction, in metres;
   L: Dimension of roof measured parallel to wind direction, in metres;
   X: Distance to center of pressure from windward edge of roof, in metres; and
   θ : Angle of plane of roof from horizontal, in degrees.

Other structures - Method 2		All Heights
Figure 7.2-15	Force Coefficients, C_f	Chimneys, Tanks, Rooftop, Equipment and 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 ($D\sqrt{q_z} > 0.17$, D in m, q_z in kN/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	1.2
Round ($D\sqrt{q_z} \leq 0.17$, D in m, q_z in kN/m^2)	All	0.7	0.8	1.2

Notes:

1. 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 metres;

   D': depth of protruding elements such as ribs and spoilers, in metres; and

   h: height of structure, in metres; and

   q_z: velocity pressure evaluated at height z above ground, in kN/m^2.

Other structures - Method 2		All Heights
Figure 7.2-16	Force Coefficients, C_f	Solid Freestanding Walls and Solid Signs

At Ground Level		Above Ground Level	
ν	C_f	M/N	C_f
≤ 3	1.2	≤ 6	1.2
5	1.3	10	1.3
8	1.4	16	1.4
10	1.5	20	1.5
20	1.75	40	1.75
30	1.85	60	1.85
≥ 40	2.0	≥ 80	2.0

Notes:

1. The term "signs" in notes below applies also to "freestanding walls".
2. Signs with openings comprising less than 30% of the gross area are classified as solid signs.
3. Signs for which the distance from the ground to the bottom edge is less than 0.25 times the vertical dimension shall be considered to be at ground level.
4. To allow for both normal and oblique wind directions, two cases shall be considered
   a. resultant force acts normal to the face of the sign on a vertical line passing through the geometric center, and
   b. resultant force acts normal to the face of the sign at a distance from a vertical line passing through the geometric center equal to 0.2 times the average width of the sign.
5. Notation:
   ν : ratio of height to width
   M: larger dimension of sign, in metres; and
   N: smaller dimension of sign, in metres.

Other structures - Method 2		All Heights
Figure 7.2-17	**Force Coefficients, C_f**	**Open Signs and Lattice Frameworks**

ε	Flat-Sided Members	Rounded Members	
		$D\sqrt{q_z} \leq 0.17$	$D\sqrt{q_z} > 0.17$
< 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:

   ε : ratio of solid area to gross area;

   D: diameter of a typical round member, in metres;

   q_z: velocity pressure evaluated at height z above ground in kN/m^2.

Other structures - Method 2		All Heights
Figure 7.2-18	**Force Coefficients, C_f**	**Trussed Towers**
Open Structures		

Tower Cross Section	C_f
Square	$4.0\,\varepsilon^2 - 5.9\,\varepsilon + 4.0$
Triangle	$3.4\,\varepsilon^2 - 4.7\,\varepsilon + 3.4$

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:

   $0.51\,\varepsilon^2 + 0.57$, 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:

   $1 + 0.75\,\varepsilon$, 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. Notation:

   ε : ratio of solid area to gross area of one tower face for the segment under consideration.

Terrain Exposure Constants

Table 7.2-1

Exposure	α	Z_g(m)	$\hat{a}$	$\hat{b}$	$\bar{a}$	$\bar{b}$	c	ℓ(m)	$\bar{\varepsilon}$	Z_{min}(m)*
B	7.0	365	1/7	0.84	1/4.0	0.45	0.30	100	1/3.0	10
C	9.5	275	1/9.5	1.00	1/6.5	0.65	0.20	150	1/5.0	5
D	11.5	215	1/11.5	1.07	1/9.0	0.80	0.15	200	1/8.0	2.5

*z_{min} = minimum height used to ensure that the equivalent height $\bar{z}$ is greater of 0.6h or z_{min}.

For buildings with h $\leq$ z_{min}, $\bar{z}$ shall be taken as z_{min}.

Velocity Pressure Exposure Coefficients, K_h and K_z

Table 7.2-2

Height above Ground Level, z	**Exposure (Note 1)**			
	B		**C**	**D**
(m)	**Case 1**	**Case 2**	**Cases 1 & 2**	**Cases 1 & 2**
0-5	0.72	0.59	0.86	1.04
6	0.72	0.62	0.90	1.08
8	0.72	0.67	0.95	1.13
10	0.72	0.72	1.00	1.18
12	0.76	0.76	1.04	1.22
14	0.79	0.79	1.07	1.25
16	0.82	0.82	1.10	1.28
18	0.85	0.85	1.13	1.31
20	0.88	0.88	1.16	1.33
22	0.90	0.90	1.18	1.35
25	0.93	0.93	1.21	1.38
30	0.98	0.98	1.26	1.43
35	1.03	1.03	1.30	1.47
40	1.07	1.07	1.34	1.50
50	1.14	1.14	1.40	1.56
60	1.20	1.20	1.46	1.61
75	1.28	1.28	1.53	1.67
90	1.35	1.35	1.59	1.73
105	1.41	1.41	1.64	1.77
120	1.46	1.46	1.69	1.82
135	1.51	1.51	1.73	1.85
150	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 7.2-6.
   Case 2: a. All main wind force resisting systems in buildings except those in low-rise buildings designed using Figure 7.2-6.
   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 $5\text{m} \le z \le z_g$: $K_z = 2.01\ (z/z_g)^{2/\alpha}$

   For $z < 5\text{m}$: $K_z = 2.01\ (5/z_g)^{2/\alpha}$

   Note: z shall not be taken less than 10 m for Case 1 in exposure B.

3. α and z_g are tabulated in Table 7.2-1.

4. Linear interpolation for intermediate values of height z is acceptable.

5. Exposure categories are defined in 6.4.2.

CHAPTER 8
RAIN LOADS

SECTION 8.1
SYMBOLS AND NOTATIONS

A = roof area serviced by a single drainage system, in m^2.
Q = flow rate out of a single drainage system, in m^3/s.
R = rain load on the undeflected roof, in 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.
I_n = design rainfall intensity, 180 mm/h for all regions of Saudi Arabia, for fifty years recurrence interval and for storm duration up to two hours.
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 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 mm.

SECTION 8.2
ROOF DRAINAGE

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

SECTION 8.3
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.

$$R = 0.0098\ (d_s + d_h) \qquad \textbf{(Eq. 8-1)}$$

$d_s + d_h \geq 200$ mm

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

The depth of water, d_h, above the inlet of the secondary drainage system (i.e., the hydraulic head) is a function of the rainfall intensity at the site, the area of roof serviced by that drainage system, and the size of the drainage system.
The flow rate through a single drainage system is as follows:

$$Q = 0.278 \times 10^{-6}\ A\ I_n \qquad \textbf{(Eq. 8-2)}$$

The hydraulic head, d_h, is related to flow rate, Q, for various drainage systems in Table 8-1. The hydraulic head, d_h, is zero when the secondary drainage system is simply overflow all along a roof edge.

SECTION 8.4
PONDING INSTABILITY

"Ponding" refers to the retention of water due solely to the deflection of relatively flat roofs. Roofs with a slope less than 1.2° (degrees) shall be investigated by structural analysis to ensure 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.

SECTION 8.5
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 8.3).

Such roofs shall also be checked for ponding instability (determined from Section 8.4). Table 8-1 shows flow rate in cubic meters per second of various drainage systems at various hydraulic heads.

Table 8-1:
Flow rate, Q (m^3/sec) of various drainage systems at various hydraulic heads, d_h in millimetres.

	HYDRAULIC HEAD d_h mm						
Drainage System	**25**	**50**	**75**	**100**	**125**	**175**	**200**
100 mm diameter drain	.0051	.0107					
150 mm diameter drain	.0063	.0120	.0240				
200 mm diameter drain	.0079	.0145	.0353	.0694			
150 mm wide, channel scupper*	.0011	.0032	.0057	.0088	.0122	.0202	.0248
600 mm wide, channel scupper	.0045	.0126	.0227	.0353	.0490	.0810	.0992
150 mm wide, 100 mm high, closed scupper*	.0011	.0032	.0057	.0088	.0112	.0146	.0160
600 mm wide, 100 mm high, closed scupper	.0045	.0126	.0227	.0353	.0447	.0583	.0638
150 mm wide, 150 mm high, closed scupper	.0011	.0032	.0057	.0088	.0122	.0191	.0216
600 mm wide, 150 mm high, closed scupper	.0045	.0126	.0227	.0353	.0490	.0765	.0866

* Channel scuppers are open-topped (i.e., 3-sided). Closed scuppers are 4-sided.

CHAPTER 9
SEISMIC DESIGN CRITERIA

SECTION 9.1
GENERAL

9.1.1 **Purpose.** Chapters 9 through 16 present criteria for the design and construction of buildings and similar structures subject to earthquake ground motions. The specified earthquake loads are based on post-elastic energy dissipation in the structure, and because of this fact, the provisions for design, detailing, and construction shall be satisfied even for structures and members for which load combinations that do not contain the earthquake effect indicate larger demands than combinations including earthquake.

9.1.2 **Scope and Applicability.**

9.1.2.2 **Scope.** Every building, and portion thereof, shall be designed and constructed to resist the effects of earthquake motions as prescribed by these provisions. Certain nonbuilding structures, as described in Chapter 13, are within the scope and shall be designed and constructed as required for buildings. Additions to existing structures also shall be designed and constructed to resist the effects of earthquake motions as prescribed by these provisions. Existing structures and alterations to existing structures need only comply with these provisions when required by Chapter 16.

9.1.2.2 **Applicability.** Structures shall be designed and constructed in accordance with the requirements of the following chapters based on the type of structure:

a. Buildings – Chapter 10

b. Nonbuilding Structures – Chapter 13

SECTION 9.2
DEFINITIONS

The definitions presented in this Section provide the meaning of the terms used in Chapter 9 through 16 of these provisions.

Addition. An increase in building area, aggregate floor area, height, or number of stories 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 these provisions for the intended use.

Architectural Component Support. Those structural members or assemblies of members, including braces, frames, struts, and attachments that transmit all loads and forces between architectural systems, components, or elements and the structure.

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.

Base Shear. Total design lateral force or shear at the base.

Basement. A basement is any story below the lowest story above grade.

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 re-entrant 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 re-entrant corners.

Building. Any structure whose use could include 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 foundation.

Component. A part or 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 0.06 sec.
>
> **Component, rigid.** Component, including its attachments, having a fundamental period less than or equal to 0.06 sec.

Concrete, Plain. Concrete that is either unreinforced or contains less reinforcement than the minimum amount specified in SBC 304 for reinforced concrete.

Concrete, Reinforced. Concrete reinforced with no less than the minimum amount required by SBC 304, prestressed or nonprestressed, and designed on the assumption that the two materials act together in resisting forces.

Confined Region. That portion of a reinforced concrete or reinforced masonry component in which the concrete or masonry is confined by closely spaced special transverse reinforcement restraining the concrete or masonry in directions perpendicular to the applied stress.

Construction Documents. The written, graphic, electronic, and pictorial documents describing the design, locations, and physical characteristics of the project required to verify compliance with this Code.

Container. A large-scale independent component used as a receptacle or vessel to accommodate plants, refuse, or similar uses, not including liquids.

Deformability. The ratio of the ultimate deformation to the limit deformation.

High deformability element. An element whose deformability is not less than 3.5 when subjected to four fully reversed cycles at the limit deformation.

Limited deformability element. An element that is neither a low deformability nor 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% of the maximum strength.

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

Design Earthquake. The earthquake effects that are two-thirds of the corresponding maximum considered earthquake.

Designated Seismic Systems. The seismic force-resisting system and those architectural, electrical, and mechanical systems or their components that require design in accordance with Section 12.1 and for which the component importance factor, I_p, is 1.0.

Diaphragm. Roof, floor, or other membrane or bracing system acting to transfer the lateral forces to the vertical resisting elements. Diaphragms are classified as either flexible or rigid according to the requirements of Section 10.3.1.

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 in a manner analogous to the flanges of a beam. Also applies to shear walls.

Displacement.

Design displacement. The design earthquake lateral displacement, excluding additional displacement due to actual and accidental torsion, required for design of the isolation system.

Total design displacement. The design earthquake lateral displacement, including additional displacement due to actual and accidental torsion, required for design of the isolation system or an element thereof.

Total maximum displacement. The maximum considered earthquake lateral displacement, including additional displacement due to actual and accidental torsion, required for verification of the stability of the isolation system or elements thereof, design of structure separations, and vertical load testing of isolator unit prototypes.

Displacement Restraint System. A collection of structural elements that limits lateral displacement of seismically isolated structures due to the maximum considered earthquake.

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.

Essential Facility. A structure required for post-earthquake recovery.

FRAME.

Braced frame. An essentially vertical truss, or its equivalent, of the concentric or eccentric type that is provided in a bearing wall, building frame, or dual system to resist seismic forces.

Concentrically braced frame (CBF). A braced frame in which the members are subjected primarily to axial forces.

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

Ordinary concentrically braced frame (OCBF). A steel concentrically braced frame in which members and connections are designed in accordance with the provisions of SBC 306 without modification.

Special concentrically braced frame (SCBF). A steel or composite steel and concrete concentrically braced frame in which members and connections are designed for ductile behavior. Special concentrically braced frames shall conform to Section 15.4.

Moment frame.

Intermediate moment frame (IMF). A moment frame in which members and joints are capable of resisting forces by flexure as well as along the axis of the members. Intermediate moment frames of reinforced concrete shall conform to SBC 304. Intermediate moment frames of structural steel construction shall conform to Ref. 11.1-1. Intermediate moment frames of composite construction shall conform to Ref. 11.3-1, Part II, Section 6.4b, 7, 8, and 10.

Ordinary moment frame (OMF). A moment frame in which members and joints are capable of resisting forces by flexure as well as along the axis of the members. Ordinary moment frames shall conform to SBC 304.

Special moment frame (SMF). A moment frame in which members and joints are capable of resisting forces by flexure as well as along the axis of the members. Special moment frames shall conform to SBC 304 for concrete or Ref. 11.1-1 for steel.

FRAME 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 frame system. A structural system with an essentially complete space frame providing support for vertical loads. Seismic force resistance is provided by moment-resisting frames and shear walls or braced frames as prescribed in Section 10.2.1.

Space frame system. A structural system composed of interconnected members, other than bearing walls, that is capable of supporting vertical loads and, when 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 store-front 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 (2.0 m) from the structure, between the structure and a point (2.0 m) 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.

High Temperature Energy Source. A fluid, gas, or vapor whose temperature exceeds 105 °C.

Inspection, Special. The observation of the work by the special inspector to determine compliance with the approved construction documents and these standards.

Continuous special inspection. The full-time observation of the work by an approved 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 an approved 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. The authority having jurisdiction shall have the option to approve the quality assurance personnel of a fabricator as a special inspector.

Inverted Pendulum-Type Structures.

Structures that have a large portion of their mass concentrated near the top and, thus, have essentially one degree of freedom in horizontal translation. The structures are usually T-shaped with a single column supporting the beams or framing at the top.

Joint. The geometric volume common to intersecting members.

LOAD.

Dead load. The gravity load due to the weight of all permanent structural and nonstructural components of a building such as walls, floors, roofs, and the operating weight of fixed service equipment.

Gravity load (W). The total dead load and applicable portions of other loads as defined in Section 10.7.

Live load. The load superimposed by the use and occupancy of the building not including the wind load, earthquake load, or dead load, see Section 10.7.

Maximum Considered Earthquake Ground Motion. The most severe earthquake effects considered by these standards as defined in Section 9.4.3.

Nonbuilding Structure. A structure, other than a building, constructed of a type included in Chapter 13 and within the limits of Section 13.1.1.

Occupancy Importance Factor. A factor assigned to each structure according to its Occupancy Category as prescribed in Section 9.5.

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 displacement of the structure resulting from various loading conditions.

Quality Assurance Plan. A detailed written procedure that establishes the systems and components subject to special inspection and testing. The type and frequency of testing and the extent and duration of special inspection are given in the quality assurance plan.

Registered Design Professional. An engineer, registered or licensed to practice professional engineering.

Roofing Unit. A unit of roofing tile or similar material weighing more than 5 N.

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 9.6.

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.

Seismic Response Coefficient. Coefficient C_s as determined from Section 10.9.2.1.

Shallow Anchor. Anchors with embedment length-to-diameter ratios of less than 8.

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 as defined in Section 9.4.2.

Site Coefficients. The values of F_a and F_v as indicated in Tables 9.4.3a and 9.4.3b, respectively.

Special Lateral Reinforcement. Reinforcement composed of spirals, closed stirrups, or hoops and supplementary crossties provided to restrain the concrete and qualify the portion of the component, where used, as a confined region.

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.

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

Story Above Grade. Any story having its finished floor surface entirely above grade, except that a story shall be considered as a story above grade where the finished floor surface of the story immediately above is more than (2.0 m) above the grade plane, more than (2.0 m) above the finished ground level for more than 40% of the total structure perimeter, or more than (4.0 m) above the finished ground level at any point.

Story Drift. The difference of horizontal deflections at the top and bottom of the story as determined in Section 10.9.7.1.

Story Drift Ratio. The story drift, as determined in Section 10.9.7.1, divided by the story height.

Story Shear. The summation of design lateral seismic forces at levels above the story 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 Code (or the referenced standards) 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 Code.

Structure. That which is built or constructed and limited to buildings and nonbuilding structures as defined herein.

Structural Observations. The visual observations performed by the registered design professional in responsible charge (or another registered design professional) to determine that the seismic force-resisting system is constructed in general conformance with the construction documents.

Subdiaphragm. A portion of a diaphragm used to transfer wall anchorage forces to diaphragm crossties.

Testing Agency. A company or corporation that provides testing and/or inspection services. The person in charge of the special inspector(s) and the testing services shall be a registered design professional.

Tie-down (hold-down). A device used to resist uplift of the boundary elements of shear walls. These devices are intended to resist load without significant slip between the device and the shear wall boundary element or be shown with cyclic testing to not reduce the wall capacity or ductility.

Torsional Force Distribution. The distribution of horizontal shear through a rigid diaphragm when the center of mass of the structure at the level under consideration does not coincide with the center of rigidity (sometimes referred to as diaphragm rotation).

Toughness. The ability of a material to absorb energy without losing significant strength.

Utility or Service Interface. The connection of the structure's mechanical and electrical distribution systems to the utility or service company's distribution system.

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 degrees 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 wall that supports more than (1500 N/m) of vertical load in addition to its own weight.
2. Any concrete or masonry wall that supports more than (3000 N/m) of vertical load in addition to its own weight.

Nonbearing wall. Any wall that is not a bearing wall.

Nonstructural wall. All walls other than bearing walls or shear walls.

Shear wall (vertical diaphragm). A wall, bearing or nonbearing, designed to resist lateral seismic forces acting in the plane of the wall (sometimes referred to as a vertical diaphragm).

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.

SECTION 9.3
SYMBOLS AND NOTATIONS

The unit dimensions used with the items covered by the symbols shall be consistent throughout except where specifically noted. The symbols and definitions presented in this Section apply to these provisions as indicated.

A , B, C, D	=	the Seismic Design Categories as defined in Tables 9.6.a and 9.6.b
A, B, C, D, E, F	=	the Site Classes as defined in Section 9.4.2
A_{ch}	=	cross-sectional area (mm^2) of a component measured to the outside of the special lateral reinforcement
A_o	=	the area of the load-carrying foundation (m^2)
A_{sh}	=	total cross-sectional area of hoop reinforcement (mm^2), including supplementary crossties, having a spacing of s_h and crossing a section with a core dimension of h_c.
A_{vd}	=	required area of leg (mm^2) of diagonal reinforcement
a_p	=	the amplification factor related to the response of a system or component as affected by the type of seismic attachment, determined in Section 12.1.3

b	=	the shortest plan dimension of the structure, in (mm) measured perpendicular to d
C_d	=	the deflection amplification factor as given in Table 10.2
C_s	=	the seismic response coefficient determined in Section 10.9.2 (dimensionless)
C_{sm}	=	the modal seismic response coefficient determined in Section 10.10.5 (dimensionless)
C_{vx}	=	the vertical distribution factor as determined in Section 10.9.4
c	=	distance from the neutral axis of a flexural member to the fiber of maximum compressive strain (mm)
D	=	the effect of dead load
D_p	=	relative seismic displacement that the component must be designed to accommodate as defined in Section 12.1.4
d	=	overall depth of member (mm) in Chapter 10
d_p	=	the longest plan dimension of the structure, in (mm)
E	=	the effect of horizontal and vertical earthquake-induced forces, in Section 10.4
e	=	the actual eccentricity, (mm), measured in plan between the center of mass of the structure above the isolation interface and the center of rigidity of the isolation system, plus accidental eccentricity, in (mm), taken as 5% of the maximum building dimension perpendicular to the direction of force under consideration
F_a	=	acceleration-based site coefficient (at 0.2-sec period)
F_i , F_n, F_x	=	the portion of the seismic base shear, V, induced at Level i, n, or x, respectively, as determined in Section 10.9.4
F_p	=	the seismic force acting on a component of a structure
F_v	=	velocity-based site coefficient (at 1.0-sec period)
F_{xm},	=	the portion of the seismic base shear, V_m, induced at Level x as determined in Section 10.10.6
$f_c^{'}$	=	specified compressive strength of concrete used in design
$f_S^{'}$	=	ultimate tensile strength (MPa) of the bolt, stud, or insert leg wires. For A307 bolts or A108 studs, it is permitted to be assumed to be 415 MPa.
f_y	=	specified yield strength of reinforcement (MPa)
g	=	the acceleration due to gravity
H	=	thickness of soil
h	=	the 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	=	the roof elevation of a structure in Chapter 12
h	=	the effective height of the building as determined in Chapter 10
h_c	=	the core dimension of a component measured to the outside of the special lateral reinforcement (mm)
h_i, h_n, h_x	=	the height above the base Level i, n, or x, respectively
h_{sx}	=	the story height below Level $x = (h_x - h_{x-1})$
I	=	the occupancy importance factor in Section 9.5
I_p	=	the component importance factor as prescribed in Section 12.1.5

i	=	the building level referred to by the subscript i; i = 1 designates the first level above the base
K_p	=	the stiffness of the component or attachment, Section 12.3.3
KL/r	=	the lateral slenderness of a compression member measured in terms of its effective buckling length, KL, and the least radius of gyration of the member cross-section, r
k	=	the distribution exponent given in Section 10.9.4
k	=	the stiffness of the building
L	=	the overall length of the building (m) at the base in the direction being analyzed
l	=	the dimension of a diaphragm perpendicular to the direction of application of force. For open-front structures, ℓ is the length from the edge of the diaphragm at the open-front to the vertical resisting elements parallel to the direction of the applied force. For a cantilevered diaphragm, ℓ is the length of the cantilever
M_f	=	the foundation overturning design moment as defined in Section 10.9.6 (kN-m)
M_t	=	the torsional moment resulting from the location of the building masses, Section 10.9.5
M_{ta}	=	the accidental torsional moment as determined in Section 10.9.5
M_x	=	the building overturning design moment at Level x as defined in Section 10.9.6
N	=	number of stories, Section 10.9.3
$\bar{N}$	=	standard penetration resistance, ASTM D1586-84
N	=	average field standard penetration resistance for the top (30 m), see Section 14.1.1
N_{ch}	=	average standard penetration resistance for cohesionless soil layers for the top (30 m), see Section 14.1.1
n	=	designates the level that is uppermost in the main portion of the building
P_n	=	the algebraic sum of the shear wall and the minimum gravity loads on the joint surface acting simultaneously with the shear (N)
P_x	=	the total unfactored vertical design load at, and above, Level x for use in Section 10.9.7.2
Q_E	=	plasticity index, ASTM D4318-93
R		The effect of horizontal seismic (earthquake-induced) forces, Section 10.4
R_p	=	the component response modification factor as defined in Section 12.1.3
r_x	=	the ratio of the design story shear resisted by the most heavily loaded single element in the story, in direction x, to the total story shear
S_S	=	the mapped maximum considered earthquake, 5% damped, spectral response acceleration at short periods as defined in Section 9.4.1
S_1	=	the mapped maximum considered earthquake, 5% damped, spectral response acceleration at a period of 1 sec as

		defined in Section 9.4.1
S_{DS}	=	the design, 5% damped, spectral response acceleration at short periods as defined in Section 9.4.4
S_{D1}	=	the design, 5% damped, spectral response acceleration at a period of 1 sec as defined in Section 9.4.4
S_{MS}	=	the maximum considered earthquake, 5% damped, spectral response acceleration at short periods adjusted for site class effects as defined in Section 9.4.3
S_{M1}	=	the maximum considered earthquake, 5% damped, spectral response acceleration at a period of 1 sec adjusted for site class effects as defined in Section 9.4.3
$\overline{s_u}$	=	average undrained shear strength in top (30 m); see Section 14.1.1, ASTM D2166-91 or ASTM D2850-87
s_h	=	spacing of special lateral reinforcement (mm)
T	=	the fundamental period of the building as determined in Section 10.9.3
T_a	=	the approximate fundamental period of the building as determined in Section 10.9.3.2
T_m	=	the modal period of vibration (sec) of the mth mode of the building as determined in Section 10.10.5
T_p	=	the fundamental period of the component and its attachment, Section 12.3.3
T_o	=	$0.2S_{D1}/S_{DS}$
T_s	=	S_{D1}/S_{DS}
V	=	the total design lateral force or shear at the base
V_t	=	the design value of the seismic base shear as determined in Section 10.10.8
V_x	=	the seismic design shear in story x as determined in Section 10.9.5
$\overline{V_s}$	=	average shear wave velocity in top (30 m), see Section 14.1.1
$\overline{V_{so}}$	=	the average shear wave velocity for the soils beneath the foundation at small strain levels, (m/s)
W	=	the total gravity load of the building. For calculation of seismic-isolated building period, W is the total seismic dead load weight of the building
W_c	=	the gravity load of a component of the building
W_m	=	the effective modal gravity load determined in accordance with Eq. 10.10.5-2
W_p	=	component operating weight (N)
w	=	the width of a diaphragm or shear wall in the direction of application of force. For sheathed diaphragms, the width shall be defined as the dimension between the outside faces of the tension and compression chords
w	=	moisture content (in percent), ASTM D2216-92 [3]
w_i, w_n, w_x	=	the portion of W that is located at or assigned to Level *i, n,* or x, respectively
x	=	the level under consideration
x	=	1 designates the first level above the base
y	=	elevations difference between points of attachment
z	=	the level under consideration; z = 1 designates the first level above the base

β	=	ratio of shear demand to shear capacity for the story between Level x and x - 1
γ	=	the average unit weight of soil (kg/m^3)
Δ	=	the design story drift as determined in Section 10.9.7.1
Δ_a	=	the allowable story drift as specified in Section 10.12
δ_{max}	=	the maximum displacement at Level x, considering torsion, Section 10.9.5.2
δ_{avg}	=	the average of the displacements at the extreme points of the structure at Level x, Section 10.9.5.2
θ	=	the stability coefficient for P-delta effects as determined in Section 10.9.7.2
ρ	=	a reliability coefficient based on the extent of structural redundance present in a building
ϕ	=	the strength reduction factor or resistance factor
ϕ_{im}	=	the displacement amplitude at the ith level of the building for the fixed-base condition when vibrating in its mth mode, Section 10.10.5
Ω_o	=	over strength factor as defined in Table 10.2

SECTION 9.4
SEISMIC GROUND MOTION VALUES

9.4.1 **Mapped Acceleration Parameters.** The Kingdom of Saudi Arabia has been divided into seven regions for determining the maximum considered earthquake ground motion as shown in Figure 9.4.1(a). The parameter S_S shall be determined from the 0.2 second spectral response accelerations shown on Figures 9.4.1(b) through 9.4.1(i). The parameter S_1 shall be determined from the 1.0 second spectral response accelerations shown on Figures 9.4.1(j) through 9.4.1(q). 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 Seismic Design Category A.

9.4.2 **Site Class.** Based on the site soil properties, the site shall be classified as Site Class A, B, C, D, E or F in accordance with Chapter 14. Where the soil properties are not known in sufficient detail to determine the Site Class, Site Class D or E shall be used, as per Section 14.1.

9.4.3 **Site Coefficients and Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameters.** The maximum considered earthquake spectral response acceleration for short periods (S_{MS}) and at 1-sec (S_{M1}), adjusted for site class effects, shall be determined by Eqs. 9.4.3-1 and 9.4.3-2, respectively.

$$S_{MS} = F_a S_S \qquad \textbf{(Eq. 9.4.3-1)}$$

$$S_{M1} = F_v S_1 \qquad \textbf{(Eq. 9.4.3-2)}$$

where

S_1 = the mapped maximum considered earthquake spectral response acceleration at a period of 1-sec as determined in accordance with Section 9.4.1

S_S = the mapped maximum considered earthquake spectral response acceleration at short periods as determined in accordance with Section 9.4.1 where site coefficients F_a and F_v are defined in Table 9.4.3a and Table 9.4.3b, respectively.

9.4.4 **Design Spectral Response Acceleration Parameters.** Design earthquake spectral response acceleration at short periods, S_{DS}, and at 1-sec period, S_{D1}, shall be determined from Eqs. 9.4.4-1 and 9.4.4-2, respectively.

$$S_{DS} = \frac{2}{3} S_{MS} \qquad \textbf{(Eq. 9.4.4-1)}$$

$$S_{D1} = \frac{2}{3} S_{M1} \qquad \textbf{(Eq. 9.4.4-2)}$$

TABLE 9.4.3a:
VALUES OF F_a AS A FUNCTION OF SITE CLASS AND MAPPED SHORT PERIOD MAXIMUM CONSIDERED EARTHQUAKE SPECTRAL ACCELERATION

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration at Short Periods				
	$S_S \leq 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	a	a	a	a	a

Note: Use straight-line interpolation for intermediate values of S_S.

a Site-specific geotechnical investigation and dynamic site response analyses shall be performed except that for structures with periods of vibration equal to or less than 0.5-seconds, values of F_a for liquefiable soils may be assumed equal to the values for the site class determined without regard to liquefaction in Step 3 of Section 14.1.2.

TABLE 9.4.3b:
VALUES OF F_v AS A FUNCTION OF SITE CLASS AND MAPPED 1-SECOND PERIOD MAXIMUM CONSIDERED EARTHQUAKE SPECTRAL ACCELERATION

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration at 1-Second Periods				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
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	a	a	a	a	a

Note: Use straight-line interpolation for intermediate values of S_1.

a Site-specific geotechnical investigation and dynamic site response analyses shall be performed except that for structures with periods of vibration equal to or less than 0.5-seconds, values of F_v for liquefiable soils may be assumed equal to the values for the site class determined without regard to liquefaction in Step 3 of Section 14.1.2.

9.4.5 **Design Response Spectrum.** Where a design response spectrum is required by these provisions, the design response spectrum curve shall be developed as indicated in Figure 9.4.5 and as follows:

1. For periods less than or equal to T_o, the design spectral response acceleration, S_a, shall be taken as given by Eq. 9.4.5-1:

$$S_a = S_{DS}\,(0.4 + 0.6\,\frac{T}{T_o}) \qquad \textbf{(Eq. 9.4.5-1)}$$

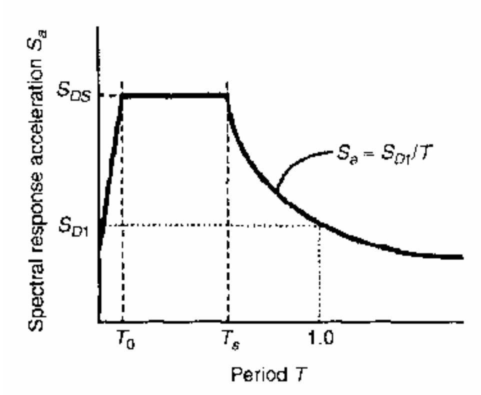


FIGURE 9.4.5
DESIGN RESPONSE SPECTRUM

2. For periods greater than or equal to T_o and less than or equal to T_s, the design spectral response acceleration, S_a, shall be taken as equal to S_{DS}.

3. For periods greater than T_s, the design spectral response acceleration, S_a, shall be taken as given by Eq. 9.4.5-2:

$$S_a = \frac{S_{D1}}{T} \qquad \textbf{(Eq. 9.4.5-2)}$$

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

S_{D1} = the design spectral response acceleration at 1-sec period, in units of g-sec

T = the fundamental period of the structure (sec)

T_o = 0.2 S_{D1}/S_{DS} and

T_S = S_{D1}/S_{DS}

SECTION 9.5
OCCUPANCY IMPORTANCE FACTOR

An occupancy importance factor, I, shall be assigned to each structure in accordance with Table 9.5. The Occupancy Category shall be determined from Table 1.6-1.

TABLE 9.5: OCCUPANCY IMPORTANCE FACTORS

Occupancy Category	I
I or II	1.0
III	1.25
IV	1.5

SECTION 9.6
SEISMIC DESIGN CATEGORY

Structures shall be assigned a Seismic Design Category in accordance with Section 9.6.1.

9.6.1 **Determination of Seismic Design Category.** All structures shall be assigned to a Seismic Design Category based on their Occupancy Category and the design spectral response acceleration coefficients, S_{DS} and S_{D1}, determined in accordance with Section 9.4.4. Each building and structure shall be assigned to the most severe Seismic Design Category in accordance with Table 9.6.a or 9.6.b, irrespective of the fundamental period of vibration of the structure, T.

TABLE 9.6. a: SEISMIC DESIGN CATEGORY BASED ON SHORT PERIOD RESPONSE ACCELERATIONS

Value of S_{DS}	Occupancy Category		
	I-II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$S_{DS} \ge 0.50g$	D	D	D

TABLE 9.6. b: SEISMIC DESIGN CATEGORY BASED ON 1-SECOND PERIOD RESPONSE ACCELERATIONS

Value of S_{D1}	Occupancy Category		
	I-II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$S_{D1} \ge 0.20g$	D	D	D

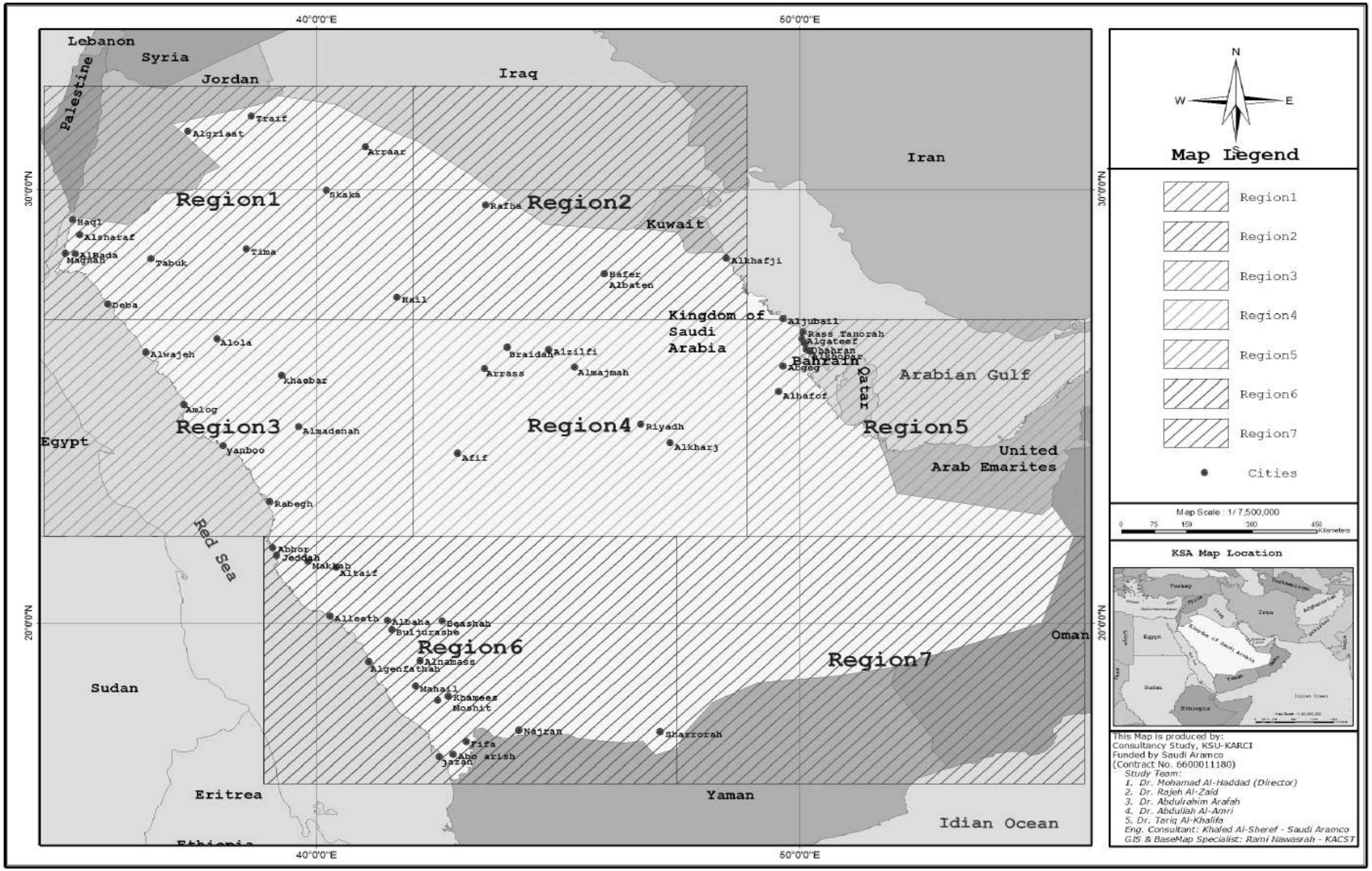


Figure 9.4.1(a): Regions for Determination of the Maximum Considered Earthquake Ground Motion in the Kingdom of Saudi Arabia.

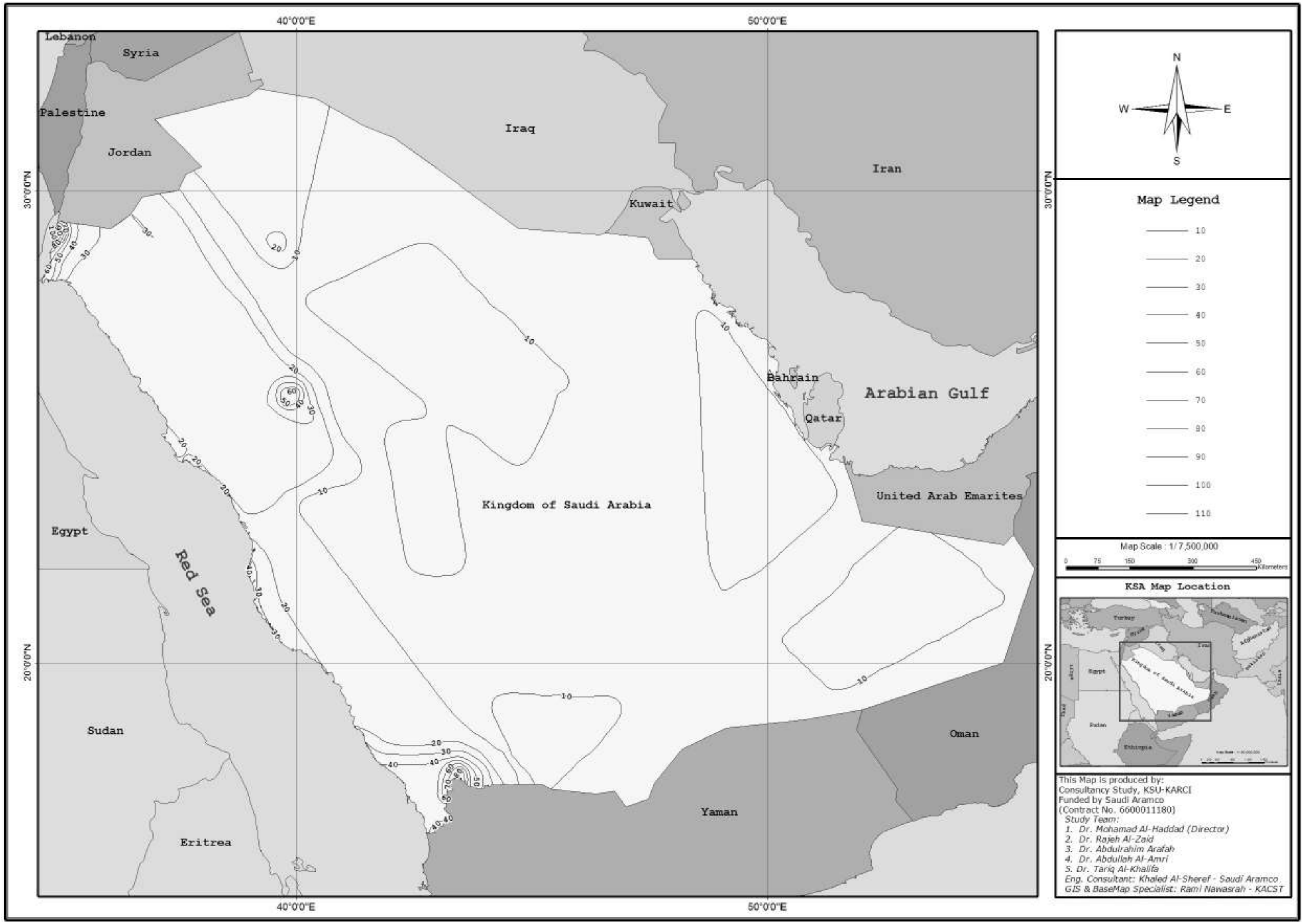


Figure 9.4.1(b): Maximum Considered Earthquake Ground Motion for the Kingdom of 0.2 SEC Spectral Response Acceleration (S_s in %g) (5 Percent of Critical Damping), Site Class B.

(All Regions)

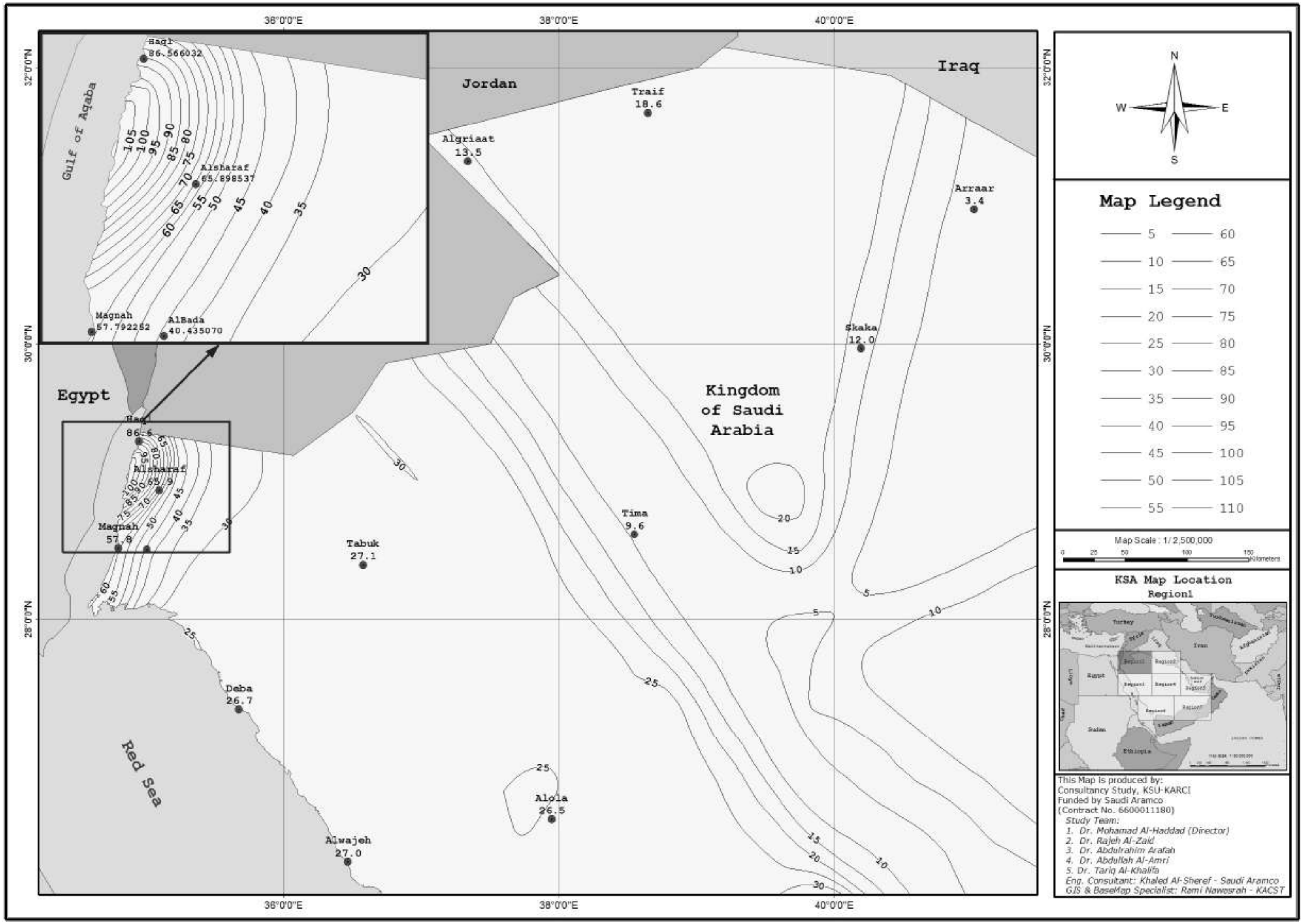


Figure 9.4.1(c): Maximum Considered Earthquake Ground Motion for the Kingdom of 0.2 SEC Spectral Response Acceleration (S_s in %g) (5 Percent of Critical Damping), Site Class B.

(Region 1)

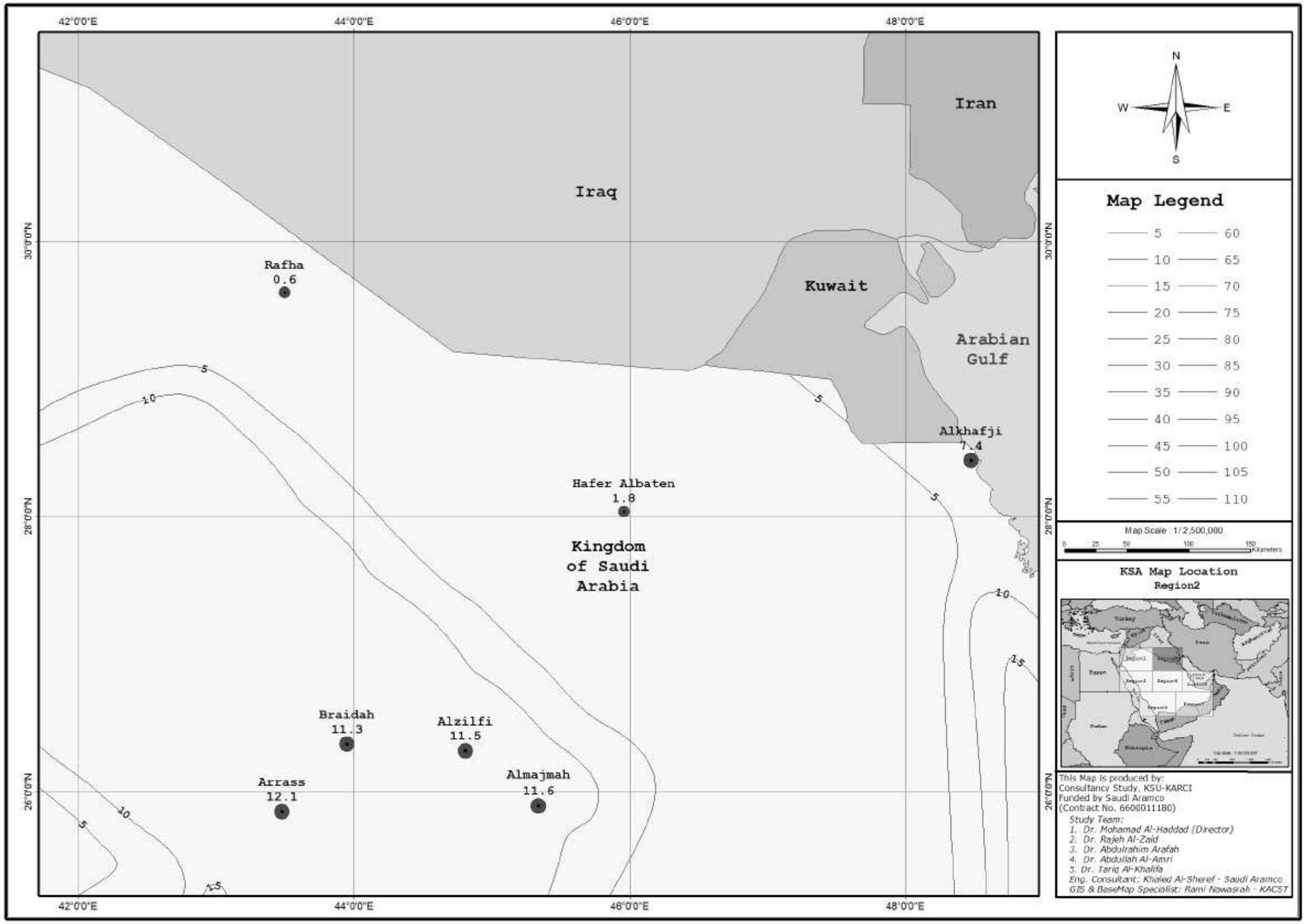


Figure 9.4.1(d): Maximum Considered Earthquake Ground Motion for the Kingdom of 0.2 SEC Spectral Response Acceleration (S_s in%g) (5 Percent of Critical Damping), Site Class B.

(Region 2)

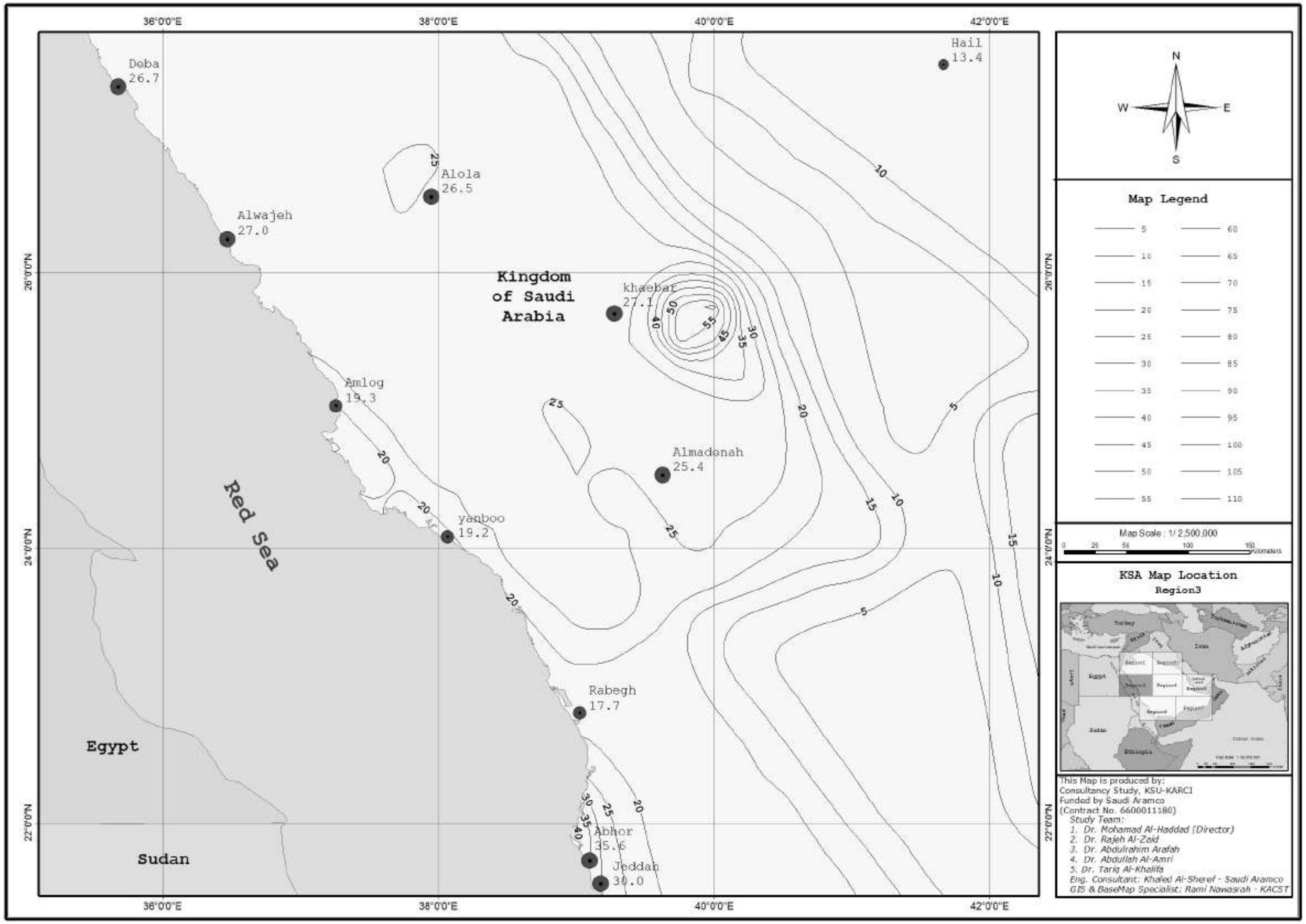


Figure 9.4.1(e): Maximum Considered Earthquake Ground Motion for the Kingdom of 0.2 SEC Spectral Response Acceleration (S_s in %g) (5 Percent of Critical Damping), Site Class B.

(Region 3)

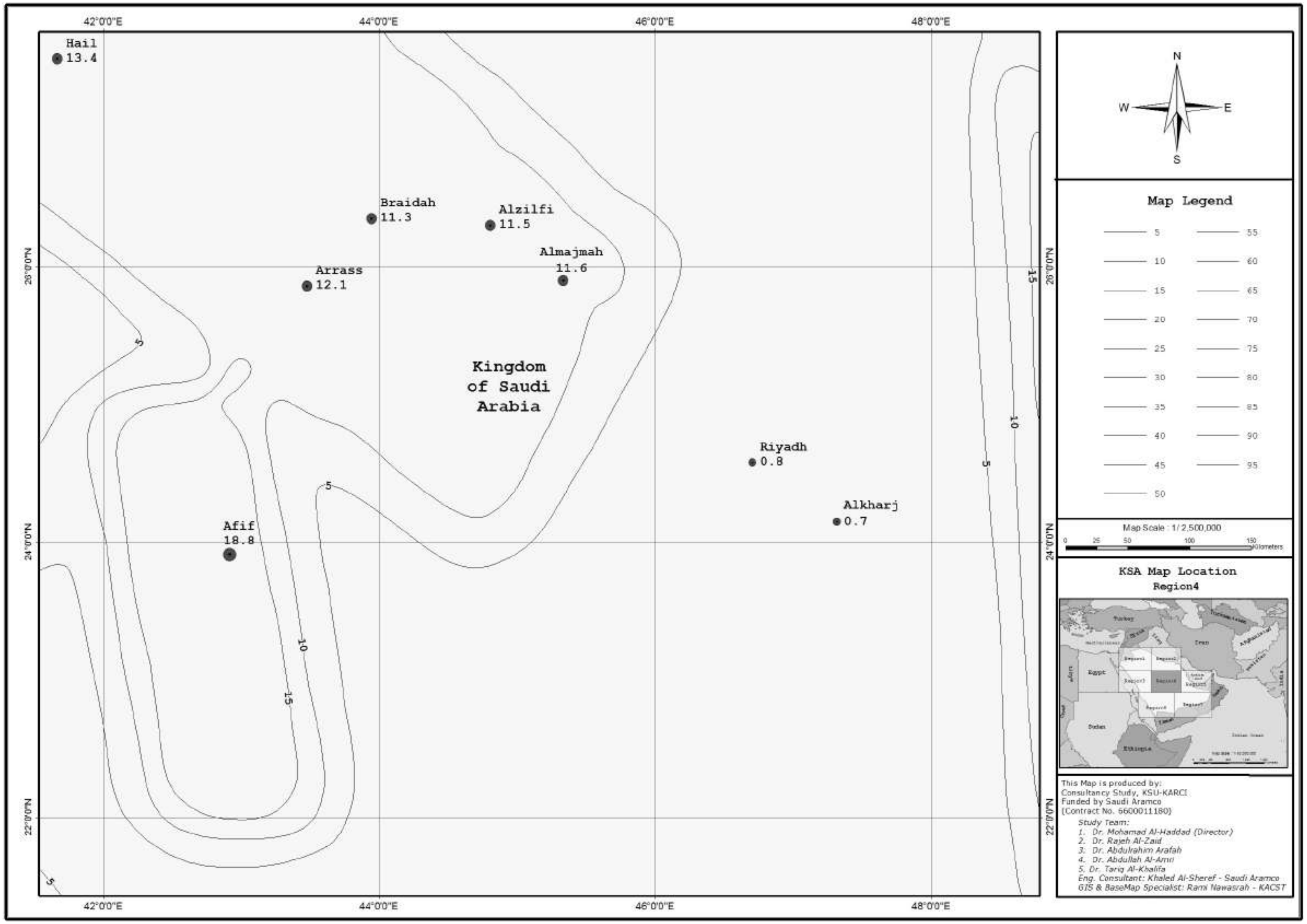


Figure 9.4.1(f): Maximum Considered Earthquake Ground Motion for the Kingdom of 0.2 SEC Spectral Response Acceleration (S_s in %g) (5 Percent of Critical Damping), Site Class B.

(Region 4)

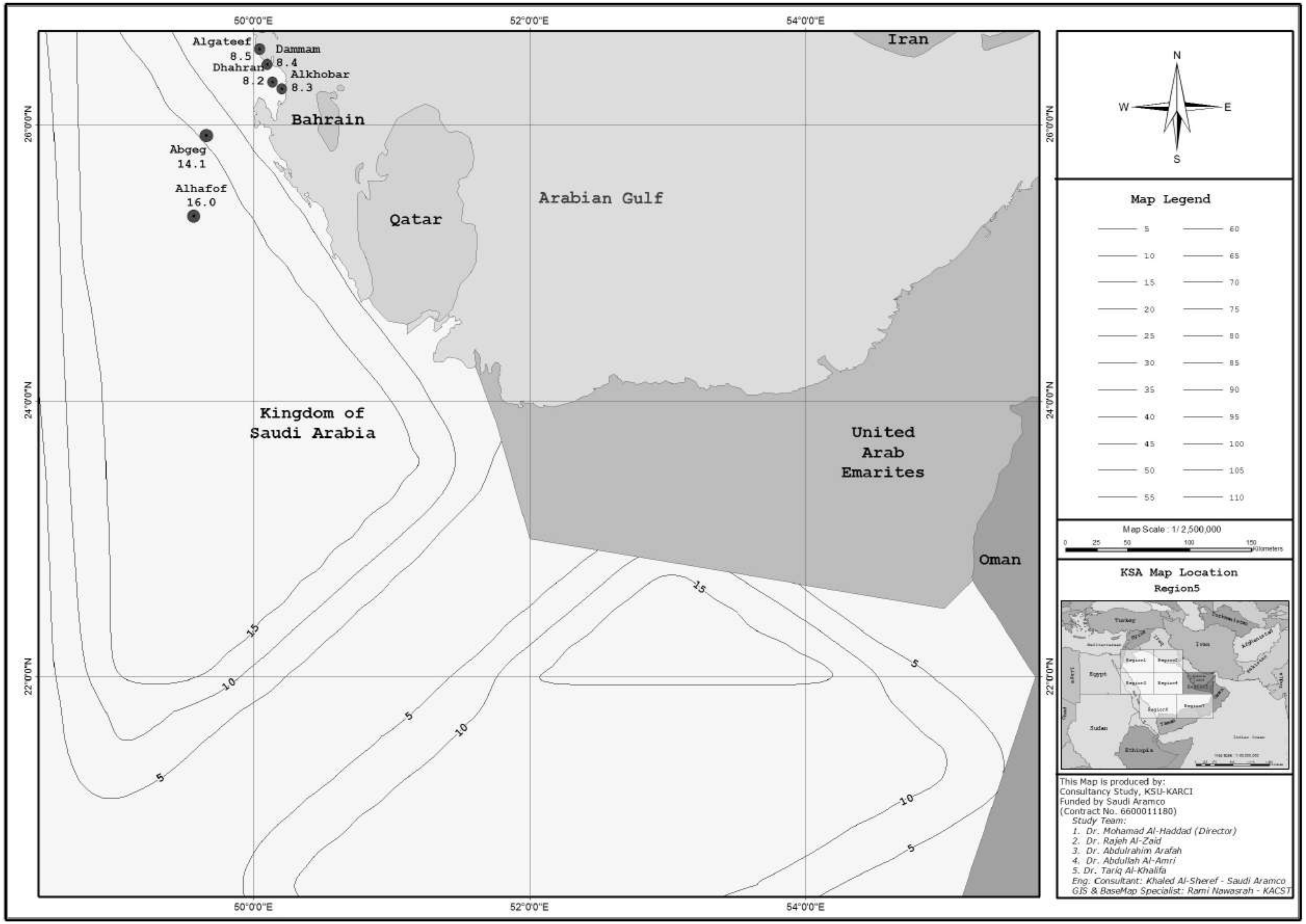


Figure 9.4.1(g): Maximum Considered Earthquake Ground Motion for the Kingdom of 0.2 SEC Spectral Response Acceleration (S_s in%g) (5 Percent of Critical Damping), Site Class B.

(Region 5)

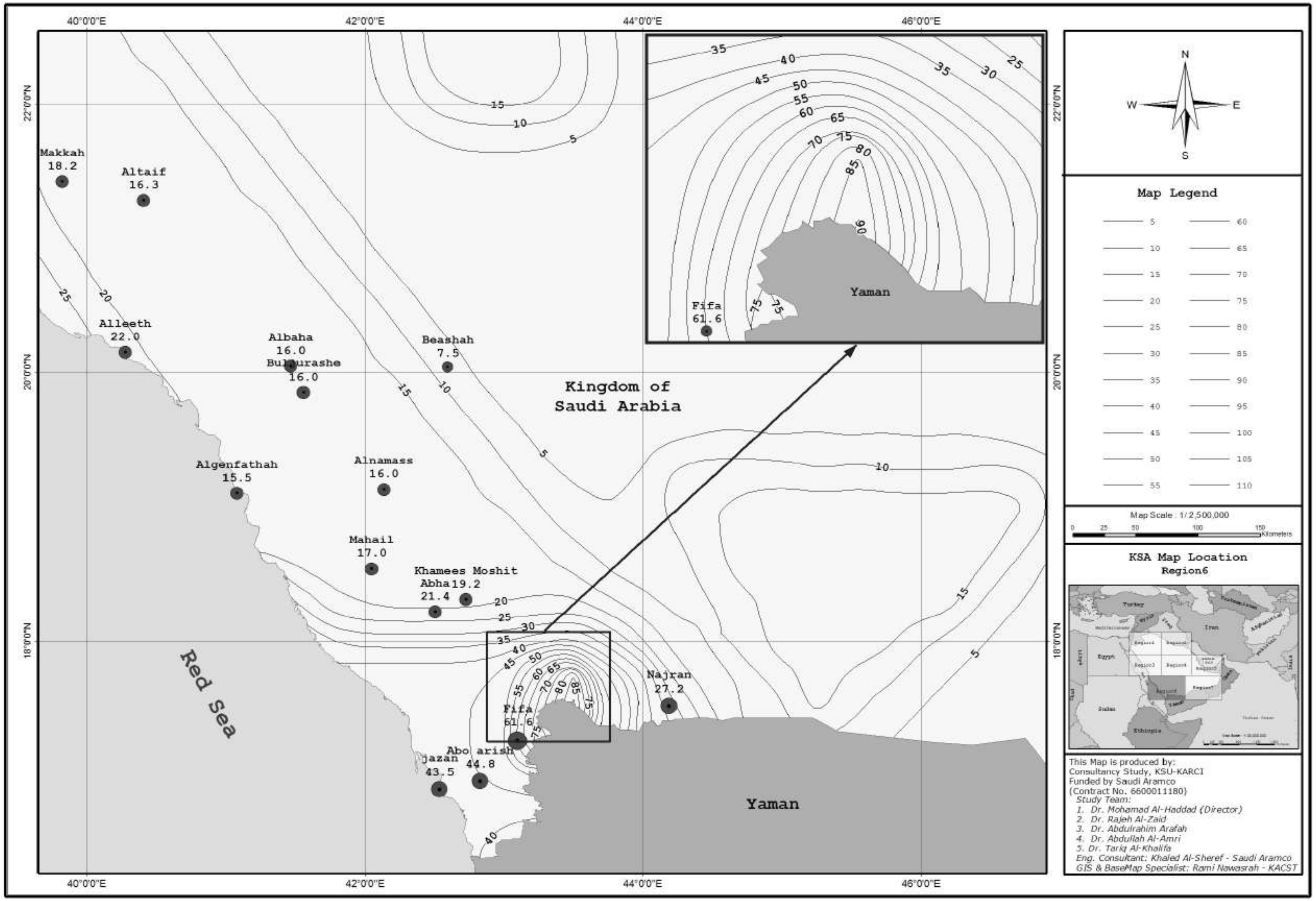


Figure 9.4.1(h): Maximum Considered Earthquake Ground Motion for the Kingdom of 0.2 SEC Spectral Response Acceleration (S_s in %g) (5 Percent of Critical Damping), Site Class B.

(Region 6)

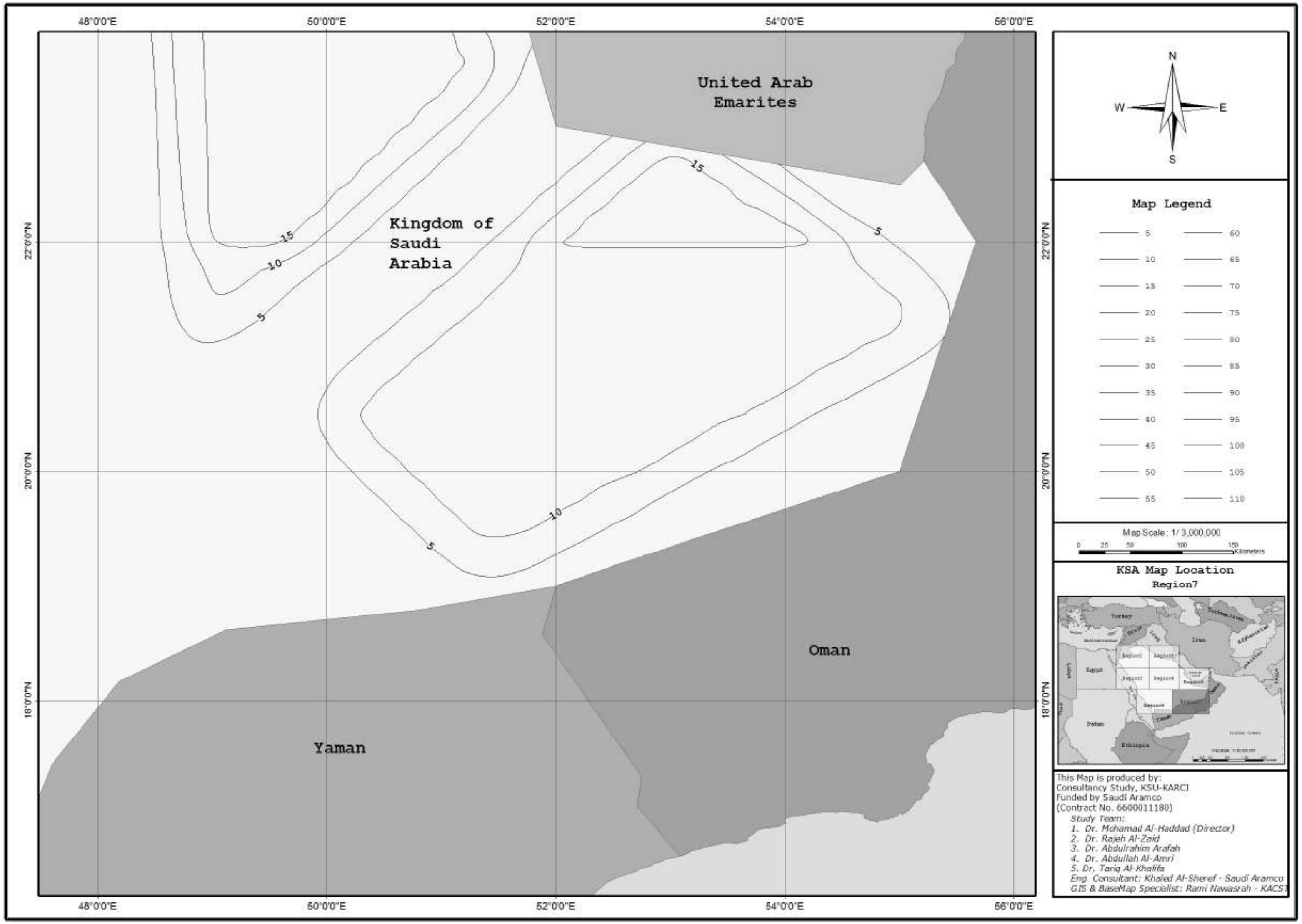


Figure 9.4.1(i): Maximum Considered Earthquake Ground Motion for the Kingdom of 0.2 SEC Spectral Response Acceleration (S_s in %g) (5 Percent of Critical Damping), Site Class B.

(Region 7)

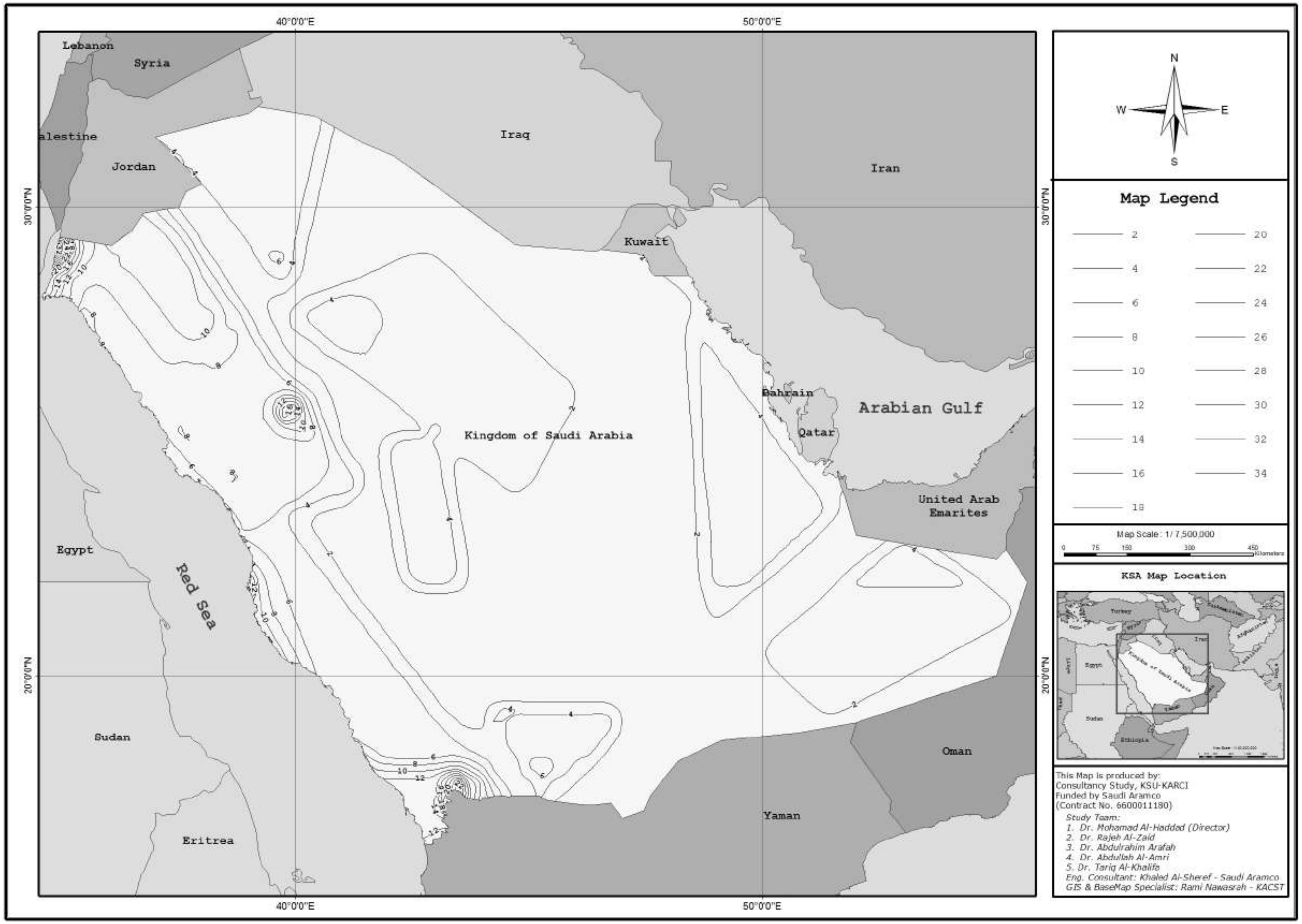


Figure 9.4.1(j): Maximum Considered Earthquake Ground Motion for the Kingdom of 1 SEC Spectral Response Acceleration (S_1 in %g) (5 Percent of Critical Damping), Site Class B.
(All Regions)

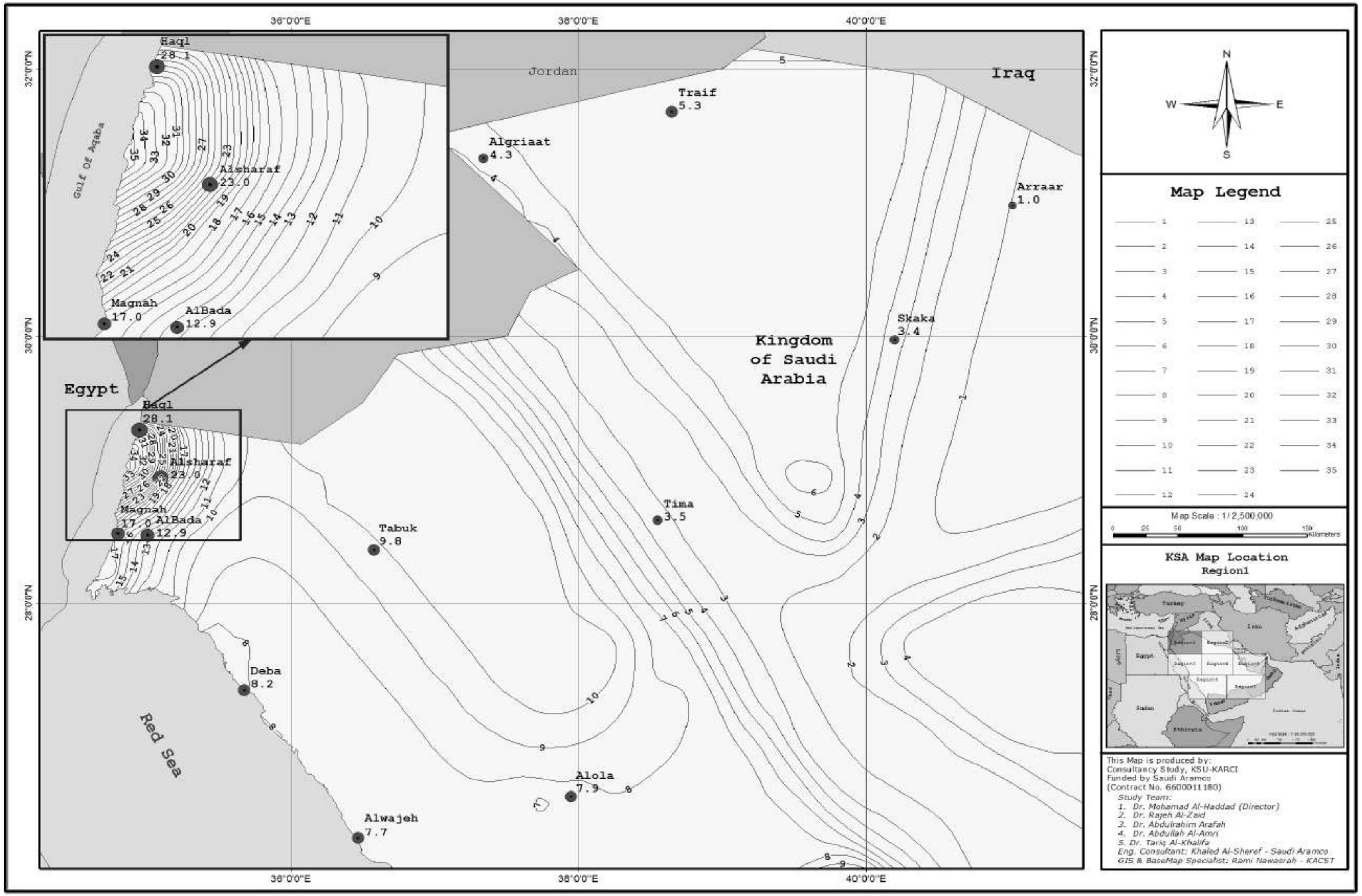


Figure 9.4.1(k): Maximum Considered Earthquake Ground Motion for the Kingdom of 1 SEC Spectral Response Acceleration (S_1 in %g) (5 Percent of Critical Damping), Site Class B.

(Region 1)

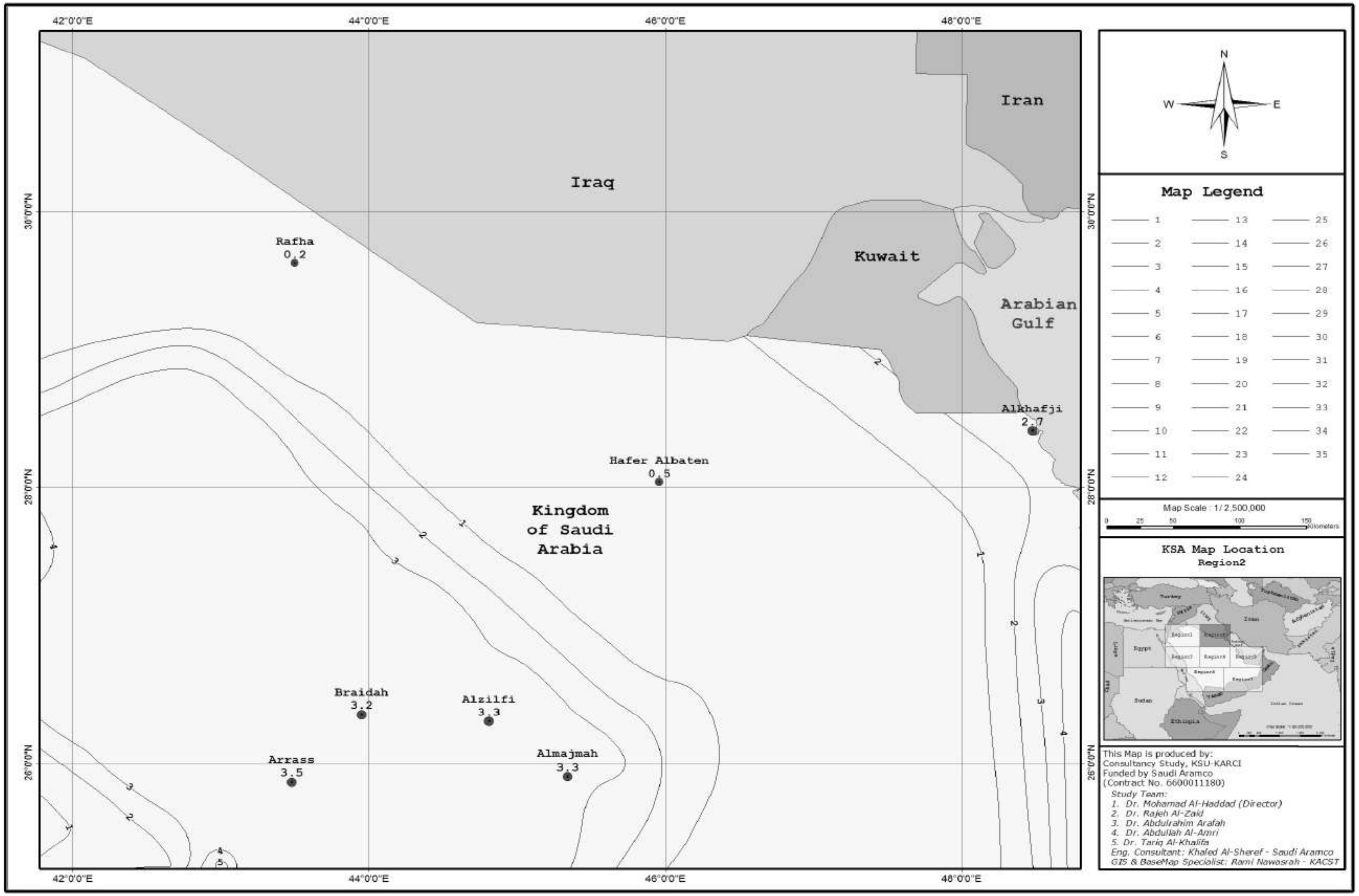


Figure 9.4.1(l): Maximum Considered Earthquake Ground Motion for the Kingdom of 1 SEC Spectral Response Acceleration (S_1 in % g) (5 Percent of Critical Damping), Site Class B.

(Region 2)

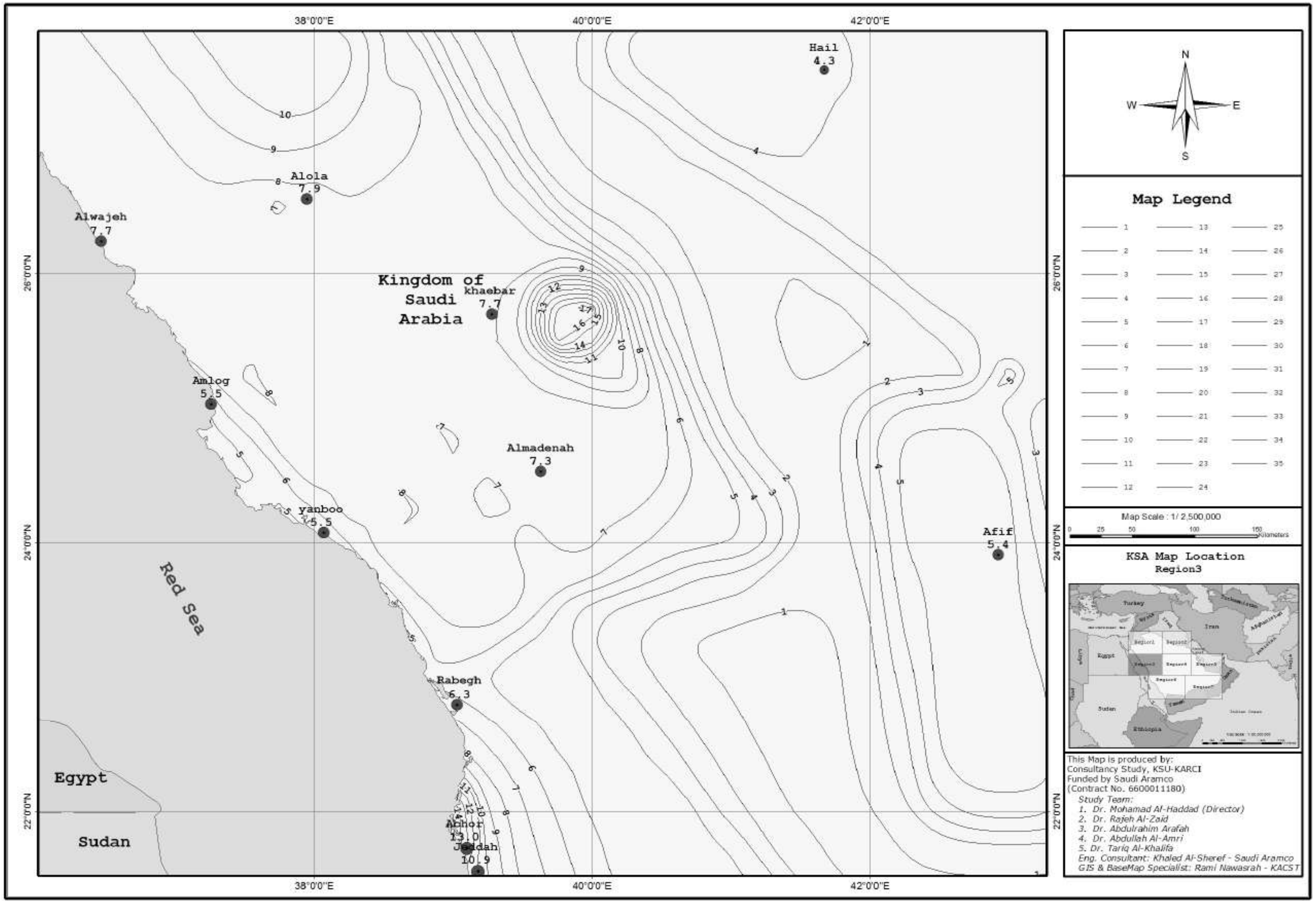


Figure 9.4.1(m): Maximum Considered Earthquake Ground Motion for the Kingdom of 1 SEC Spectral Response Acceleration (S_1 in %g) (5 Percent of Critical Damping), Site Class B.
(Region 3)

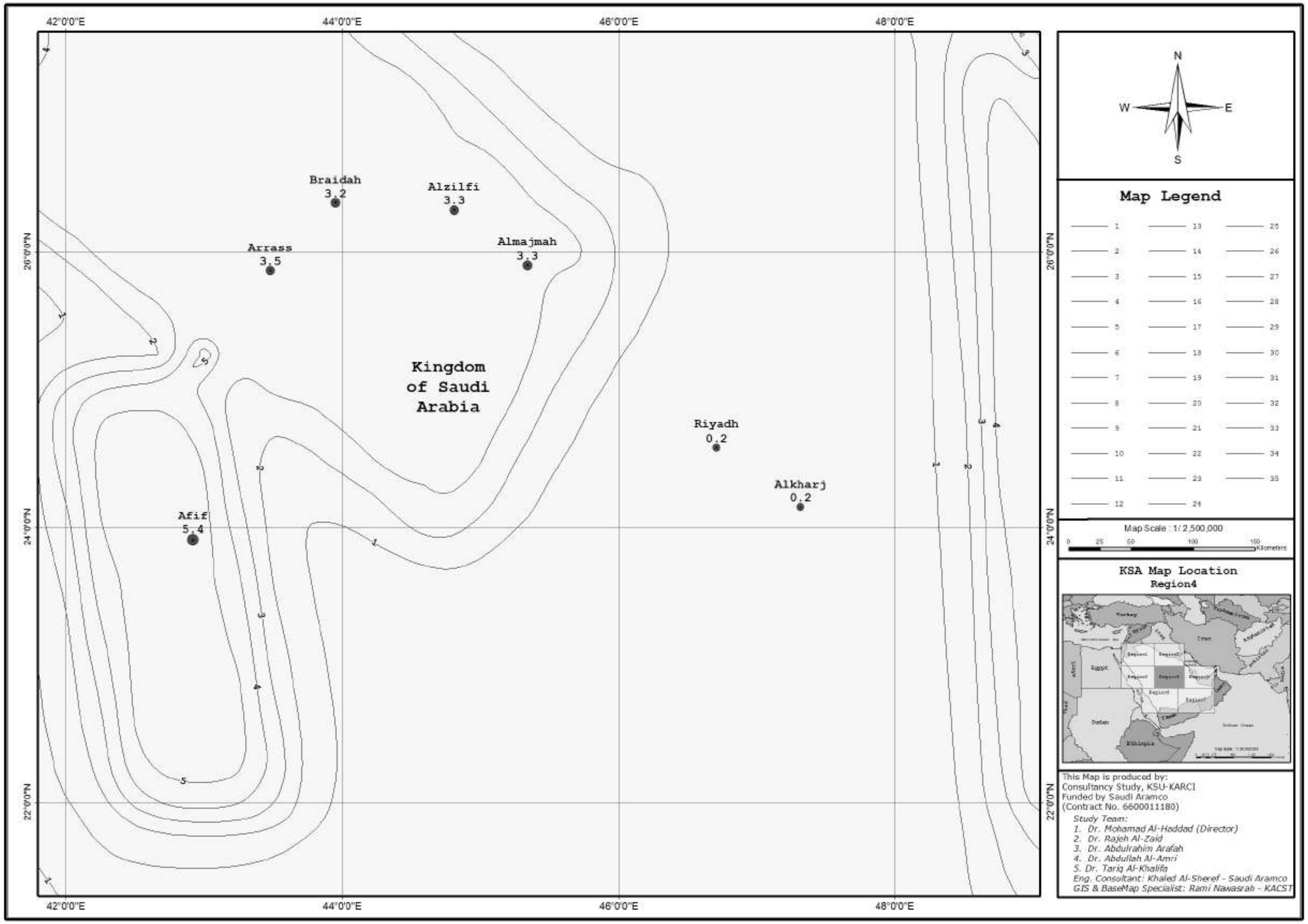


Figure 9.4.1(n): Maximum Considered Earthquake Ground Motion for the Kingdom of 1 SEC Spectral Response Acceleration (S_1 in %g) (5 Percent of Critical Damping), Site Class B.

(Region 4)

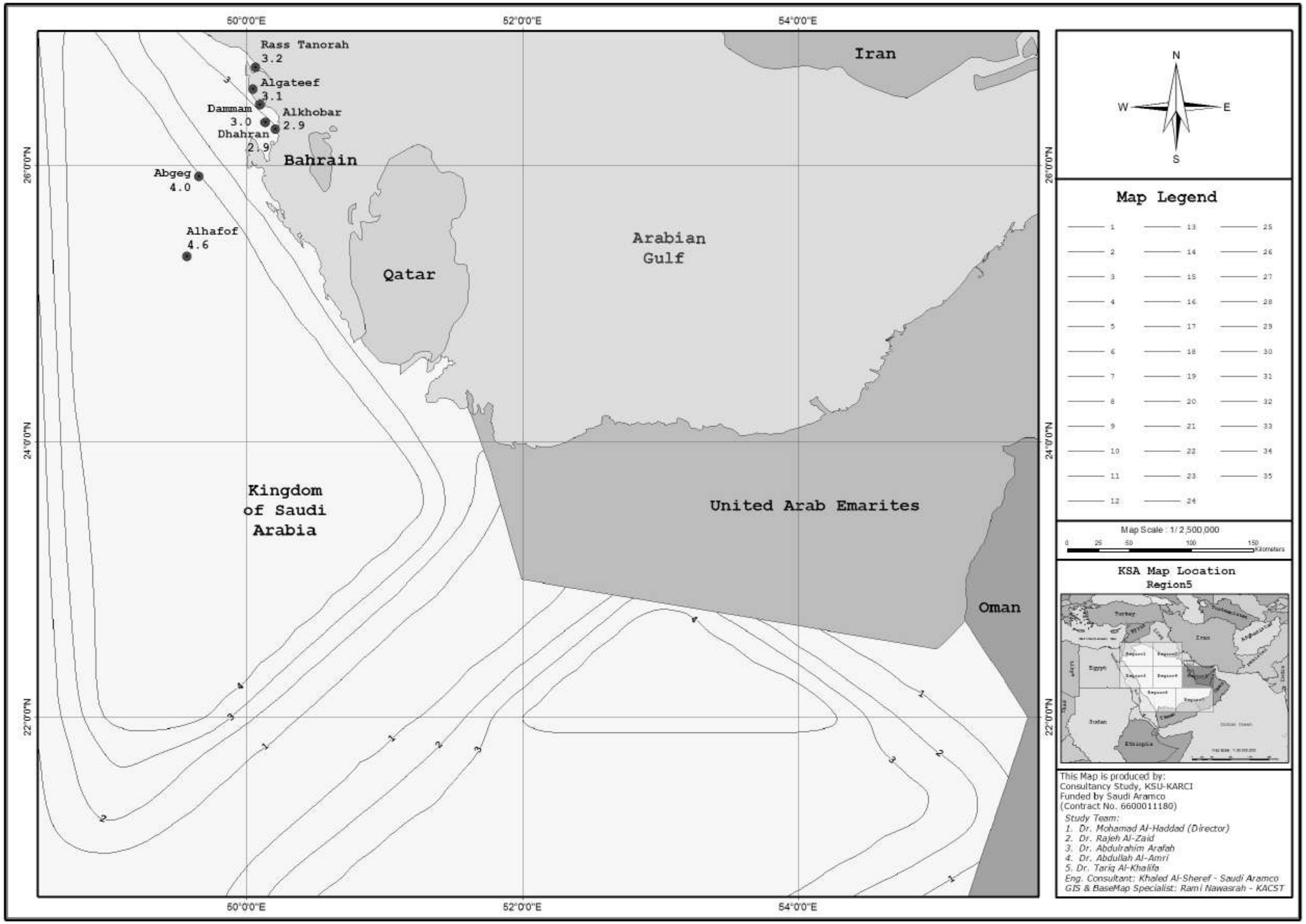


Figure 9.4.1(o): Maximum Considered Earthquake Ground Motion for the Kingdom of 1 SEC Spectral Response Acceleration (S_1 in %g) (5 Percent of Critical Damping), Site Class B.

(Region 5)

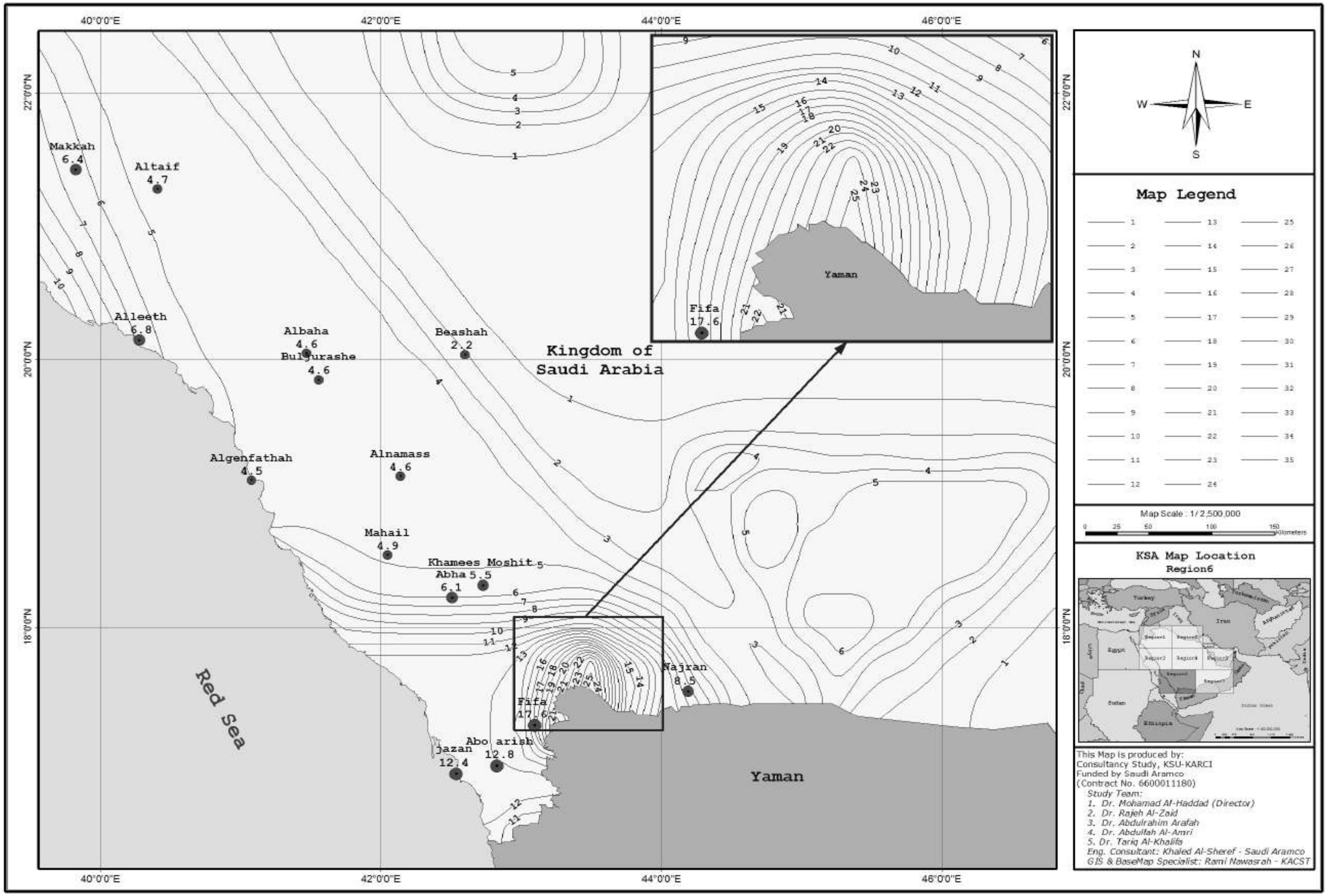


Figure 9.4.1(p): Maximum Considered Earthquake Ground Motion for the Kingdom of 1 SEC Spectral Response Acceleration (S_1 in %g) (5 Percent of Critical Damping), Site Class B.

(Region 6)

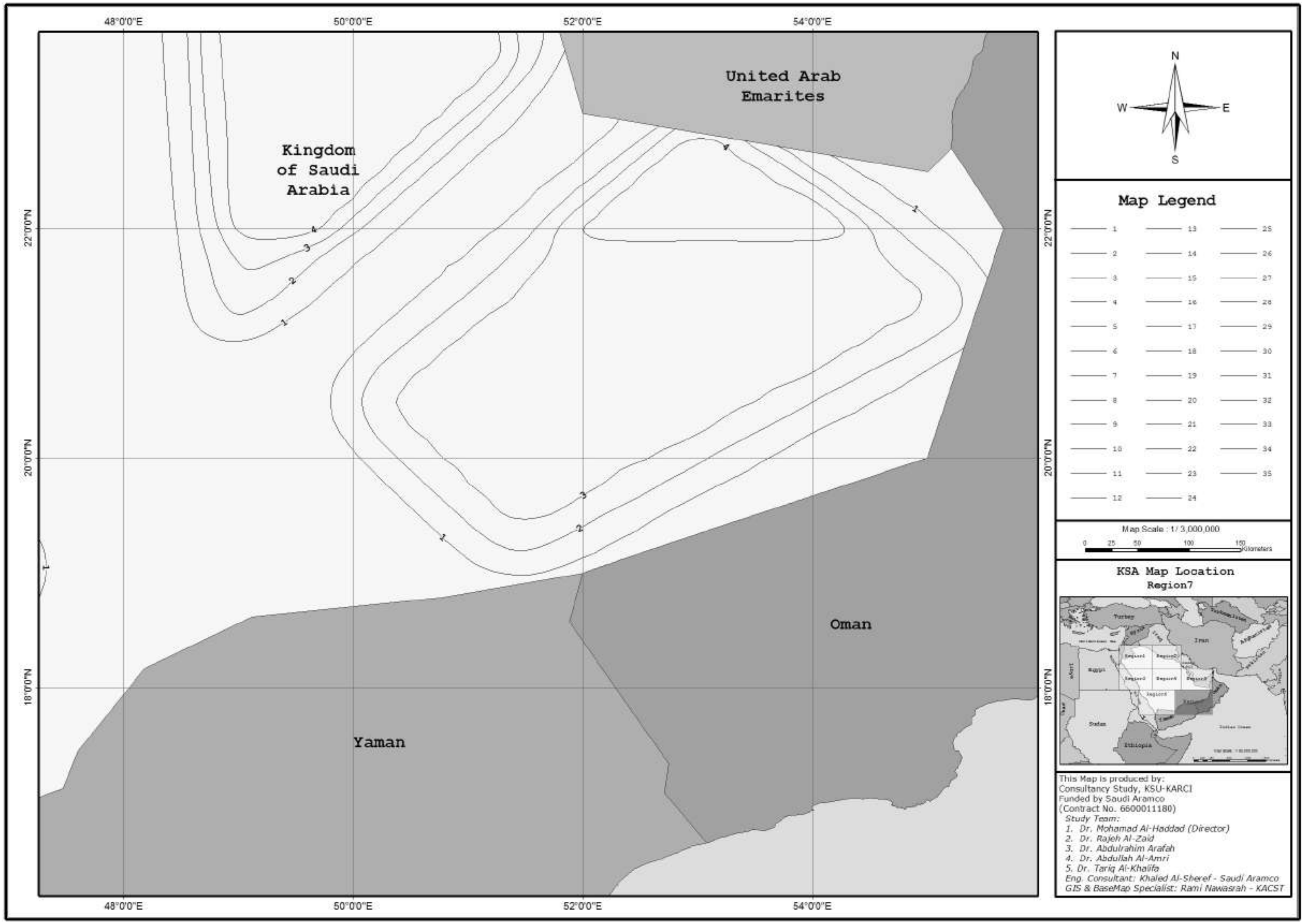


Figure 9.4.1(q): Maximum Considered Earthquake Ground Motion for the Kingdom of 1 SEC Spectral Response Acceleration ($S_{1\ in}$ %g) (5 Percent of Critical Damping), Site Class B.

(Region 7)

CHAPTER 10
SEISMIC DESIGN REQUIREMENTS FOR BUILDING STRUCTURES

SECTION 10.1
STRUCTURAL DESIGN BASIS

10.1.1 **Basic Requirements.** The seismic analysis and design procedures to be used in the design of structures and their components shall be as prescribed in this Section. The structure shall include complete lateral and vertical force-resisting systems capable of providing adequate strength, stiffness, and energy dissipation capacity to withstand the design ground motions within the prescribed limits of deformation and strength demand. The design ground motions shall be assumed to occur along any horizontal direction of a structure. The adequacy of the structural systems shall be demonstrated through the construction of a mathematical model and evaluation of this model for the effects of design ground motions. The design seismic forces, and their distribution over the height of the structure, shall be established in accordance with one of the applicable procedures indicated in Section 10.6 and the corresponding internal forces and deformations in the members of the structure shall be determined. An approved alternative procedure shall not be used to establish the seismic forces and their distribution unless the corresponding internal forces and deformations in the members are determined using a model consistent with the procedure adopted.

Individual members shall be provided with adequate strength to resist the shears, axial forces, and moments determined in accordance with these provisions, and connections shall develop the strength of the connected members or the forces indicated above. The deformation of the structure shall not exceed the prescribed limits when the structure is subjected to the design seismic forces.

A continuous load path, or paths, with adequate strength and stiffness shall be provided to transfer all forces from the point of application to the final point of resistance. The foundation shall be designed to resist the forces developed and accommodate the movements imparted to the structure by the design ground motions. The dynamic nature of the forces, the expected ground motion, and the design basis for strength and energy dissipation capacity of the structure shall be included in the determination of the foundation design criteria.

Allowable Stress Design is permitted to be used to evaluate sliding, overturning, and soil bearing at the soil-structure interface regardless of the design approach used in the design of the structure.

SECTION 10.2
STRUCTURAL SYSTEM SELECTION

The basic lateral and vertical seismic force-resisting system shall conform to one of the types indicated in Table 10.2. Each type is subdivided by the types of vertical element 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 10.2. The appropriate response modification coefficient, R, system overstrength factor, Ω_o, and the deflection amplification factor (C_d) indicated in Table 10.2 shall be used in determining the base shear, element design forces, and design story drift. Special framing requirements are indicated in Section 10.11 and Sections 11.1, 11.2, 11.3 and 11.4 for structures assigned to the various Seismic Design Categories.

Seismic force-resisting systems that are not contained in Table 10.2 shall be 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 10.2 for equivalent response modification coefficient, *R,* system overstrength coefficient, Ω_o, and deflection amplification factor, C_d, values.

10.2.1 **Dual System.** For a dual system, the moment frame shall be capable of resisting at least 25% of the design seismic forces. The total seismic-force resistance is to be provided by the combination of the moment frame and the shear walls or braced frames in proportion to their rigidities.

10.2.2 **Combinations of Framing Systems.** Different seismic force-resisting systems are permitted along the two orthogonal axes of the structure. Combinations of seismic force-resisting systems shall comply with the requirements of this Section.

10.2.2.1 **R and Ω_o Factors.** The response modification coefficient, *R,* in the direction under consideration at any story shall not exceed the lowest response modification coefficient, *R,* for the seismic force-resisting system in the same direction considered above that story excluding penthouses. For other than dual systems, where a combination of different structural systems is utilized to resist lateral forces in the same direction, the value of *R* used in that direction shall not be greater than the least value of any of the systems utilized in the same direction. If a system other than a dual system with a response modification coefficient, *R,* with a value of less than 5 is used as part of the seismic force-resisting system in any direction of the structure, the lowest such value shall be used for the entire structure. The system overstrength factor, Ω_o, in the direction under consideration at any story shall not be less than the largest value of this factor for the seismic force-resisting system in the same direction considered above that story.

Exceptions: The limit does not apply to supported structural systems with a weight equal to or less than 10% of the weight of the structure.

10.2.2.2 **Combination Framing Detailing Requirements.** The detailing requirements of Section 10.11 required by the higher response modification coefficient, *R*, shall be used for structural components common to systems having different response modification coefficients.

10.2.3 **Seismic Design Categories B and C. The** structural framing system for structures assigned to Seismic Design Categories B and C shall comply with the structure height and structural limitations in Table 10.2.

TABLE 10.2:
DESIGN COEFFICIENTS AND FACTORS FOR BASIC SEISMIC FORCE-RESISTING SYSTEMS

Basic Seismic Force-Resisting System	Response Modification Coefficient, R^a	System Over-strength Factor, Ω_o [f]	Deflection Amplification Factor, C_d [b]	Structural System Limitations and Building Height (m) Limitations[c] — Seismic Design Category A&B	C	D[d]
Bearing Wall Systems						
Special reinforced concrete shear walls	4	2.5	5	NL	NL	50
Ordinary reinforced concrete shear walls	3	2.5	4	NL	NL	NP
Special reinforced masonry shear walls	4	2.5	3.5	NL	NL	50
Intermediate reinforced masonry shear walls	2.5	2.5	2.25	NL	NL	NP
Ordinary reinforced masonry shear walls	1.5	2.5	1.75	NL	50	NP
Building Frame Systems						
Steel eccentrically braced frames, moment resisting connections at columns away from Links	7	2	4	NL	NL	50
Steel eccentrically braced frames, non-moment resisting connections at columns away from links	6	2	4	NL	NL	50
Special steel concentrically braced frames	5	2	5	NL	NL	50
Ordinary steel concentrically braced frames	4	2	4.5	NL	NL	10[i]
Special reinforced concrete shear walls	5	2.5	5	NL	NL	50
Ordinary reinforced concrete shear walls	4	2.5	4.5	NL	NL	NP
Composite eccentrically braced frames	7	2	4	NL	NL	50
Composite concentrically braced frames	4	2	4.5	NL	NL	50
Ordinary composite braced frames	2.5	2	3	NL	NL	NP
Composite steel plate shear walls	5	2.5	5.5	NL	NL	50
Special composite reinforced concrete shear walls with steel elements	5	2.5	5	NL	NL	50
Ordinary composite reinforced concrete shear walls with steel elements	4	2.5	4.25	NL	NL	NP
Special reinforced masonry shear walls	4	2.5	4	NL	NL	50
Intermediate reinforced masonry shear walls	3	2.5	4	NL	NL	NP
Ordinary reinforced masonry shear walls	2	2.5	2.25	NL	50	NP
Moment Resisting Frame Systems						
Special steel moment frames	7	3	5.5	NL	NL	NL
Intermediate steel moment frames	4	3	4	NL	NL	10g
Ordinary steel moment frames	3	3	3	NL	NL	NPg,h
Special reinforced concrete moment frames	6.5	3	5.5	NL	NL	NL
Intermediate reinforced concrete moment frames	4	3	4.5	NL	NL	NP
Ordinary reinforced concrete moment frames	2.5	3	2.5	NL	NP	NP

TABLE 10.2: DESIGN COEFFICIENTS AND FACTORS FOR BASIC SEISMIC FORCE-RESISTING SYSTEMS – continued

Basic Seismic Force-Resisting System	Response Modification Coefficient, R^a	System Over-strength Factor, Ω_o [f]	Deflection Amplification Factor, C_d[b]	Structural System Limitations and Building Height (m) Limitations[c]		
				Seismic Design Category		
				A&B	C	D[d]
Dual Systems with Special Moment Frames Capable of Resisting at Least 25% of Prescribed Seismic Forces						
Steel eccentrically braced frames, moment resisting connections, at columns away from links	7	2.5	4	NL	NL	NL
Steel eccentrically braced frames, non-moment resisting connections, at columns away from links	6	2.5	4	NL	NL	NL
Special steel concentrically braced frames	7	2.5	6.5	NL	NL	NL
Special reinforced concrete shear walls	6.5	2.5	6.5	NL	NL	NL
Ordinary reinforced concrete shear walls	5.5	2.5	6	NL	NL	NP
Composite eccentrically braced frames	6.5	2.5	4	NL	NL	NL
Composite concentrically braced frames	5	2.5	5	NL	NL	NL
Composite steel plate shear walls	6.5	2.5	6.5	NL	NL	NL
Special composite reinforced concrete shear walls with steel elements	6.5	2.5	6.5	NL	NL	NL
Ordinary composite reinforced concrete shear walls with steel elements	5.5	2.5	6	NL	NL	NP
Special reinforced masonry shear walls	5.5	3	6.5	NL	NL	NL
Intermediate reinforced masonry shear walls	4.5	2.5	5	NL	NL	NL
Ordinary steel concentrically braced frames	5	2.5	5	NL	NL	NL
Dual Systems with Intermediate Moment Frames Capable of Resisting at Least 25% of Prescribed Seismic Forces						
Special steel concentrically braced frames[e]	4	2.5	4.5	NL	NL	10
Special reinforced concrete shear walls	4.5	2.5	5	NL	NL	50
Ordinary reinforced masonry shear walls	2.5	3	2.5	NL	50	NP
Intermediate reinforced masonry shear walls	4	3	4.5	NL	NL	NP
Composite concentrically braced frames	4	2.5	4.5	NL	NL	50
Ordinary composite braced frames	3.5	2.5	3	NL	NL	NP
Ordinary composite reinforced concrete shear walls with steel elements	4	3	4.5	NL	NL	NP
Ordinary steel concentrically braced frames	4	2.5	4.5	NL	NL	50
Ordinary reinforced concrete shear walls	4.5	2.5	4.5	NL	NL	NP

TABLE 10.2: DESIGN COEFFICIENTS AND FACTORS FOR BASIC SEISMIC FORCE-RESISTING SYSTEMS – continued

Basic Seismic Force-Resisting System	Response Modification Coefficient, R[a]	System Over-strength Factor, Ω_o [f]	Deflection Amplification Factor, C_d[b]	Structural System Limitations and Building Height (m) Limitations[c] — Seismic Design Category		
				A&B	C	D[d]
Inverted Pendulum Systems and Cantilevered Column Systems						
Special steel moment frames	2	2	2.5	NL	NL	NL
Ordinary steel moment frames	1	2	2.5	NL	NL	NP
Special reinforced concrete moment frames	2	2	1.25	NL	NL	NL
Structural Steel Systems Not Specifically Detailed for Seismic Resistance	2.5	3	3	NL	NL	NP

[a] Response modification coefficient, *R,* for use throughout the code. Note *R* reduces forces to a strength level, not an allowable stress level. The given values are approximate and require further study.

[b] Deflection amplification factor, C_d, for use in Sections 10.9.7.1 and 10.9.7.2

[c] NL = Not Limited and NP = Not Permitted. Heights are measured from the base of the structure as defined in Section 9.2.

[d] See Section 10.2.4.1 for a description of building systems limited to buildings with a height of 75 m or less.

[e] Ordinary moment frame is permitted to be used in lieu of intermediate moment frame in Seismic Design Categories B and C.

[f] The tabulated value of the overstrength factor, Ω_o may be reduced by subtracting 0.5 for structures with flexible diaphragms but shall not be taken as less than 2.0 for any structure.

[g] Steel ordinary moment frames and intermediate moment frames are permitted in single-story buildings up to a height of 18 m, when the moment joints of field connections are constructed of bolted end plates and the dead load of the roof does not exceed 0.75 kPa.

[h] Steel ordinary moment frames are permitted in buildings up to a height of 10 m where the dead load of the walls, floors, and roofs does not exceed 0.75 kPa.

[i] Steel ordinary concentrically braced frames are permitted in single-story buildings up to a height of 18 m when the dead load of the roof does not exceed 0.75 kPa and in penthouse structures.

10.2.4 Seismic Design Category D. The structural framing system for a structure assigned to Seismic Design Category D shall comply with Section 10.2.3 and the additional provisions of this Section.

10.2.4.1 Interaction Effects. Moment resisting frames that are enclosed or adjoined by more rigid elements 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 structure deformations corresponding to the design story drift (Δ) as determined in Section 10.9.7. In addition, the effects of these elements shall be considered when determining whether a structure has one or more of the irregularities defined in Section 10.3.

10.2.4.2 Deformational Compatibility. Every structural component not included in the seismic force-resisting system in the direction under consideration shall be designed to be adequate for the vertical load-carrying capacity and the induced moments and shears resulting from the design story drift (Δ) as determined in accordance with Section 10.9.7; see also Section 10.12.

Exception: Reinforced concrete frame members not designed as part of the seismic force-resisting system shall comply with Section 21.9 of SBC 304.

When 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.

10.2.4.3 **Special Moment Frames**. A special moment frame that is used but not required by Table 10.2 shall not be discontinued and supported by a more rigid system with a lower response modification coefficient (R) unless the requirements of Sections 10.11.2.4 and 10.11.4.2 are met. Where a special moment frame is required by Table 10.2, the frame shall be continuous to the foundation.

SECTION 10.3
DIAPHRAGM FLEXIBILITY, CONFIGURATION IRREGULARITIES AND REDUNDANCY

10.3.1 **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 10.3.1.1, 10.3.1.2 or 10.3.1.3, the structural analysis shall explicitly include consideration of the stiffness of the diaphragm (i.e. semi-rigid modeling assumption).

10.3.1.1 **Flexible Diaphragm Condition.** Diaphragms constructed of untopped steel decking shall be permitted to be idealized as flexible in structures in which the vertical elements are steel or composite braced frames, or concrete, masonry, steel or composite shear walls. Diaphragms of untopped steel decks in one- and two-family residential buildings shall also be permitted to be idealized as flexible.

10.3.1.2 **Rigid Diaphragm Condition.** Diaphragms of concrete slabs or concrete filled metal deck with span-to-depth ratios of 3 or less in structures that have no horizontal irregularities shall be permitted to be idealized as rigid.

10.3.1.3 **Calculated Flexible Diaphragm Condition.** Diaphragms not satisfying the conditions of Sections 10.3.1.1 or 10.3.1.2 shall be permitted to be idealized as flexible when the computed maximum in-plane deflection of the diaphragm itself under lateral load is more than two times the average story drift of adjoining vertical elements of the seismic force-resisting system of the associated story under equivalent tributary lateral load as shown in Figure 10.3-1. The loadings used for this calculation shall be those prescribed by Section 10.9.

10.3.2 **Irregular and Regular Classification.** Structures shall be classified as regular or irregular based on the criteria in this Section. Such classification shall be based on the plan and vertical configuration.

10.3.2.1 **Plan Irregularity.** Structures having one or more of the irregularity types listed in Table 10.3.2.1 shall be designated as having plan structural irregularity. Such structures assigned to the Seismic Design Categories listed in Table 10.3.2.1 shall comply with the requirements in the sections referenced in that table.

10.3.2.2 **Vertical Irregularity.** Structures having one or more of the irregularity types listed in Table 10.3.2.2 shall be designated as having vertical irregularity. Such structures assigned to the Seismic Design Categories listed in Table 10.3.2.2 shall comply with the requirements in the sections referenced in that table.

Exception

1. Vertical structural irregularities of Types la, lb, or 2 in Table 10.3.2.2 do not apply where no story drift ratio under design lateral seismic force is greater than 130% of the story drift ratio of the next story above. Torsional effects need not be considered in the calculation of story drifts. The story drift ratio relationship for the top 2 stories of the structure are not required to be evaluated.
2. Irregularities Types la, lb, and 2 of Table 10.3.2.2 are not required to be considered for 1- or 2-story buildings in Seismic Design Categories A, B, C, or D.

10.3.3 **Redundancy.** A reliability factor, ρ, shall be assigned to all structures in accordance with this Section, based on the extent of structural redundancy inherent in the lateral force-resisting system.

10.3.3.1 **Seismic Design Categories A, B, and C.** For structures in Seismic Design Categories A, B, and C, the value of ρ is 1.0.

10.3.3.2 **Seismic Design Category D.** For structures in Seismic Design Category D, the value of ρ is 1.25.

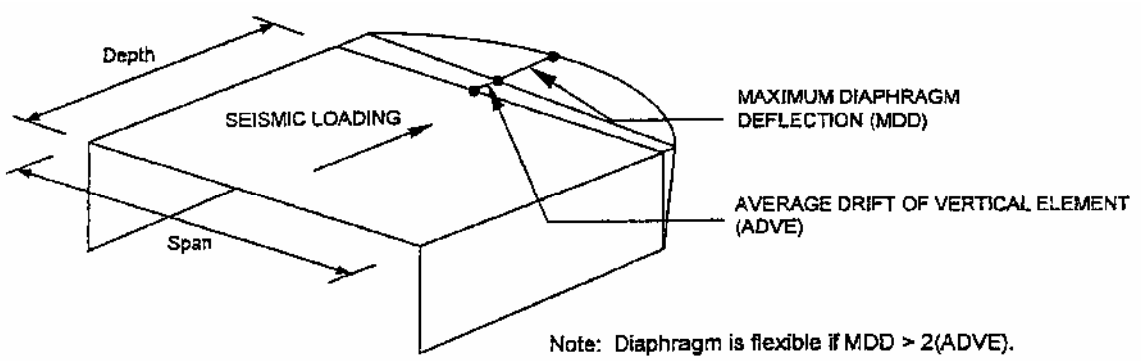


Figure 10.3-1: Flexible Diaphragm.

TABLE 10.3.2.1:
PLAN STRUCTURAL IRREGULARITIES

Irregularity Type and Description		Reference Section	Seismic Design Category Application
1a.	Torsional Irregularity Torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	10.11.4.2 10.9.5.2	D C, D
1b.	Extreme Torsional Irregularity Extreme Torsional Irregularity is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	10.11.4.2 10.9.5.2	D C and D
2.	Re-entrant Corners Plan configurations of a structure and its lateral force-resisting system contain re-entrant corners, where both projections of the structure beyond a re-entrant corner are greater than 15% of the plan dimension of the structure in the given direction.	10.11.4.2	D
3.	Diaphragm Discontinuity Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50% of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50% from one-story to the next.	10.11.4.2	D
4.	Out-of-Plane Offsets Discontinuities in a lateral force-resistance path, such as out-of-plane offsets of the vertical elements.	10.11.4.2 10.11.2.11	D, B, C, D
5.	Nonparallel Systems The vertical lateral force-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral force-resisting system.	10.11.3.1	C, D

TABLE 10.3.2.2: VERTICAL STRUCTURAL IRREGULARITIES

	Irregularity Type and Description	Reference Section	Seismic Design Category Application
la.	Stiffness Irregularity: Soft Story A soft story is one in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above.	10.6.1	D
lb.	Stiffness Irregularity: Extreme Soft Story An extreme soft story is one in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.	10.6.1	D
2.	Weight (Mass) Irregularity Mass irregularity shall be considered to exist where the effective mass of any story is more than 150% of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.	10.6.1	D
3.	Vertical Geometric Irregularity Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral force-resisting system in any story is more than 130% of that in an adjacent story.	10.6.1	D
4.	In-Plane Discontinuity in Vertical Lateral Force-Resisting Elements In-plane discontinuity in vertical lateral force-resisting elements shall be considered to exist where an in-plane offset of the lateral force-resisting elements is greater than the length of those elements or there exists a reduction in stiffness of the resisting element in the story below.	10.6.1 10.11.2.11	B, C, D
5.	Discontinuity in Lateral Strength: Weak Story A weak story is one in which the story lateral strength is less than 80% of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	10.11.2.2 10.6.1	B, C, D D

SECTION 10.4
SEISMIC LOAD EFFECTS AND COMBINATIONS

The effects on the structure and its components due to seismic forces shall be combined with the effects of other loads in accordance with the combinations of load effects given in Chapter 2. For use with those combinations, the earthquake-induced force effect shall include vertical and horizontal effects as given by Eq. 10.4-1 or 10.4-2, as applicable. The vertical seismic effect term $0.2S_{DS}D$ need not be included where S_{DS} is equal to or less than 0.125 in Eqs. 10.4-1, 10.4-2, 10.4.1-1, and 10.4.1-2. The vertical seismic effect term $0.2S_{DS}D$ need not be included in Eq. 10.4-2 when considering foundation overturning.

For Eq. 2.3.2-5 in Section 2.3.2 or Eq. 2.4.1-5 and Eq. 2.4.1-6 in Section 2.4.1:

$$E = \rho Q_E + 0.2S_{DS}D \qquad \textbf{(Eq. 10.4-1)}$$

For Eq. 2.3.2-7 in Section 2.3.2 or Eq. 2.4.1-8 in Section 2.4.1:

$$E = \rho\, Q_E - 0.2 S_{DS} D \qquad \textbf{(Eq. 10.4-2)}$$

Where

E	=	the effect of horizontal and vertical earthquake-induced forces
S_{DS}	=	the design spectral response acceleration at short periods obtained from Section 9.4.4
D	=	the effect of dead load, D
Q_E	=	the effect of horizontal seismic (earthquake-induced) forces
ρ	=	the reliability factor as per Section 10.3.3

10.4.1 **Special Seismic Load.** Where specifically indicated in SBC 301, the special seismic load of Eq. 10.4.1-1 shall be used to compute E for use in Eq. 2.3.2-5 in Section 2.3.2 or Eq. 2.4.1-5 and Eq. 2.4.1-6 in Section 2.4.1 and the special seismic load of Eq. 10.4.1-2 shall be used to compute E in Eq. 2.3.2-7 in Section 2.3.2 or Eq. 2.4.1-8 in Section 2.4.1:

$$E = \Omega_o Q_E + 0.2 S_{DS} D \qquad \textbf{(Eq. 10.4.1-1)}$$

$$E = \Omega_o Q_E - 0.2 S_{DS} D \qquad \textbf{(Eq. 10.4.1-2)}$$

Where

Ω_o = over strength factor as defined in Table 10.2.

The value of the quantity $\Omega_o Q_E$ in Eqs. 10.4.1-1 and 10.4.1-2 need not be taken greater than the capacity of other elements of the structure to transfer force to the component under consideration.

Where allowable stress design methodologies are used with the special load of this Section applied in Eq. 2.4.1-5 or Eq. 2.4.1-6 of Section 2.4.1, 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 SBC 301 or the material reference standard.

SECTION 10.5
DIRECTION OF LOADING

The directions of application of seismic forces used in the design shall be those which will produce the most critical load effects. It shall be permitted to satisfy this requirement using the procedures of Section 10.5.1 for Seismic Design Category A and B, Section 10.5.2 for Seismic Design Category C, and Section 10.5.3 for Seismic Design Category D. All structural components and their connections shall be provided with strengths sufficient to resist the effects of the seismic forces prescribed herein. Loads shall be combined as prescribed in Section 10.4.

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

10.5.2 **Seismic Design Category C.** Loading applied to structures assigned to Seismic Design Category C shall, as a minimum, conform to the requirements of Section 10.5.1 for Seismic Design Categories A and B and the requirements of this Section. Structures that have plan structural irregularity Type 5 in Table 10.3.2.1 shall be analyzed for seismic forces using a three-dimensional representation and the following procedure:

a. The structure shall be analyzed using the equivalent lateral force analysis procedure of Section 10.9 or the modal response spectrum analysis procedure of Section 10.10, as permitted under Section 10.6.1, 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 may be assumed to be satisfied if components and their foundations are designed for the following combination of prescribed loads: 100% of the forces for one direction plus 30% of the forces for the perpendicular direction; the combination requiring the maximum component strength shall be used.

10.5.3 **Seismic Design Category D**. Structures assigned to Seismic Design Category D shall, as a minimum, conform to the requirements of Section 10.5.2 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% 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. The procedure of Section 10.5.2a shall be permitted to be used to satisfy this requirement. Two-dimensional analyses shall be permitted for structures with flexible diaphragms.

SECTION 10.6
ANALYSIS PROCEDURES

A structural analysis conforming to one of the types permitted in Section 10.6.1 shall be made for all structures. Application of loading shall be as indicated in Section 10.5, and as required by the selected analysis procedure. All members of the structure's seismic force-resisting system and their connections shall have adequate strength to resist the forces, Q_E, predicted by the analysis, in combination with other loads, as required by Section 10.4. Drifts predicted by the analysis shall be within the limits specified by Section 10.12.

Exception: For structures designed using the index force analysis procedure of Section 10.7 or the simplified analysis procedure of Section 10.8, drift need not be evaluated.

10.6.1 **Analysis Procedures.** The structural analysis required by Section 10.6 shall consist of one of the types permitted in Table 10.6.1, based on the structure's Seismic Design Category, structural system, dynamic properties and regularity, or as per Section 10.14.

SECTION 10.7
INDEX FORCE ANALYSIS PROCEDURE FOR SEISMIC DESIGN OF BUILDINGS

See Section 10.6.1 for limitations on the use of this procedure. An index force analysis shall consist of the application of static lateral index forces to a linear mathematical model of the structure, independently in each of two orthogonal directions. The lateral index forces shall be as given by Eq. 10.7-1 and shall be applied simultaneously at each floor level. For purposes of analysis, the structure shall be considered to be fixed at the base:

$$F_x = 0.01\,\omega_x \qquad \textbf{(Eq. 10.7-1)}$$

where

F_x = the design lateral force applied at story x

ω_x = the portion of the total gravity load of the structure, W, located or assigned to Level x

W = the effective seismic weight of the structure, including the total dead load and other loads listed below:

1. In areas used for storage, a minimum of 25% of the floor live load (floor live load in public garages and open parking structures need not be included.)
2. Where an allowance for partition load is included in the floor load design, the actual partition weight or a minimum weight of 0.5 kN/m^2 of floor area, whichever is greater.
3. Total operating weight of permanent equipment.

TABLE 10.6.1:
PERMITTED ANALYTICAL PROCEDURES

Seismic Design Category	Structural Characteristics	Index Force Analysis Section 10.7	Simplified Analysis Section 10.8	Equivalent Lateral Force Analysis Section 10.9	Modal Response Spectrum Analysis Section 10.10
A	Regular or irregular	P	P	P	P
B, C	Regular or irregular	NP	NP	P	P
D	Regular structures with $T< 3.5\ T_s$	NP	NP	P	P
	Irregular structures with $T < 3.5\ T_s$ and having only plan irregularities type 2, 3, 4, or 5 of Table 10.3.2.1 or vertical irregularities type 4 or 5 of Table 10.3.2.2	NP	NP	P	P
	All other structures	NP	NP	NP	P

Notes: P - indicates permitted, NP -indicates not permitted

SECTION 10.8
SIMPLIFIED ANALYSIS PROCEDURE FOR SEISMIC DESIGN OF BUILDINGS

For purposes of this analysis procedure, a building is considered to be fixed at the base. See Section 10.6 for limitations on the use of this procedure.

10.8.1 **Seismic Base Shear.** The seismic base shear, V, in a given direction shall be determined in accordance with the following formula:

$$V = \frac{1.2\,S_{DS}}{R}\,W \qquad \textbf{(Eq. 10.8.1)}$$

where

S_{DS}	=	the design elastic response acceleration at short period as determined in accordance with Section 9.4.4
R	=	the response modification factor from Table 10.2
W	=	the effective seismic weight of the structure as defined in Section 10.7

10.8.2 **Vertical Distribution.** The forces at each level shall be calculated using the following formula:

$$F_x = \frac{1.2\,S_{DS}}{R}\,\omega_x \qquad \textbf{(Eq. 10.8.2)}$$

where

ω_x = the portion of the effective seismic weight of the structure, W, at level x

10.8.3 **Horizontal Distribution.** Diaphragms constructed of untopped steel decking are permitted to be considered as flexible.

10.8.4 **Design Drift.** For the purposes of Section 10.12, the design story drift, Δ, shall be taken as 1% of the story height unless a more exact analysis is provided.

SECTION 10.9
EQUIVALENT LATERAL FORCE PROCEDURE

10.9.1 **General.** Section 10.9 provides required minimum standards for the equivalent lateral force procedure of seismic analysis of structures. An equivalent lateral force analysis shall consist of the application of equivalent static lateral forces to a linear mathematical model of the structure. The directions of application of lateral forces shall be as indicated in Section 10.5. The lateral forces applied in each direction shall be the total seismic base shear given by Section 10.9.2 and shall be distributed vertically in accordance with the provisions of Section 10.9.4. For purposes of analysis, the structure is considered to be fixed at the base. See Section 10.6 for limitations on the use of this procedure.

10.9.2 **Seismic Base Shear.** The seismic base shear (V) in a given direction shall be determined in accordance with the following equation:

$$V = C_s\,W \qquad \textbf{(Eq. 10.9.2-1)}$$

where

C_s = the seismic response coefficient determined in accordance with Section 10.9.2.1.

W = the total dead load and applicable portions of other loads as indicated in Section 10.7

10.9.2.1 Calculation of Seismic Response Coefficient. When the fundamental period of the structure is computed, the seismic design coefficient (C_S) shall be determined in accordance with the following equation:

$$C_S = \frac{S_{DS}}{R/I} \qquad \textbf{(Eq. 10.9.2.1-1)}$$

where

S_{DS} = the design spectral response acceleration in the short period range as determined from Section 9.4.4

R = the response modification factor in Table 10.2

I = the occupancy importance factor determined in accordance with Section 9.5

The value of the seismic response coefficient, (Cs), need not be greater than the following equation:

$$C_S = \frac{S_{D1}}{T\left(R/I\right)} \qquad \textbf{(Eq. 10.9.2.1-2)}$$

but shall not be taken less than

$$C_s = 0.044\, S_{DS} I \qquad \textbf{(Eq. 10.9.2.1-3)}$$

where I and R are defined above and

S_{D1} = the design spectral response acceleration at a period of 1.0 sec, in units of g-sec, as determined from Section 9.4.4

T = the fundamental period of the structure (sec) as determined in Section 10.9.3

For regular structures 5 stories or less in height and having a period, T, of 0.5 sec or less, the seismic response coefficient, C_S shall be permitted to be calculated using values of 1.5 g and 0.6 g, respectively, for the mapped maximum considered earthquake spectral response accelerations S_S and S_1.

10.9.3 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 10.9.3.1 and the approximate fundamental period (T_a) determined from Eq. 10.9.3.2-1. As an alternative to performing an analysis to determine the fundamental period (T), it shall be permitted to use the approximate building period, (T_a), calculated in accordance with Section 10.9.3.2, directly.

10.9.3.1 Upper Limit on Calculated Period. The fundamental building period (T) determined in a properly substantiated analysis shall not exceed the product of the coefficient for upper limit on calculated period (C_u) from Table 10.9.3.1 and the approximate fundamental period *(T_a)* determined in accordance with Section 10.9.3.2.

TABLE 10.9.3.1: COEFFICIENT FOR UPPER LIMIT ON CALCULATED PERIOD

Design Spectral Response Acceleration at 1 Second, 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
≤ 0.05	1.7

10.9.3.2 **Approximate Fundamental Period.** The approximate fundamental period *(T_a)*, in seconds, shall be determined from the following equation:

$$T_a = C_t h_n^{x} \qquad \textbf{(Eq. 10.9.3.2-1)}$$

where h_n is the height in (m) above the base to the highest level of the structure and the coefficients C_t and x are determined from Table 10.9.3.2.

Alternatively, it shall be permitted to determine the approximate fundamental period *(T_a)*, in seconds, from the following equation for structures not exceeding 12 stories in height in which the lateral-force-resisting system consists entirely of concrete or steel moment resisting frames and the story height is at least 3 m:

$$T_a = 0.1N \qquad \textbf{(Eq. 10.9.3.2-la)}$$

where N = number of stories

The approximate fundamental period, T_a, in seconds for masonry or concrete shear wall structures shall be permitted to be determined from Eq. 10.9.3.2-2 as follows:

$$T_a = \frac{0.0062}{\sqrt{C_w}} h_n \qquad \textbf{(Eq. 10.9.3.2-2)}$$

where h_n is as defined above and C_w, is calculated from Eq. 10.9.3.2-3 as follows:

$$C_w = \frac{100}{A_B} \sum_{i=1}^{n} \left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{\left[1 + 0.83\left(\frac{h_i}{D_i}\right)^2\right]} \qquad \textbf{(Eq. 10.9.3.2-3)}$$

where

A_B = the base area of the structure m^2

A_i = the area of shear wall "i" in m^2

D_i = the length of shear wall "i" in m

n = the number of shear walls in the building effective in resisting lateral forces in the direction under consideration

TABLE 10.9.3.2:
VALUES OF APPROXIMATE PERIOD PARAMETERS C_t AND x

Structure Type	C_t	x
Moment resisting frame systems of steel in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by more rigid components that will prevent the frames from deflecting when subjected to seismic forces	0.068	0.8
Moment resisting frame systems of reinforced concrete in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by more rigid components that will prevent the frame from deflecting when subjected to seismic forces	0.044	0.9
Eccentrically braced steel frames	0.07	0.75
All other structural systems	0.055	0.75

10.9.4 **Vertical Distribution of Seismic Forces.** The lateral seismic force (F_x) (kN) induced at any level shall be determined from the following equations:

$$F_x = C_{vx} V \qquad \textbf{(Eq. 10.9.4-1)}$$

and

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad \textbf{(Eq. 10.9.4-2)}$$

where

C_{vx} = vertical distribution factor

V = total design lateral force or shear at the base of the structure, (kN)

w_i and w_x = the portion of the total gravity load of the structure (W) located or assigned to Level i or x

h_i and h_x = the height (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 sec or less, $k = 1$

for structures having a period of 2.5 sec or more, $k = 2$

for structures having a period between 0.5 and 2.5 seconds, k shall be 2 or shall be determined by linear interpolation between 1 and 2

10.9.5 **Horizontal Shear Distribution and Torsion.** The seismic design story shear in any story (V_x) (kN) shall be determined from the following equation:

$$V_x = \sum_{i=x}^{n} F_i \qquad \textbf{(Eq. 10.9.5)}$$

where F_i = the portion of the seismic base shear (V) (kN) induced at Level i.

10.9.5.1 **Direct Shear.** The seismic design story shear (V_x) (kN) shall be distributed to the various vertical elements of the seismic force-resisting system in the story under consideration based on the relative lateral stiffness of the vertical resisting elements and the diaphragm.

10.9.5.2 **Torsion.** Where diaphragms are not flexible, the design shall include the torsional moment *(M_t)* (kN·m) resulting from the location of the structure masses plus the accidental torsional moments *(M_{ta})* (kN·m) caused by assumed displacement of the mass each way from its actual location by a distance equal to 5% 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% 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.

Structures of Seismic Design Categories C and D where Type 1 torsional irregularity exists as defined in Table 10.3.2.1 shall have the effects accounted for by multiplying M_{ta} at each level by a torsional amplification factor (A_x) determined from the following equation:

$$A_x = \left(\frac{\delta_{max}}{1.2\,\delta_{avg}} \right)^2 \qquad \textbf{(Eq. 10.9.5.2)}$$

where

δ_{max} = the maximum displacement at Level x (mm)

δ_{avg} = the average of the displacements at the extreme points of the structure at Level x (mm)

The torsional amplification factor (A_x) is not required to exceed 3.0. The more severe loading for each element shall be considered for design.

10.9.6 **Overturning.** The structure shall be designed to resist overturning effects caused by the seismic forces determined in Section 10.9.4. At any story, the increment of overturning moment in the story under consideration shall be distributed to the various vertical elements of the lateral force-resisting system in the same proportion as the distribution of the horizontal shears to those elements.
The overturning moments at Level x *(M_x)* (kN·m) shall be determined from the following equation:

$$M_x = \sum_{i=x}^{n} F_i (h_i - h_x) \qquad \textbf{(Eq. 10.9.6)}$$

where

F_i = the portion of the seismic base shear (V) induced at Level i

h_i and h_x = the height (m) from the base to Level i or x

The foundations of structures, except inverted pendulum-type structures, shall be permitted to be designed for 75% of the foundation overturning design moment (M_f) (kN·m) at the foundation-soil interface determined using the equation for the overturning moment at Level x *(M_x)* (kN·m).

10.9.7 **Drift Determination and P-Delta Effects.** Story drifts and, where required, member forces and moments due to P-delta effects shall be determined in accordance with this Section. Determination of story drifts shall be based on the application of the design seismic forces to a mathematical model of the physical structure. The model shall include the stiffness and strength of all elements that are significant to the distribution of forces and deformations in the structure and shall represent the spatial distribution of the mass and stiffness of the structure. In addition, the model shall comply with the following:

1. Stiffness properties of reinforced concrete and masonry elements shall consider the effects of cracked sections, and
2. For steel moment resisting frame systems, the contribution of panel zone deformations to overall story drift shall be included.

10.9.7.1 **Story Drift Determination.** The design story drift (Δ) shall be computed as the difference of the deflections at the top and bottom of the story under consideration. Where allowable stress design is used, Δ shall be computed using code-specified earthquake forces without reduction.

Exception: For structures of Seismic Design Categories C and D having plan irregularity Types la or 1b of Table 10.3.2.1, the design story drift, D, shall be computed as the largest difference of the deflections along any of the edges of the structure at the top and bottom of the story under consideration.

The deflections of Level *x* at the center of the mass (δ_x) (mm) shall be determined in accordance with the following equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I} \qquad \textbf{(Eq. 10.9.7.1)}$$

where

C_d = the deflection amplification factor in Table 10.2

δ_{xe} = the deflections determined by an elastic analysis

I = the importance factor determined in accordance with Section 9.5

The elastic analysis of the seismic force-resisting system shall be made using the prescribed seismic design forces of Section 9.5.5.4. For the purpose of this Section, the value of the base shear, V, used in Eq. 10.9.2-1 need not be limited by the value obtained from Eq. 10.9.2.1-3.

For determining compliance with the story drift limitation of Section 10.12, the deflections at the center of mass of Level *x* (δ_x) (mm) shall be calculated as required in this Section. For the purposes of this drift analysis only, the upper-bound limitation specified in Section 9.5.5.3 on the computed fundamental period, T, in seconds, of the building does not apply for computing forces and displacements.

Where applicable, the design story drift (Δ) (mm) shall be increased by the incremental factor relating to the P-delta effects as determined in Section 10.9.7.2.

When calculating drift, the redundancy coefficient, ρ, is not used.

10.9.7.2 **P-Delta Effects.** P-delta effects on story shears and moments, the resulting member forces and moments, and the story drifts induced by these effects are not required to be considered when the stability coefficient (θ) as determined by the following equation is equal to or less than 0.10:

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d} \qquad \textbf{(Eq. 10.9.7.2-1)}$$

where

P_x = the total vertical design load at and above Level x. (kN); when computing P_x, no individual load factor need exceed 1.0

Δ = the design story drift as defined in Section 10.9.7.1 occurring simultaneously with V_x, (mm)

V_x = the seismic shear force acting between Levels x and x - 1, (kN)

h_{sx} = the story height below Level x, (mm)

C_d = the deflection amplification factor in Table 10.2

The stability coefficient (θ) shall not exceed θ_{max} determined as follows:

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25 \qquad \textbf{(Eq. 10.9.7.2-2)}$$

where β is the ratio of shear demand to shear capacity for the story between Level x and x - 1. This ratio may be conservatively taken as 1.0.

When 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 (a_d) shall be determined by rational analysis. To obtain the story drift for including the P-delta effect, the design story drift determined in Section 10.9.7.1 shall be multiplied by 1.0/(1 - θ).

When θ is greater than θ_{max}, the structure is potentially unstable and shall be redesigned.

When the P-delta effect is included in an automated analysis, Eq. 10.9.7.2-2 must still be satisfied, however, the value of θ computed from Eq. 10.9.7.2-1 using the results of the P-delta analysis may be divided by (1 + θ) before checking Eq. 10.9.7.2-2.

SECTION 10.10
MODAL ANALYSIS PROCEDURE

10.10.1 **General.** Section 10.10 provides required standards for the modal analysis procedure of seismic analysis of structures. See Section 10.6 for requirements for use of this procedure. The symbols used in this method of analysis have the same meaning as those for similar terms used in Section 10.7, with the subscript m denoting quantities in the m^{th} mode.

10.10.2 **Modeling.** A mathematical model of the structure shall be constructed that represents the spatial distribution of mass and stiffness throughout the structure.

For regular structures with independent orthogonal seismic force-resisting systems, independent two-dimensional models are permitted to be constructed to represent each system. For irregular structures or structures without independent orthogonal systems, a three-dimensional model incorporating 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 are not rigid compared to the vertical elements of the lateral force-resisting system, the model should include representation of the diaphragm's flexibility 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:

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

10.10.3 **Modes.** An analysis shall be conducted to determine the natural modes of vibration for the structure, including the period of each mode, the modal shape vector ϕ, the modal participation factor, and modal mass. The analysis shall include a sufficient number of modes to obtain a combined modal mass participation of at least 90% of the actual mass in each of two orthogonal directions.

10.10.4 **Periods.** The required periods, mode shapes, and participation factors of the structure in the direction under consideration shall be calculated by established methods of structural analysis for the fixed-base condition using the masses and elastic stiffnesses of the seismic force-resisting system.

10.10.5 **Modal Base Shear.** The portion of the base shear contributed by the m^{th} mode (V_m) shall be determined from the following equations:

$$V_m = C_{sm} W_m \qquad \textbf{Eq. (10.10.5-1)}$$

$$W_m = \frac{\left(\sum_{i=1}^{n} w_i \phi_{im} \right)^2}{\sum_{i=1}^{n} w_i \phi_{im}^2} \qquad \textbf{Eq. (10.10.5-2)}$$

where

C_{sm} = the modal seismic design coefficient determined below

W_m = the effective modal gravity load

w_i = the portion of the total gravity load of the structure at Level i

ϕ_{im} = the displacement amplitude at the i^{th} level of the structure when vibrating in its m^{th} mode

The modal seismic design coefficient (C_{sm}) shall be determined in accordance with the following equation:

$$C_{sm} = \frac{S_{am}}{R/I} \qquad \textbf{(Eq. 10.10.5-3)}$$

where

S_{am} = the design spectral response acceleration at period T_m determined from the general design response spectrum of Section 9.4.5.

R = the response modification factor determined from Table 10.2.

I = the occupancy importance factor determined in accordance with Section 9.5.

T_m = the modal period of vibration (in seconds) of the mth mode of the structure

Exception: When the general design response spectrum of Section 9.4.5 is used for structures where any modal period of vibration *(T_m)* exceeds 4.0 sec, the modal seismic design coefficient *(C_{sm})* for that mode shall be determined by the following equation:

$$C_{sm} = \frac{4S_{D1}}{(R/I)T_m^2} \qquad \textbf{(Eq. 10.10.5-4)}$$

10.10.6 Modal Forces, Deflections, and Drifts. The modal force *(F_{xm})* at each level shall be determined by the following equations:

$$F_{xm} = C_{vxm} V_m \qquad \textbf{(Eq. 10.10.6-1)}$$

and

$$C_{vxm} = \frac{w_x \phi_{xm}}{\sum_{i=1}^{n} w_i \phi_{im}} \qquad \textbf{(Eq. 10.10.6-2)}$$

where

C_{vxm} = the vertical distribution factor in the m^{th} mode

V_m = the total design lateral force or shear at the base in the m^{th} mode

w_i and w_x = the portion of the total gravity load of the structure (W) located or assigned to Level i or x

ϕ_{xm} = the displacement amplitude at the x^{th} level of the structure when vibrating in its m^{th} mode

ϕ_{im} = the displacement amplitude at the i^{th} level of the structure when vibrating in its m^{th} mode

The modal deflection at each level *(δ_{xm})* shall be determined by the following equations:

$$\delta_{xm} = \frac{C_d \delta_{xem}}{I} \qquad \textbf{(Eq. 10.10.6-3)}$$

and

$$\delta_{xem} = \left(\frac{g}{4\pi^2}\right)\left(\frac{T_m^2 F_{xm}}{w_x}\right) \qquad \textbf{(Eq. 10.10.6-4)}$$

where

C_d = the deflection amplification factor determined from Table 10.2

δ_{xem} = the deflection of Level x in the m^{th} mode at the center of the mass at Level x determined by an elastic analysis

g = the acceleration due to gravity (m^2/sec)

I = the occupancy importance factor deter-mined in accordance with Section 9.5

T_m = the modal period of vibration, in seconds, of the m^{th} mode of the structure

F_{xm} = the portion of the seismic base shear in the m^{th} mode, induced at Level x, and

w_x = the portion of the total gravity load of the structure *(W)* located or assigned to Level x

The modal drift in a story (Δ_m) shall be computed as the difference of the deflections (δ_{xm}) at the top and bottom of the story under consideration.

10.10.7 **Modal Story Shears and Moments.** The story shears, story overturning moments, and the shear forces and overturning moments in vertical elements of the structural system at each level due to the seismic forces determined from the appropriate equation in Section 10.10.6 shall be computed for each mode by linear static methods.

10.10.8 **Design Values.** The design value for the modal base shear (V_t), each of the story shear, moment and drift quantities, and the deflection at each level shall be determined by combining their modal values as obtained from Sections 10.10.6 and 10.10.7. The combination shall be carried out by taking the square root of the sum of the squares of each of the modal values or where closely spaced periods in the translational and torsional modes result in significant cross-correlation of the modes, the complete quadratic combination (CQC) method, shall be used.

A base shear (V) shall be calculated using the equivalent lateral force procedure in Section 10.9. For the purpose of this calculation, a fundamental period of the structure (T), in seconds, shall not exceed the coefficient for upper limit on the calculated period (C_u) times the approximate fundamental period of the structure (T_a). Where the design value for the modal base shear (V_t) is less than 85% of the calculated base shear (V) using the equivalent lateral force procedure, the design story shears, moments, drifts, and floor deflections shall be multiplied by the following modification factor:

$$0.85 \frac{V}{V_t} \qquad \textbf{(Eq. 10.10.8)}$$

where

V = the equivalent lateral force procedure base shear, calculated in accordance with this section and Section 10.9, and

V_t = the modal base shear, calculated in accordance with this section

10.10.9 **Horizontal Shear Distribution.** The distribution of horizontal shear shall be in accordance with the requirements of Section 10.9.5 except that amplification of torsion per Section 10.9.5.2 is not required for that portion of the torsion, A_x, included in the dynamic analysis model.

10.10.10 **Foundation Overturning.** The foundation overturning moment at the foundation-soil interface may be reduced by 10%.

10.10.11 **P-Delta Effects.** The P-delta effects shall be determined in accordance with Section 10.9.7. The story drifts and base shear used to determine the story shears shall be determined in accordance with Section 10.9.7.1.

SECTION 10.11
DESIGN AND DETAILING REQUIREMENTS

The design and detailing of the components of the seismic force-resisting system shall comply with the requirements of this Section. Foundation design shall conform to the applicable requirements of Section 10.13. The materials and the systems composed of those materials shall conform to the requirements and limitations of Sections 11.1 through 11.4 for the applicable category.

10.11.1 **Seismic Design Category A**. The design and detailing of structures assigned to Category A shall comply with the requirements of this Section.

10.11.1.1 **Load Path Connections.** All parts of the structure between separation joints shall be interconnected to form a continuous path to the seismic force-resisting system, and the connections shall be capable of transmitting the seismic force (F_p) induced by the parts being connected. Any smaller portion of the structure shall be tied to the remainder of the structure with elements having a design strength capable of transmitting a seismic force of 0.133 times the short period design spectral response acceleration coefficient, S_{DS}, times the weight of the smaller portion or 5% of the portion's weight, whichever is greater. This connection force does not apply to the overall design of the lateral-force-resisting system. Connection design forces need not exceed the maxi-mum forces that the structural system can deliver to the connection.

A positive connection for resisting a horizontal force acting parallel to the member shall be provided for each beam, girder, or truss to its support. The connection shall have a minimum strength of 5% of the dead plus live load reaction. One means to provide the strength is to use connecting elements such as slabs.

10.11.1.2 **Anchorage of Concrete or Masonry Walls.** Concrete and masonry walls shall be anchored to the roof and all floors and members that provide lateral support for the wall or which are supported by the wall. The anchorage shall provide a direct connection between the walls and the roof or floor construction. The connections shall be capable of resisting the horizontal forces specified in Section 10.11.1.1 but not less than a minimum strength level, horizontal force of 4 kN/m of wall substituted for E in the load combinations.

10.11.2 **Seismic Design Category B.** Structures assigned to Seismic Design Category B shall conform to the requirements of Section 10.11.1 for Seismic Design Category A and the requirements of this Section.

10.11.2.1 **P-Delta Effects.** P-delta effects shall be included where required by Section 10.9.7.2.

10.11.2.2 **Openings.** Where openings occur in shear walls, diaphragms, or other plate-type elements, reinforcement at the edges of the openings shall be designed to transfer the stresses into the structure. The edge reinforcement shall extend into the body of the wall or diaphragm a distance sufficient to develop the force in the reinforcement. The extension must be sufficient in length to allow dissipation or transfer of the force without exceeding the shear and tension capacity of the diaphragm or the wall.

10.11.2.3 **Direction of Seismic Load.** The direction of application of seismic forces used in design shall be that which will produce the most critical load effect in each component. This requirement will be deemed satisfied if the design seismic forces are applied separately and independently in each of two orthogonal directions.

10.11.2.4 **Discontinuities in Vertical System.** Structures with a discontinuity in lateral capacity, vertical irregularity Type 5 as defined in Table 10.3.2.2, shall not be more than 2 stories or 9 m in height where the "weak" story has a calculated strength of less than 65% of the story above.

Exception: The limit does not apply where the "weak" story is capable of resisting a total seismic force equal to Ω_o times the design force prescribed in Section 10.7.

10.11.2.5 **Nonredundant Systems.** The design of a structure shall consider the potentially adverse effect that the failure of a single member, connection, or component of the seismic force-resisting system will have on the stability of the structure; see Section 1.4.

10.11.2.6 **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.

10.11.2.7 **Diaphragms.** 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 which will permit the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads.

Floor and roof diaphragms shall be designed to resist F_p where F_p is the larger of:

1. The portion of the design seismic force at the level of the diaphragm that depends on the diaphragm for transmission to the vertical elements of the seismic force-resisting system, or
2. $F_p = 0.2 S_{DS} I w_p + V_{px}$, **(Eq. 10.11.2.7)**

where

F_p = the seismic force induced by the parts

I = occupancy importance factor (Table 9.5)

S_{DS} = the short period site design spectral response acceleration coefficient, Section 9.4.1

w_p = the weight of the diaphragm and other elements of the structure attached to it

V_{px} = the portion of the seismic shear force at the level of the diaphragm, required to be transferred to the components of the vertical seismic force-resisting system because of the offsets or changes in stiffness of the vertical components above or below the diaphragm.

Diaphragms shall be designed for both the shear and bending stresses resulting from these forces. Diaphragms shall have ties or struts to distribute the wall anchorage forces into the diaphragm. Diaphragm connections shall be positive, mechanical, or welded type connections.

At diaphragm discontinuities, such as openings and re-entrant corners, the design shall ensure 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.

10.11.2.8 **Anchorage of Concrete or Masonry Walls.** Exterior and interior bearing walls and their anchorage shall be designed for a force normal to the surface equal to 40% of the short period design spectral response acceleration, S_{DS}, times the occupancy importance factor, I, multiplied by the weight of wall (W_c) associated with the anchor, with a minimum force of 10% of the weight of the wall. Interconnection of 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. The connections shall also satisfy Section 10.11.1.2.

The anchorage of concrete or masonry walls to supporting construction shall provide a direct connection capable of resisting the greater of the force $0.4\ S_{DS}I\ W_C$ as given above or 5.84 $S_{DS}I$ kN/m of wall or the force specified in Section 10.11.1.2. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 1200 mm.

10.11.2.9 **Inverted Pendulum-Type Structures.** 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 10.7 and varying uniformly to a moment at the top equal to one-half the calculated bending moment at the base.

10.11.2.10 **Anchorage of Nonstructural Systems.** When required by Chapter 12, all portions or components of the structure shall be anchored for the seismic force, F_p, prescribed therein.

10.11.2.11 **Elements Supporting Discontinuous Walls or Frames.** Columns, beams, trusses, or slabs supporting discontinuous walls or frames of structures having plan irregularity Type 4 of Table 10.3.2.1 or vertical irregularity Type 4 of Table 10.3.2.2 shall have the design strength to resist the maximum axial force that can develop in accordance with the special seismic loads of Section 10.4.1.

10.11.3 **Seismic Design Category C.** Structures assigned to Category C shall conform to the requirements of Section 10.11.2 for Category B and to the requirements of this Section.

10.11.3.1 **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. Collector elements, splices, and their connections to resisting elements shall resist the special seismic loads of Section 10.4.1.

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 Eq. 10.11.4.4.

The quantity Ω_o E in Eq. 10.4.1-1 need not exceed the maximum force that can be transferred to the collector by the diaphragm and other elements of the lateral force-resisting system.

10.11.3.2 **Anchorage of Concrete or Masonry Walls.** Concrete or masonry walls shall be anchored to all floors, roofs, and members that provide out-of-plane lateral support for the wall or that are supported by the wall. The anchorage shall provide a positive direct connection between the wall and floor, roof, or supporting member capable of resisting horizontal forces specified in this Section for structures with flexible diaphragms or with Section 12.1.3 (using a_p of 1.0 and R_p of 2.5) for structures with diaphragms that are not flexible.

Anchorage of walls to flexible diaphragms shall have the strength to develop the out-of-plane force given by Eq. 10.11.3.2:

$$F_p = 0.8S_{DS}IW_p \qquad \textbf{(Eq. 10.11.3.2)}$$

where

F_p = the design force in the individual anchors

S_{DS} = the design spectral response acceleration at short periods per Section 9.4.4

I = the occupancy importance factor per Section 9.5

W_p = the weight of the wall tributary to the anchor

Diaphragms shall be provided with continuous ties or struts between diaphragm chords to distribute these anchorage forces into the diaphragms. Added chords may be used to form subdiaphragms to transmit the anchorage forces to the main continuous cross ties. The maximum length-to-width ratio of the structural subdiaphragm 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.

The strength design forces for steel elements of the wall anchorage system, other than anchor bolts and reinforcing steel, shall be 1.4 times the forces otherwise required by this Section.

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.

Diaphragm to 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.

When 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.

When 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.

10.11.4 **Seismic Design Category D.** Structures assigned to Category D shall conform to the requirements of Section 10.11.3 for Category C and to the requirements of this Section.

10.11.4.1 **Collector Elements.** In addition to the requirements of Section 10.11.3.1, collector elements, splices, and their connections to resisting elements shall resist the forces determined in accordance with Section 10.11.4.4.

10.11.4.2 **Plan or Vertical Irregularities.** When the ratio of the strength provided in any story to the strength required is less than two-thirds of that ratio for the story immediately above, the potentially adverse effect shall be analyzed and the strengths shall be adjusted to compensate for this effect.

For structures having a plan structural irregularity of Type 1, 2, 3, or 4 in Table 10.3.2.1 or a vertical structural irregularity of Type 4 in Table 10.3.2.2, the design forces determined from Section 10.9.2 shall be increased 25% 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 special seismic loads of Section 10.4.1, in accordance with Section 10.11.3.1.

10.11.4.3 **Vertical Seismic Forces.** The vertical component of earthquake ground motion shall be considered in the design of horizontal cantilever and horizontal prestressed components. The load combinations used in evaluating such components shall include E as defined by Eqs. 10.4-1 and 10.4-2. 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 10.4.

10.11.4.4 **Diaphragms.** The deflection in the plane of the diaphragm shall not exceed the permissible deflection of the attached elements. Permissible deflection shall be that deflection that will permit the attached elements to maintain structural integrity under the individual loading and continue to support the prescribed loads. Floor and roof diaphragms shall be designed to resist design seismic forces determined in accordance with Eq. 10.11.4.4 as follows:

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \qquad \textbf{(Eq. 10.11.4.4)}$$

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

The force determined from Eq. 10.11.4.4 need not exceed $0.4S_{DS}I\ w_{px}$ but shall not be less than $0.2S_{DS}Iw_{px}$. When 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. 10.11.4.4.

SECTION 10.12
DEFLECTION, DRIFT LIMITS, AND BUILDING SEPARATION

10.12.1 **Drift Limits.** The design story drift (Δ) as determined in Section 10.9.7 or 10.10.6, shall not exceed the allowable story drift (Δ_a) as obtained from Table 10.12 for any story. For structures with significant torsional deflections, the maximum drift shall include torsional effects.

10.12.2 **Building Separation.** All structures shall be separated from adjoining structures. Separations shall allow for the displacement (δ_x) as determined in Section 10.9.7.1. Adjacent buildings on the same property shall be separated by at least δ_{xt} where

$$\delta_{xt} = \sqrt{(\delta_{x1})^2 + (\delta_{x2})^2} \qquad \textbf{(Eq. 10.12)}$$

and δ_{x1} and δ_{x2} are total deflections for building 1 & 2 respectively.

TABLE 10.12:
ALLOWABLE STORY DRIFT, Δ_a[a]

Structure	Occupancy Category		
	I, II	III	IV
Structures, other than masonry shear wall or masonry wall frame structures, four stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}$[b]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures[c]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
Masonry wall frame structures	$0.013h_{sx}$	$0.013h_{sx}$	$0.010h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

a h_{sx} is the story height below Level x.

b There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 10.12 is not waived.

c 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.

SECTION 10.13
FOUNDATION DESIGN REQUIREMENTS

10.13.1 **General.** This Section includes only those foundation requirements that are specifically related to seismic-resistant construction. It assumes compliance with other basic requirements, which include, but are not limited to, the extent of the foundation investigation, fills present or to be placed in the area of the structure, slope stability, subsurface drainage, settlement control, and pile, requirements. Except as specifically noted, the term "pile" as used in Sections 10.13.4.4 and 10.13.5.4 includes foundation piers, caissons, and piles, and the term "pile cap" includes the elements to which piles are connected, including grade beams and mats.

10.13.2 **Seismic Design Category A**. There are no special requirements for the foundations of structures assigned to Category A.

10.13.3 **Seismic Design Category B.** The determination of the site coefficient, Section 9.4.3, shall be documented and the resisting capacities of the foundations, subjected to the prescribed seismic forces of Chapters 9 and 10, shall meet the following requirements.

10.13.3.1 **Structural Components.** The design strength of foundation components subjected to seismic forces alone or in combination with other prescribed loads and their detailing requirements shall conform to the requirements of Sections 11.1 through 11.4. The strength of foundation components shall not be less than that required for forces acting without seismic forces.

10.13.3.2 **Soil Capacities.** The capacity of the foundation soil in bearing, or the capacity of the soil interface between pile or pier and the soil, shall be sufficient to support the structure with all prescribed loads, without seismic forces, taking due account of the settlement that the structure can withstand. For the load combination, including earthquake as specified in Section 10.4, the soil capacities must be sufficient to resist loads at acceptable strains considering both the short duration of loading and the dynamic properties of the soil.

10.13.4 **Seismic Design Category C.** Foundations for structures assigned to Category C shall conform to all of the requirements for Categories A and B and to the additional requirements of this Section.

10.13.4.1 **Investigation.** When required by the authority having jurisdiction, a written geotechnical or geologic report shall be submitted. This report shall include, in addition to the requirements of Section 10.13.1 and the evaluations required in Section 10.13.3, the results of an investigation to evaluate the following potential earthquake hazards:

1. Slope instability
2. Liquefaction
3. Lateral spreading
4. Surface rupture

The investigation shall contain recommendations for appropriate foundation designs or other measures to mitigate the effects of the above hazards.

10.13.4.2 **Pole-Type Structures.** When construction employing posts or poles as columns embedded in earth or embedded in concrete footings in the earth is used to resist lateral loads, the depth of embedment required for posts or poles to resist seismic forces shall be determined by means of the design criteria established in the foundation investigation report.

10.13.4.3 **Foundation Ties.** Individual pile caps, drilled piers, or caissons shall be interconnected by ties. All ties shall have a design strength in tension or compression greater than a force equal to 10% of S_{DS} times the larger pile cap or column factored dead plus factored live load unless it is demonstrated that equivalent restraint will be provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade or confinement by competent rock, hard cohesive soils, very dense granular soils, or other approved means.

10.13.4.4 **Special Pile Requirements.** Concrete piles, concrete-filled steel pipe piles, drilled piers, or caissons require minimum bending, shear, tension, and elastic strain capacities. Refer to Section 15.3.1 for supplementary provisions.

10.13.5 **Foundation Requirements for Seismic Design Category D.** Foundations for structures assigned to Seismic Design Category D shall conform to all of the requirements for Seismic Design Category C construction and to the additional requirements of this Section. Design and construction of concrete foundation components shall conform to the requirements of SBC 304 Section 21.8, except as modified by the requirements of this Section.

10.13.5.1 **Investigation.** The owner shall submit to the authority having jurisdiction a written report that includes an evaluation of the items in Section 10.13.4.1 and the determination of lateral pressures on basement and retaining walls due to earthquake motions.

10.13.5.2 **Foundation Ties.** Individual spread footings founded on soil defined in section 14.1.1 as Site class E or F shall be interconnected by ties. Ties shall conform to Section 10.13.4.3.

10.13.5.3 **Liquefaction Potential and Soil Strength Loss.** The geotechnical report required by Section 10.13.5.1 shall assess potential consequences of liquefaction and soil strength loss, including estimation of differential settlement, lateral movement or reduction in foundation soil-bearing capacity, and shall discuss 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.

10.13.5.4 **Special Pile and Grade Beam Requirements.** Piling shall be designed and constructed to withstand maximum imposed curvatures from earthquake ground motions and structure response. Curvatures shall include free-field soil strains (without the structure) modified for soil-pile-structure interaction coupled with pile deformations induced by lateral pile resistance to structure seismic forces. Concrete piles in Site class E or F shall be designed and detailed in accordance with SBC 304 Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 within seven pile diameters of the pile cap and the interfaces of soft to medium stiff or liquefiable strata. Refer to Section 15.3.2 for supplementary provisions in addition to those given in Section 15.3.1. Batter piles and their connections shall be capable of resisting forces and moments from the special seismic load combinations of Section 10.4.1. For precast, prestressed concrete piles, detailing provisions as given in Section 15.3.2.4 shall apply.

Section 21.8.3.3 of SBC 304 need not apply when grade beams have the required strength to resist the forces from the special seismic loads of Section 10.4.1. Section 21.8.4.4 (a) of SBC 304 need not apply to concrete piles. Section 21.8.4.4 (b) of SBC 304 need not apply to precast, prestressed concrete piles.

Design of anchorage of piles into the pile cap shall consider the combined effect of axial forces due to uplift and bending moments due to fixity to the pile cap. For piles required to resist uplift forces or provide rotational restraint, 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 1.3 times the pile pullout resistance, or the axial tension force resulting from the special seismic loads of Section 10.4.1.

2. In the case of rotational restraint, the lesser of the axial and shear forces and moments resulting from the special seismic loads of Section 10.4.1 or development of the full axial, bending, and shear nominal strength of the pile.

Splices of pile segments shall develop the nominal strength of the pile section, 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 special seismic loads of Section 10.4.1.

Pile moments, shears, and lateral deflections used for design shall be established considering the interaction of the shaft and soil. Where the ratio of the depth of embedment of the pile-to-the-pile diameter or width is less than or equal to 6, the pile may be assumed to be flexurally rigid with respect to the soil.

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 or widths. Pile group effects on vertical nominal strength shall be included where pile center-to-center spacing is less than three pile diameters or widths.

SECTION 10.14
SUPPLEMENTARY METHODS OF ANALYSIS

10.14.1 **Linear Response History Analysis.** A linear response history analysis shall consist of an analysis of a linear mathematical model of the structure to determine its response, through methods of numerical integration, to suites of ground motion acceleration histories compatible with the design response spectrum for the site. For purposes of analysis, the structure shall be permitted to be considered to be fixed at the base, or alternatively, it shall be permitted to use realistic assumptions with regard to the stiffness of foundations.

10.14.2 **Nonlinear Response History Analysis.** A nonlinear response history analysis shall consist of an analysis of a mathematical model of the structure that directly accounts for the nonlinear hysteretic behavior of the structure's components to determine its response through methods of numerical integration to suites of ground motion acceleration histories compatible with the design response spectrum for the site.

10.14.3 **Soil-Structure Interaction.** Incorporate the effects of soil-structure interaction is optional. The use of this option will decrease the design values of the base shear, lateral forces, and overturning moments but may increase the computed values of the lateral displacements and the secondary forces associated with the P-delta effects.

CHAPTER 11
MATERIAL SPECIFIC SEISMIC DESIGN AND DETAILING REQUIREMENTS

SECTION 11.1
STEEL

11.1.1 **Reference Documents.** The design, construction, and quality of steel components that resist seismic forces shall conform to the requirements of SBC 306 and the references listed (Ref. 11.1-1 through Ref. 11.1-5) except that modifications are necessary to make the references compatible with the provisions of this document. Section 15.4 provides supplementary provisions for this compatibility.

SECTION 11.2
STRUCTURAL CONCRETE

11.2.1 **Reference Documents.** The quality and testing of materials and the design and construction of structural concrete components that resist seismic forces shall conform to the requirements of SBC 304. Section 15.5 provides the supplementary provisions for this compatibility. The load combinations of Section 2.4.1 are not applicable for design of reinforced concrete to resist earthquake loads.

SECTION 11.3
COMPOSITE STRUCTURES

11.3.1 **Reference Documents.** The design, construction, and quality of composite steel and concrete components that resist seismic forces shall conform to the relevant requirements of SBC 304 and SBC 306 and the references listed (Ref. 11.3-1 and Ref. 11.3-2) except as modified by the provisions of this chapter.

SECTION 11.4
MASONRY

11.4.1 **Reference Documents.** The design, construction, and quality assurance of masonry components that resist seismic forces shall conform to the requirements of SBC 304 and Ref. 11.4-1 except that modifications are necessary to make the reference compatible with the provisions of this document. Section 15.6 provides the supplementary provisions for this compatibility.

CHAPTER 12
SEISMIC DESIGN REQUIREMENTS FOR NON-STRUCTURAL COMPONENTS

SECTION 12.1
GENERAL

Chapter 12 establishes minimum design criteria for architectural, mechanical, electrical, and non-structural systems, components, and elements permanently attached to structures including supporting structures and attachments (hereinafter referred to as "components"). The design criteria establish minimum equivalent static force levels and relative displacement demands for the design of components and their attachments to the structure, recognizing ground motion and structural amplification, component toughness and weight, and performance expectations. Seismic Design Categories for structures are defined in Section 9.6. For the purposes of this Section, components shall be considered to have the same Seismic Design Category as that of the structure that they occupy or to which they are attached unless otherwise noted.

This Chapter also establishes minimum seismic design force requirements for nonbuilding structures that are supported by other structures where the weight of the nonbuilding structure is less than 25% of the combined weight of the nonbuilding structure and the supporting structure. Seismic design requirements for nonbuilding structures that are supported by other structures where the weight of the nonbuilding structure is 25% or more of the combined weight of the nonbuilding structure and supporting structure are prescribed in Chapter 13. Seismic design requirements for nonbuilding structures that are supported at grade are prescribed in Chapter 13; however, the minimum seismic design forces for nonbuilding structures that are supported by another structure shall be determined in accordance with the requirements of Section 12.1.3 with R_p equal to the value of R specified in Chapter 13 and $a_p = 2.5$ for nonbuilding structures with flexible dynamic characteristics and $a_p = 1.0$ for nonbuilding structures with rigid dynamic characteristics. The distribution of lateral forces for the supported nonbuilding structure and all nonforce requirements specified in Chapter 13 shall apply to supported nonbuilding structures.

In addition, all components are assigned a component importance factor (I_p) in this chapter. The default value for I_p is 1.00 for typical components in normal service. Higher values for I_p are assigned for components, which contain hazardous substances, must have a higher level of assurance of function, or otherwise require additional attention because of their life safety characteristics. Component importance factors are prescribed in Section 12.1.5.

All architectural, mechanical, electrical, and other non-structural components in structures shall be designed and constructed to resist the equivalent static forces and displacements determined in accordance with this Chapter. The design and evaluation of support structures and architectural components and equipment shall consider their flexibility as well as their strength.

Exception: The following components are exempt from the requirements of this Chapter:

1. All components in Seismic Design Category A.

2. Architectural components in Seismic Design Category B other than parapets supported by bearing walls or shear walls provided that the importance factor *(I_p)* is equal to 1.0.

3. Mechanical and electrical components in Seismic Design Category B.

4. Mechanical and electrical components in structures assigned to Seismic Design Category C provided that the importance factor *(I_p)* is equal to 1.0.

5. Mechanical and electrical components in Seismic Design Category D where I_p = 1.0 and flexible connections between the components and associated ductwork, piping, and conduit are provided and that are mounted at (1.25 m) or less above a floor level and weigh (1800 N) or less.

6. Mechanical and electrical components in Seismic Design Category D weighing (100 N) or less where I_p = 1.0 and flexible connections between the components and associated ductwork, piping, and conduit are provided, or for distribution systems, weighing (7 N/m) or less.

The functional and physical interrelationship of components and their effect on each other shall be designed so that the failure of an essential or nonessential architectural, mechanical, or electrical component shall not cause the failure of a nearby essential architectural, mechanical, or electrical component.

12.1.1 Reference Standards.

12.1.1.1 Consensus Standards. The cited references (Ref. 12-1 through 12-13) are consensus standards and are to be considered part of these provisions to the extent referred to in this chapter.

12.1.1.2 Accepted Standards. The cited references (Ref. 12-14 through 12-21) are standards developed within the industry and represent acceptable procedures for design and construction.

12.1.2 Component Force Transfer. Components shall be attached such that the component forces are transferred to the structure. Component seismic attachments shall be bolted, welded, or otherwise positively fastened without consideration of frictional resistance produced by the effects of gravity. A continuous load path of sufficient strength and stiffness between the component and the supporting structure shall be provided. Local elements of the supporting structure shall be designed and constructed for the component forces where they control the design of the elements or their connections. The component forces shall be those determined in Section 12.1.3, except that modifications to F_p and R_p due to anchorage conditions need not be considered. The design documents shall include sufficient information relating to the attachments to verify compliance with the requirements of this chapter.

12.1.3 Seismic Forces. Seismic forces *(F_p)* shall be determined in accordance with Eq. 12.1.3-1:

$$F_p = \frac{0.4 a_p S_{DS} W_p}{R_p / I_p}\left(1 + 2\frac{z}{h}\right) \qquad \textbf{(Eq. 12.1.3-1)}$$

F_p is not required to be taken as greater than

$$F_p = 1.6\, S_{DS} I_p W_p \qquad \textbf{(Eq. 12.1.3-2)}$$

and F_p shall not be taken as less than

$$F_p = 0.3\, S_{DS} I_p W_p \qquad \textbf{(Eq. 12.1.3-3)}$$

where

F_p = seismic design force centered at the component's center of gravity and distributed relative to component's mass distribution

S_{DS} = spectral acceleration, short period, as determined from Section 9.4.4

a_p = component amplification factor that varies from 1.00 to 2.50 (select appropriate value from Table 12.2.2 or 12.3.2)

I_p = component importance factor that varies from 1.00 to 1.50 (see Section 12.1.5)

W_p = component operating weight

R_p = component response modification factor that varies from 1.50 to 5.00 (select appropriate value from Tables 12.2.2 or 12.3.2)

z = height in structure of point of attachment of component with respect to the base. For items at or below the base, z shall be taken as 0. The value of z/h need not exceed 1.0

h = average roof height of structure with respect to the base

The force (F_p) shall be applied independently, longitudinally and laterally in combination with service loads associated with the component. Combine horizontal and vertical load effects as indicated in Section 10.4 substituting F_p for the term Q_E. The reliability/redundancy factor, ρ, is permitted to be taken equal to 1.

When positive and negative wind loads exceed F_p for nonbearing exterior wall, these wind loads shall govern the design. Similarly, when the building code horizontal loads exceed F_p for interior partitions, these building code loads shall govern the design.

12.1.4 **Seismic Relative Displacements.** Seismic relative displacements (D_p) shall be determined in accordance with the following equations:

For two connection points on the same Structure A or the same structural system, one at a height h_x and the other at a height h_y, D_p shall be determined as

$$D_p = \delta_{xA} - \delta_{yA} \qquad \textbf{(Eq. 12.1.4-1)}$$

D_p *is* not required to be taken as greater than

$$D_p = (h_x - h_y)\, \Delta_{aA} / h_{sx} \qquad \textbf{(Eq. 12.1.4-2)}$$

For two connection points on separate Structures A or B or separate structural systems, one at a height h_x and the other at a height h_y, D_p shall be determined as

$$D_p = |\delta_{xA}| + |\delta_{yB}| \qquad \textbf{(Eq. 12.1.4-3)}$$

D_p *is* not required to be taken as greater than

$$D_p = h_x \ \Delta_{aA} / h_{sx} + h_y \Delta_{aB} / h_{sx} \qquad \textbf{(Eq. 12.1.4-4)}$$

where

D_p = relative seismic displacement that the component must be designed to accommodate

δ_{xA} = deflection at building Level x of Structure A, determined by an elastic analysis as defined in Section 10.9.7.1

δ_{yA} = deflection at building Level y of Structure A, determined by an elastic analysis as defined in Section 10.9.7.1

δ_{yB} = deflection at building Level y of Structure B, determined by an elastic analysis as defined in Section 10.9.7.1

h_x = height of Level x to which upper connection point is attached

h_y = height of Level y to which lower connection point is attached

Δ_{aA} = allowable story drift for Structure A as defined in Table 10.12

Δ_{aB} = allowable story drift for Structure B as defined in Table 10.12

h_{sx} = story height used in the definition of the allowable drift Δ_a in Table 10.12, note that Δ_a/h_{sx} = the drift index

The effects of seismic relative displacements shall be considered in combination with displacements caused by other loads as appropriate.

12.1.5 **Component Importance Factor.** The component importance factor (I_p) shall be selected as follows:

I_p = 1.5 for life safety component required to function after an earthquake (e.g., fire protection sprinkler system)

I_p = 1.5 for component that contains hazardous content

I_p = 1.5 for storage racks in structures open to the public (e.g., warehouse retails stores)

I_p = 1.0 for all other components

In addition, for structures in Occupancy Category IV:

I_p = 1.5 for all components needed for continued operation of the facility or whose failure could impair the continued operation of the facility

12.1.6 **Component Anchorage.** Components shall be anchored in accordance with the following provisions.

12.1.6.1 The force in the connected part shall be determined based on the prescribed forces for the component specified in Section 12.1.3. Where component anchorage is provided by shallow expansion anchors, shallow chemical anchors, or shallow (low deformability) cast-in-place anchors, a value of Rp = 1.5 shall be used in Section 12.1.3 to determine the forces in the connected part.

12.1.6.2 Anchors embedded in concrete or masonry shall be proportioned to carry the least of the following:

a. The design strength of the connected part,

b. 1.3 times the force in the connected part due to the prescribed forces, or

c. The maximum force that can be transferred to the connected part by the component structural system.

12.1.6.3 Determination of forces in anchors shall take into account the expected conditions of installation including eccentricities and prying effects.

12.1.6.4 Determination of force distribution of multiple anchors at one location shall take into account the stiffness of the connected system and its ability to redistribute loads to other anchors in the group beyond yield.

12.1.6.5 Powder driven fasteners shall not be used for tension load applications in Seismic Design Category D unless approved for such loading.

12.1.6.6 The design strength of anchors in concrete shall be determined in accordance with the provisions of Section 11.2.

12.1.7 **Construction Documents.** Construction documents shall be prepared to comply with the requirements indicated in Table 12.1.7.

SECTION 12.2
ARCHITECTURAL COMPONENT DESIGN

12.2.1 **General.** Architectural systems, components, or elements (hereinafter referred to as "components") listed in Table 12.2.2 and their attachments shall meet the requirements of Sections 12.2.2 through 12.2.9.

12.2.2 **Architectural Component Forces and Displacements.** Architectural components shall meet the force requirements of Section 12.1.3 and Table 12.2.2.

Components supported by chains or otherwise suspended from the structural system above are not required to meet the lateral seismic force requirements and seismic relative displacement requirements of this Section provided that they cannot be damaged to become a hazard or cannot damage any other component when subject to seismic motion and they have ductile or articulating connections to the structure at the point of attachment. The gravity design load for these items shall be three times their operating load.

12.2.3 **Architectural Component Deformation.** Architectural components that could pose a life safety hazard shall be designed for the seismic relative displacement requirements of Section 12.1.4. Architectural components shall be designed for vertical deflection due to joint rotation of cantilever structural members.

12.2.4 **Exterior Nonstructural Wall Elements and Connections.**

12.2.4.1 **General.** Exterior nonstructural wall panels or elements that are attached to or enclose the structure shall be designed to resist the forces in accordance with Eq. 12.1.3-1 or 12.1.3-2, and shall accommodate movements of the structure resulting from response to the design basis ground motion, D_p, or temperature changes. Such elements shall be supported by means of positive and direct structural supports or by mechanical connections and fasteners. The support system shall be designed in accordance with the following:

a. Connections and panel joints shall allow for the story drift caused by relative seismic displacements (D_p) determined in Section 12.1.4, or 13 mm, whichever is greatest.

b. Connections to permit movement in the plane of the panel for story drift shall be sliding connections using slotted or oversize holes, connections that permit movement by bending of steel, or other connections that provide equivalent sliding or ductile capacity.

c. The connecting member itself shall have sufficient ductility and rotation capacity to preclude fracture of the concrete or brittle failures at or near welds.

d. All fasteners in the connecting system such as bolts, inserts, welds, and dowels and the body of the connectors shall be designed for the force (F_p) determined by Eq. 12.1.3-2 with values of R_p and a_p taken from Table 12.2.2 applied at the center of mass of the panel.

e. Anchorage using flat straps embedded in concrete or masonry shall be attached to or hooked around reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel or to assure that pullout of anchorage is not the initial failure mechanism.

TABLE 12.1.7:
CONSTRUCTION DOCUMENTS

Component Description	Section Reference		Required Seismic Design Categories
	Quality Assurance	Design	
Exterior wall panels, including anchorage	15.2.4.8 No. 1	12.2.4	D
Suspended ceiling system, including anchorage	15.2.4.8 No. 2	12.2.6	D
Access floors, including anchorage	15.2.4.8 No. 2	12.2.7	D
Steel storage racks, including anchorage	15.2.4.8 No. 2	12.2.9	D
Glass in glazed curtain walls, glazed storefronts, and interior glazed partitions, including anchorage	15.2.4.8 No. 3	12.2.10	D
HVAC ductwork containing hazardous materials, including anchorage	15.2.4.9 No. 3	12.3.10	C, D
Piping systems and mechanical units containing flammable, combustible, or highly toxic materials	15.2.4.9 No. 2	12.3.11 12.3.12 12.3.13	C, D
Anchorage of electrical equipment for emergency or standby power systems	15.2.4.9 No. 1	12.3.14	C, D
Project-specific requirements for mechanical and electrical components and their anchorage	15.2.5.5	12.3	C, D

TABLE 12.2.2:
ARCHITECTURAL COMPONENT COEFFICIENTS

Architectural Component or Element	a_p [a]	R_p [b]
Interior Nonstructural Walls and Partitions		
Plain (unreinforced) masonry walls	1	1.5
All other walls and partitions	1	2.5
Cantilever Elements (Unbraced or Braced to Structural Frame Below Its Center of Mass)		
Parapets and cantilever interior nonstructural walls	2.5	2.5
Chimneys and stacks when laterally braced or supported by the structural frame	2.5	2.5
Cantilever Elements (Braced to Structural Frame Above Its Center of Mass)		
Parapets	1.0	2.5
Chimneys and stacks	1.0	2.5
Exterior nonstructural walls	1.0[b]	2.5
Exterior Nonstructural Wall Elements and Connections		
Wall element	1	2.5
Body of wall panel connections	1	2.5
Fasteners of the connecting system	1.25	1
Veneer		
Limited deformability elements and attachments	1	2.5
Low deformability elements and attachments	1	2.5
Penthouses (Except when Framed by an Extension of the Building Frame)		
Ceilings	2.5	3.5
All	1	2.5
Cabinets		
Storage cabinets and laboratory equipment	1	2.5
Access Floors		
Special access floors (designed in accordance with Section 12.2.7.2)	1	2.5
All other	1	1.5
Appendages and Ornamentations	2.5	2.5
Signs and Billboards	2.5	2.5
Other Rigid Components		
High deformability elements and attachments	1	3.5
Limited deformability elements and attachments	1	2.5
Low deformability materials and attachments	1	1.5
Other Flexible Components		
High deformability elements and attachments	2.5	3.5
Limited deformability elements and attachments	2.5	2.5
Low deformability materials and attachments	2.5	1.5

[a] A lower value for a_p shall not be used unless justified by detailed dynamic analysis. The value for a_p shall not be less than 1.00. The value of $a_p = 1$ is for equipment generally regarded as rigid and rigidly attached. The value of $a_p = 2.5$ is for equipment generally regarded as flexible or flexibly attached. See Section 9.2 for definitions of rigid and flexible.

[b] Where flexible diaphragms provide lateral support for walls and partitions, the design forces for anchorage to the diaphragm shall be as specified in Section 10.11.

12.2.4.2 **Glass**. Glass in glazed curtain walls and storefronts shall be designed and installed in accordance with Section 12.2.10.

12.2.5 **Out-of-Plane Bending.** Transverse or out of plane bending or deformation of a component or system that is subjected to forces as determined in Section 12.2.2 shall not exceed the deflection capability of the component or system.

12.2.6 **Suspended Ceilings.** Suspended ceilings shall be designed to meet the seismic force requirements of Section 12.2.6.1. In addition, suspended ceilings shall meet the requirements of either industry standard construction as modified in Section 12.2.6.2 or integral construction as specified in Section 12.2.6.3.

12.2.6.1 **Seismic Forces.** Suspended ceilings shall be designed to meet the force requirements of Section 12.1.3.

The weight of the ceiling, W_p, shall include the ceiling grid and panels; light fixtures if attached to, clipped to, or laterally supported by the ceiling grid; and other components which are laterally supported by the ceiling. W_p shall be taken as not less than 20 N/m^2.

The seismic force, F_p, shall be transmitted through the ceiling attachments to the building structural elements or the ceiling-structure boundary.

Design of anchorage and connections shall be in accordance with these provisions.

12.2.6.2 **Industry Standard Construction.** Unless designed in accordance with Section 12.2.6.3, suspended ceilings shall be designed and constructed in accordance with this Section.

12.2.6.2.1 **Seismic Design Category C.** Suspended ceilings in Seismic Design Category C shall be designed and installed in accordance with the CISCA recommendations for seismic Zones 0-2, (Ref. 12-16), except that seismic forces shall be determined in accordance with Sections 12.1.3 and 12.2.6.1.

Sprinkler heads and other penetrations in Seismic Design Category C shall have a minimum of 6 mm clearance on all sides.

12.2.6.2.2 **Seismic Design Category D.** Suspended ceilings in Seismic Design Category D shall be designed and installed in accordance with the CISCA recommendations for seismic Zones 3-4 (Ref. 12-17) and the additional requirements listed in this subsection.

a. A heavy duty T-bar grid system shall be used.

b. The width of the perimeter supporting closure angle shall be not less than 50 mm. In each orthogonal horizontal direction, one end of the ceiling grid shall be attached to the closure angle. The other end in each horizontal direction shall have a 20 mm clearance from the wall and shall rest upon and be free to slide on a closure angle.

c. For ceiling areas exceeding 100 m^2, horizontal restraint of the ceiling to the structural system shall be provided. The tributary areas of the horizontal restraints shall be approximately equal.

Exception: Rigid braces are permitted to be used instead of diagonal splay wires. Braces and attachments to the structural system above shall be adequate to limit relative lateral deflections at point of attachment of ceiling grid to less than 6 mm for the loads prescribed in Section 12.1.3.

d. For ceiling areas exceeding 250 m^2, a seismic separation joint or full height partition that breaks the ceiling up into areas not exceeding 250 m^2 shall be provided unless structural analyses are performed of the ceiling bracing system for the prescribed seismic forces which demonstrate ceiling system penetrations and closure angles provide sufficient clearance to accommodate the additional movement. Each area shall be provided with closure angles in accordance with Item b and horizontal restraints or bracing in accordance with Item c.

e. Except where rigid braces are used to limit lateral deflections, sprinkler heads and other penetrations shall have a 50 mm over-size ring, sleeve, or adapter through the ceiling tile to allow for free movement of at least 25 mm in all horizontal directions. Alternatively, a swing joint that can accommodate 25 mm of ceiling movement in all horizontal directions are permitted to be provided at the top of the sprinkler head extension.

f. Changes in ceiling plan elevation shall be provided with positive bracing.

g. Cable trays and electrical conduits shall be supported independently of the ceiling.

h. Suspended ceilings shall be subject to the special inspection requirements of Section 15.2.4.8.

12.2.6.3 **Integral Ceiling/Sprinkler Construction.** As an alternative to providing large clearances around sprinkler system penetrations through ceiling systems, the sprinkler system and ceiling grid are permitted to be designed and tied together as an integral unit. Such a design shall consider the mass and flexibility of all elements involved, including: ceiling system, sprinkler system, light fixtures, and mechanical (HVAC) appurtenances. The design shall be performed by a registered design professional.

12.2.7 Access Floors.

12.2.7.1 **General.** Access floors shall be designed to meet the force provisions of Section 12.1.3 and the additional provisions of this Section. The weight of the access floor, W_p, shall include the weight of the floor system, 100% of the weight of all equipment fastened to the floor, and 25% of the weight of all equipment supported by, but not fastened to the floor. The seismic force, F_p, shall be transmitted from the top surface of the access floor to the supporting structure.

Overturning effects of equipment fastened to the access floor panels also shall be considered. The ability of "slip on" heads for pedestals shall be evaluated for suitability to transfer overturning effects of equipment.

When checking individual pedestals for overturning effects, the maximum concurrent axial load shall not exceed the portion of W_p assigned to the pedestal under consideration.

12.2.7.2 **Special Access Floors.** Access floors shall be considered to be "special access floors" if they are designed to comply with the following considerations:

1. Connections transmitting seismic loads consist of mechanical fasteners, concrete anchors, welding, or bearing. Design load capacities comply with recognized design codes and/or certified test results.

2. Seismic loads are not transmitted by friction, produced solely by the effects of gravity, powder-actuated fasteners (shot pins), or adhesives.

3. The design analysis of the bracing system includes the destabilizing effects of individual members buckling in compression.

4. Bracing and pedestals are of structural or mechanical shape produced to ASTM specifications that specify minimum mechanical properties. Electrical tubing shall not be used.

5. Floor stringers that are designed to carry axial seismic loads and that are mechanically fastened to the supporting pedestals are used.

12.2.8 Partitions.

12.2.8.1 General. Partitions that are tied to the ceiling and all partitions greater than 1.8 m in height shall be laterally braced to the building structure. Such bracing shall be independent of any ceiling splay bracing. Bracing shall be spaced to limit horizontal deflection at the partition head to be compatible with ceiling deflection requirements as determined in Section 12.2.6 for suspended ceilings and Section 12.2.2 for other systems.

12.2.8.2 Glass. Glass in glazed partitions shall be designed and installed in accordance with Section 12.2.10.

12.2.9 Steel Storage Racks. Steel storage racks sup-ported at the base of the structure shall be designed to meet the force requirements of Chapter 13. Steel storage racks supported above the base of the structure shall be designed to meet the force requirements of Sections 12.1 and 12.2.

12.2.10 Glass in Glazed Curtain Walls, Glazed Storefronts, and Glazed Partitions.

12.2.10.1 General. Glass in glazed curtain walls, glazed storefronts, and glazed partitions shall meet the relative displacement requirement of Eq. 12.2.10.1-1:

$$\Delta_{fallout} \geq 1.25\, D_p\, I \qquad \textbf{(Eq. 12.2.10.1-1)}$$

or 13 mm, whichever is greater; where

$\Delta_{fallout}$ = the relative seismic displacement (drift) causing glass fallout from the curtain wall, storefront wall, or partition (Section 12.2.10.2)

D_p = the relative seismic displacement that the component must be designed to accommodate (Eq. 12.1.4-1). D_p shall be applied over the height of the glass component under consideration

I = the occupancy importance factor (Table 9.5)

Exceptions:

1. Glass with sufficient clearances from its frame such that physical contact between the glass and frame will not occur at the design drift, as demonstrated by Eq. 12.2.10.1-2, shall be exempted from the provisions of Eq. 12.2.10.1-1:

$$D_{clear} \geq 1.25\, D_p \qquad \textbf{(Eq. 12.2.10.1-2)}$$

where

$$D_{clear} = 2c_1 \left(1 + \frac{h_p c_2}{b_p c_1}\right)$$

h_p = the height of the rectangular glass

b_p = the width of the rectangular glass

c_1 = the clearance (gap) between the vertical glass edges and the frame, and

c_2 = the clearance (gap) between the horizontal glass edges and the frame

2. Fully tempered monolithic glass in Occupancy Category I, II or III located no more than 3 m above a walking surface shall be exempted from the provisions of Eq. 12.2.10.1-1.

3. Annealed or heat-strengthened laminated glass in single thickness with interlayer no less than 0.76 mm that is captured mechanically in a wall system glazing pocket, and whose perimeter is secured to the frame by a wet glazed gunable curing elastomeric sealant perimeter bead of 13 mm minimum glass contact width, or other approved anchorage system shall be exempted from the provisions of Eq. 12.2.10.1-1.

12.2.10.2 **Seismic Drift Limits for Glass Components.** $\Delta_{fallout,}$ the drift causing glass fallout from the curtain wall, storefront, or partition shall be determined in accordance with Ref. 12-21, or by engineering analysis.

SECTION 12.3
MECHANICAL AND ELECTRICAL COMPONENT DESIGN

12.3.1 **General.** Attachments and equipment supports for the mechanical and electrical systems, components, or elements (hereinafter referred to as "components") shall meet the requirements of Sections 12.3.2 through 12.3.16.

12.3.2 **Mechanical and Electrical Component Forces and Displacements.** Mechanical and electrical components shall meet the force and seismic relative displacement requirements of Sections 12.1.3, 12.1.4, and Table 12.3.2.

Components supported by chains or otherwise suspended from the structural system above are not required to meet the lateral seismic force requirements and seismic relative displacement requirements of this Section provided they are designed to prevent damage to themselves or causing damage to any other component when subject to seismic motion. Such supports shall have ductile or articulating connections to the structure at the point of attachment. The gravity design load for these items shall be three times their operating load.

12.3.3 **Mechanical and Electrical Component Period.** The fundamental period of the mechanical and electrical component (and its attachment to the building), T_p, shall be determined by the following equation provided that the component and attachment can be reasonably represented analytically by a simple spring and mass single degree of freedom system:

$$T_p = 2\pi\sqrt{\frac{W_p}{K_p g}} \qquad \textbf{(Eq. 12.3.3)}$$

where

T_p = component fundamental period

W_p = component operating weight

g = gravitational acceleration

K_p = stiffness of resilient support system of the component and attachment, determined in terms of load per unit deflection at the center of gravity of the component

Note that consistent units must be used.

Otherwise, determine the fundamental period of the component in seconds *(T_p)* from experimental test data or by a properly substantiated analysis.

12.3.4 **Mechanical and Electrical Component Attachments.** The stiffness of mechanical and electrical component attachments shall be designed such that the load path for the component performs its intended function.

12.3.5 **Component Supports.** Mechanical and electrical component supports and the means by which they are attached to the component shall be designed for the forces determined in Section 12.1.3 and in conformance with Sections 11.1 through 11.4, as appropriate, for the materials comprising the means of attachment. Such supports include structural members, braces, frames, skirts, legs, saddles, pedestals, cables, guys, stays, snubbers, and tethers, as well as element forged or cast as a part of the mechanical or electrical component. If standard or proprietary supports are used, they shall be designed by either load rating (i.e., testing) or for the calculated seismic forces. In addition, the stiffness of the support, when appropriate, shall be designed such that the seismic load path for the component performs its intended function.

Component supports shall be designed to accommodate the seismic relative displacements between points of support determined in accordance with Section 12.1.4.

In addition, the means by which supports are attached to the component, except when integral (i.e., cast or forged), shall be designed to accommodate both the forces and displacements determined in accordance with Sections 12.1.3 and 12.1.4. If the value of I_p = 1.5 for the component, the local region of the support attachment point to the component shall be evaluated for the effect of the load transfer on the component wall.

12.3.6 **Component Certification.** Architectural, mechanical, and electrical components shall comply with the force requirements of Chapter 12. Components designated with an I_p greater than 1.0 in Seismic Design Category C and D shall meet additional requirements of Section 15.2.5.5 and in particular, mechanical and electrical equipment which must remain operable following the design earthquake shall demonstrate operability by shake table testing or experience data.

The manufacturer's certificate of compliance indicating compliance with this Section shall be submitted to the authority having jurisdiction when required by the contract documents or when required by the regulatory agency.

12.3.7 **Utility and Service Lines at Structure Interfaces.** At the interface of adjacent structures or portions of the same structure that may move independently, utility lines shall be provided with adequate flexibility to accommodate the anticipated differential movement between the portions that move independently. Differential displacement calculations shall be determined in accordance with Section 12.1.4.

12.3.8 **Site-Specific Considerations.** The possible interruption of utility service shall be considered in relation to designated seismic systems in Occupancy Category IV. Specific attention shall be given to the vulnerability of underground utilities and utility interfaces between the structure and the ground where the seismic coefficient S_{DS} at the underground utility or at the base of the structure is equal to or greater than 0.33.

TABLE 12.3.2:
MECHANICAL AND ELECTRICAL COMPONENTS SEISMIC COEFFICIENTS

Mechanical and Electrical Component or Element[b]	a_p[a]	R_p
General Mechanical Equipment Boilers and furnaces	1.0	2.5
Pressure vessels on skirts and free-standing stacks	2.5	2.5
Cantilevered chimneys	2.5	2.5
Other	1.0	2.5
Manufacturing and Process Machinery General	1.0	2.5
Conveyors (non-personnel)	2.5	2.5
Piping Systems High deformability elements and attachments	1.0	3.5
Limited deformability elements and attachments	1.0	2.5
Low deformability elements and attachments	1.0	1.5
HVAC Systems Vibration isolated	2.5	2.5
Nonvibration isolated	1.0	2.5
Mounted in-line with ductwork	1.0	2.5
Other	1.0	2.5
Elevator Components	1.0	2.5
Escalator Components	1.0	2.5
Trussed Towers (free-standing or guyed)	2.5	2.5
General Electrical Distribution systems (bus ducts, conduit, cable tray)	2.5	5.0
Equipment	1.0	2.5
Lighting Fixtures	1.0	1.5

[a] A lower value for a_p shall not be used unless justified by detailed dynamic analyses. The value for a_p shall not be less than 1.00. The value of $a_p = 1$ is for equipment generally regarded as rigid or rigidly attached. The value of $a_p = 2.5$ is for equipment generally regarded as flexible or flexibly attached. See Section 9.2 for definitions of rigid and flexible.
[b] Components mounted on vibration isolation systems shall have a bumper restraint or snubber in each horizontal direction. The design force shall be taken as $2F_p$ if the maximum clearance (air gap) between the equipment support frame and restraint is greater than 6 mm. If the maximum clearance is specified on the construction documents to be not greater than 6 mm, the design force may be taken as F_p.

12.3.9 **Storage Tanks Mounted in Structures.** Storage tanks, including their attachments and supports, shall be designed to meet the force requirements of Chapter 13.

12.3.10 **HVAC Ductwork.** Attachments and supports for HVAC ductwork systems shall be designed to meet the force and displacement provisions of Sections 12.1.3 and 12.1.4 and the additional provisions of this Section. In addition to their attachments and supports, ductwork systems designated as having an $I_p = 1.5$ themselves shall be designed to meet the force and displacement provisions of Sections 12.1.3 and 12.1.4 and the additional provisions of this Section. Where HVAC ductwork runs between structures that could displace relative to one another and for seismically isolated structures where the HVAC ductwork crosses the seismic isolation interface, the HVAC ductwork shall be designed to accommodate the seismic relative displacements specified in Section 12.1.4.

Seismic restraints are not required for HVAC ducts with $I_p = 1.0$ if either of the following conditions are met:

a. HVAC ducts are suspended from hangers 300 mm or less in length from the top of the duct to the supporting structure. The hangers shall be detailed to avoid significant bending of the hangers and their attachments,

or

b. HVAC ducts have a cross-sectional area of less than 0.6 m^2.

Equipment items installed in-line with the duct system (e.g., fans, heat exchangers, and humidifiers) weighing more than 350 N shall be supported and laterally braced independent of the duct system and shall meet the force requirements of Section 12.1.3. Appurtenances such as dampers, louvers, and diffusers shall be positively attached with mechanical fasteners. Unbraced piping attached to in-line equipment shall be provided with adequate flexibility to accommodate differential displacements.

12.3.11 **Piping Systems.** Attachments and supports for piping systems shall be designed to meet the force and displacement provisions of Sections 12.1.3 and 12.1.4 and the additional provisions of this Section. In addition to their attachments and supports, piping systems designated as having I_p = 1.5 themselves shall be designed to meet the force and displacement provisions of Sections 12.1.3 and 12.1.4 and the additional provisions of this Section. Where piping systems are attached to structures that could displace relative to one another and for seismically isolated structures where the piping system crosses the seismic isolation interface, the piping system shall be designed to accommodate the seismic relative displacements specified in Section 12.1.4.

Seismic effects that shall be considered in the design of a piping system include the dynamic effects of the piping system, its contents, and, when appropriate, its supports. The interaction between the piping system and the supporting structures, including other mechanical and electrical equipment shall also be considered.

12.3.11.1 **Pressure Piping Systems.** Pressure piping systems designed and constructed in accordance with ASME B31 (Ref. 12-5) shall be deemed to meet the force, displacement, and other provisions of this Section. In lieu of specific force and displacement provisions provided in the ASME B31, the force and displacement provisions of Sections 12.1.3 and 12.1.4 shall be used.

12.3.11.2 **Fire Protection Sprinkler Systems.** Fire protection sprinkler systems designed and constructed in accordance with NFPA 13, (Ref. 12-13) shall be deemed to meet the other requirements of this Section, except the force and displacement requirements of Sections 12.1.3 and 12.1.4 shall be satisfied.

12.3.11.3 **Other Piping Systems.** Piping designated as having an I_p = 1.5 but not designed and constructed in accordance with ASME B31 (Ref. 12-5) or NFPA 13 (Ref. 12-13) shall meet the following:

a. The design strength for seismic loads in combination with other service loads and appropriate environmental effects shall not exceed the following:

1. For piping systems constructed with ductile materials (e.g., steel, aluminum, or copper), 90% of the piping material yield strength.
2. For threaded connections with ductile materials, 70% of the piping

material yield strength.

3. For piping constructed with nonductile materials (e.g., plastic, cast iron, or ceramics), 25% of the piping material minimum specified tensile strength.
4. For threaded connections in piping constructed with non-ductile materials, 20% of the piping material minimum specified tensile strength.

b. Provisions shall be made to mitigate seismic impact for piping components constructed of nonductile materials or in cases where material ductility is reduced (e.g., low temperature applications).

c. Piping shall be investigated to ensure that the piping has adequate flexibility between support attachment points to the structure, ground, other mechanical and electrical equipment, or other piping.

d. Piping shall be investigated to ensure that the interaction effects between it and other piping or constructions are acceptable.

12.3.11.4 Supports and Attachments for Other Piping. Attachments and supports for piping not designed and constructed in accordance with ASME B31 [Ref. 12-5] or NFPA 13 [Ref. 12-13] shall meet the following provisions:

a. Attachments and supports transferring seismic loads shall be constructed of materials suitable for the application and designed and constructed in accordance with a nationally recognized structural code such as, when constructed of steel, SBC 306 or MSS SP-58 (Ref. 12-15).

b. Attachments embedded in concrete shall be suitable for cyclic loads.

c. Rod hangers shall not be used as seismic supports unless the length of the hanger from the supporting structure is 300 mm or less. Rod hangers shall not be constructed in a manner that subjects the rod to bending moments.

d. Seismic supports are not required for:

1. Ductile piping in Seismic Design Category D designated as having an $I_p = 1.5$ and a nominal pipe size of 25 mm or less when provisions are made to protect the piping from impact or to avoid the impact of larger piping or other mechanical equipment.
2. Ductile piping in Seismic Design Category C designated as having an $I_p = 1.5$ and a nominal pipe size of 50 mm or less when provisions are made to protect the piping from impact or to avoid the impact of larger piping or other mechanical equipment.
3. Ductile piping in Seismic Design Category D designated as having an $I_p = 1.0$ and a nominal pipe size of 75 mm or less.
4. Ductile piping in Seismic Design Category A, B, or C designated as having an $I_p = 1.0$ and a nominal pipe size of 150 mm or less.

e. Seismic supports shall be constructed so that support engagement is maintained.

12.3.12 Boilers and Pressure Vessels. Attachments and supports for boilers and pressure vessels shall be designed to meet the force and displacement provisions of Sections 12.1.3 and 12.1.4 and the additional provisions of this Section. In addition to their attachments and supports, boilers and pressure vessels designated as having an $I_p = 1.5$ themselves shall be designed to meet the force and displacement provisions of Sections 12.1.3 and 12.1.4.

The seismic design of a boiler or pressure vessel shall include analysis of the following: the dynamic effects of the boiler or pressure vessel, its contents, and its supports; sloshing of liquid contents; loads from attached components such as piping; and the interaction between the boiler or pressure vessel and its support.

12.3.12.1 **ASME Boilers and Pressure Vessels.** Boilers or pressure vessels designed in accordance with the ASME *Code* (Ref. 12-2) shall be deemed to meet the force, displacement, and other requirements of this Section. In lieu of the specific force and displacement provisions provided in the ASME code, the force and displacement provisions of Sections 12.1.3 and 12.1.4 shall be used.

12.3.12.2 **Other Boilers and Pressure Vessels.** Boilers and pressure vessels designated as having an $I_p = 1.5$ but not constructed in accordance with the provisions of the ASME code (Ref. 12-2) shall meet the following provisions:

a. The design strength for seismic loads in combination with other service loads and appropriate environmental effects shall not exceed the following:

1. For boilers and pressure vessels constructed with ductile materials (e.g., steel, aluminum, or copper), 90% of the material minimum specified yield strength.
2. For threaded connections in boilers or pressure vessels or their supports constructed with ductile materials, 70% of the material minimum specified yield strength.
3. For boilers and pressure vessels constructed with nonductile materials (e.g., plastic, cast iron, or ceramics), 25% of the material minimum specified tensile strength.
4. For threaded connections in boilers or pressure vessels or their supports constructed with nonductile materials, 20% of the material minimum specified tensile strength.

b. Provisions shall be made to mitigate seismic impact for boiler and pressure vessel components constructed of nonductile materials or in cases where material ductility is reduced (e.g., low temperature applications).

c. Boilers and pressure vessels shall be investigated to ensure that the interaction effects between them and other constructions are acceptable.

12.3.12.3 **Supports and Attachments for Other Boilers and Pressure Vessels.** Attachments and supports for boilers and pressure vessels shall meet the following provisions:

a. Attachments and supports transferring seismic loads shall be constructed of materials suitable for the application and designed and constructed in accordance with nationally recognized structural code such as, when constructed of steel, SBC 306.

b. Attachments embedded in concrete shall be suitable for cyclic loads.

c. Seismic supports shall be constructed so that support engagement is maintained.

12.3.13 **Mechanical Equipment, Attachments, and Supports.** Attachments and supports for mechanical equipment not covered in Sections 12.3.8 through 12.3.12 or 12.3.16 shall be designed to meet the force and displacement provisions of Sections 12.1.3 and 12.1.4 and the additional provisions of this Section. In addition to their attachments and supports, such mechanical equipment designated as having an $I_p = 1.5$, itself, shall be designated to meet the force and displacement provisions of Sections 12.1.3 and 12.1.4 and the additional provisions of this Section.

The seismic design of mechanical equipment, attachments, and their supports shall include analysis of the following: the dynamic effects of the equipment, its contents, and, when appropriate, its supports. The interaction between the equipment and the supporting structures, including other mechanical and electrical equipment, shall also be considered.

12.3.13.1 **Mechanical Equipment.** Mechanical equipment designated as having an $I_p = 1.5$ shall meet the following provisions.

a. The design strength for seismic loads in combination with other service loads and appropriate environmental effects shall not exceed the following:

1. For mechanical equipment constructed with ductile materials (e.g., steel, aluminum, or copper), 90% of the equipment material minimum specified yield strength.
2. For threaded connections in equipment constructed with ductile materials, 70% of the material minimum specified yield strength.
3. For mechanical equipment constructed with nonductile materials (e.g., plastic, cast iron, or ceramics), 25% of the equipment material minimum tensile strength.
4. For threaded connections in equipment constructed with nonductile materials, 20% of the material minimum specified yield strength.

b. Provisions shall be made to mitigate seismic impact for equipment components constructed of nonductile materials or in cases where material ductility is reduced (e.g., low temperature applications).

c. The possibility for loadings imposed on the equipment by attached utility or service lines due to differential motions of points of support from separate structures shall be evaluated.

12.3.13.2 **Attachments and Supports for Mechanical Equipment.** Attachments and supports for mechanical equipment shall meet the following provisions:

a. Attachments and supports transferring seismic loads shall be constructed of materials suitable for the application and designed and constructed in accordance with a nationally recognized structural code such as, when constructed of steel, SBC 306.

b. Friction clips shall not be used for anchorage attachment.

c. Expansion anchors shall not be used for mechanical equipment rated over 10 hp (7.45 kW).

Exception: Undercut expansion anchors.

d. Drilled and grouted-in-place anchors for tensile load applications shall use either expansive cement or expansive epoxy grout.

e. Supports shall be specifically evaluated if weak-axis bending of light-gauge support steel is relied on for the seismic load path.

f. Components mounted on vibration isolation systems shall have a bumper restraint or snubber in each horizontal direction. The design force shall be taken as $2F_p$. The intent is to prevent excessive movement and to avoid fracture of support springs and any nonductile components of the isolators.

g. Seismic supports shall be constructed so that support engagement is maintained.

12.3.14 **Electrical Equipment, Attachments, and Supports.** Attachments and supports for electrical equipment shall be designed to meet the force and displacement provisions of Sections 12.1.3 and 12.1.4 and the additional provisions of this Section. In addition to their attachments and supports, electrical equipment designated as having $I_p = 1.5$, itself, shall be designed to meet the force and displacement provisions of Sections 12.1.3 and 12.1.4 and the additional provisions of this Section.

The seismic design of other electrical equipment shall include analysis of the following: the dynamic effects of the equipment, its contents, and when appropriate, its supports. The interaction between the equipment and the supporting structures, including other mechanical and electrical equipment, shall also be considered. Where conduit, cable trays, or similar electrical distribution components are attached to structures that could displace relative to one another and for seismically isolated structures where the conduit or cable trays cross the seismic isolation interface, the conduit or cable trays shall be designed to accommodate the seismic relative displacement specified in Section 12.1.4.

12.3.14.1 **Electrical Equipment.** Electrical equipment designated as having an $I_p = 1.5$ shall meet the following provisions:

a. The design strength for seismic loads in combination with other service loads and appropriate environmental effects shall not exceed the following:

1. For electrical equipment constructed with ductile material (e.g., steel, aluminum, or copper), 90% of the equipment material minimum specified yield strength.
2. For threaded connections in equipment constructed with ductile materials, 70% of the material minimum specified yield strength.
3. For electrical equipment constructed with nonductile materials (e.g., plastic, cast iron, or ceramics), 25% of the equipment material minimum tensile strength.
4. For threaded connections in equipment constructed with nonductile materials, 20% of the material minimum specified yield strength.

b. Provisions shall be made to mitigate seismic impact for equipment components constructed of nonductile materials or in cases where material ductility is reduced (e.g., low-temperature applications).

c. The possibility for loadings imposed on the equipment by attached utility or service lines due to differential motion of points of support from separate structures shall be evaluated.

d. Batteries on racks shall have wraparound restraints to ensure that the batteries will not fall off the rack. Spacers shall be used between restraints and cells to prevent damage to cases. Racks shall be evaluated for sufficient lateral and longitudinal load capacity.

e. Internal coils of dry type transformers shall be positively attached to their supporting substructure within the transformer enclosure.

f. Slide out components in electrical control panels shall have a latching mechanism to hold contents in place.

g. Structural design of electrical cabinets shall be in conformance with standards of the industry that are acceptable to the authority having jurisdiction. Large cutouts in the lower shear panel shall be specifically evaluated if an evaluation is not provided by the manufacturer.

h. The attachment of additional items weighing more than 450 N shall be specifically evaluated if not provided by the manufacturer.

12.3.14.2 **Attachments and Supports for Electrical Equipment.** Attachments and supports for electrical equipment shall meet the following provisions:

a. Attachments and supports transferring seismic loads shall be constructed of materials suitable for the application and designed and constructed in accordance with a nationally recognized structural code such as, when constructed of steel, SBC 306.

b. Friction clips shall not be used for anchorage attachment.

c. Oversized washers shall be used at bolted connections through the base sheet metal if the base is not reinforced with stiffeners.

d. Supports shall be specifically evaluated if weak-axis bending of light-gauge support steel is relied on for the seismic load path.

e. The supports for linear electrical equipment such as cable trays, conduit, and bus ducts shall be designed to meet the force and displacement provisions of Sections 12.1.3 and 12.1.4 only if any of the following situations apply:

- Supports are cantilevered up from the floor.
- Supports include bracing to limit deflection,
- Supports are constructed as rigid welded frames,
- Attachments into concrete utilize nonexpanding insets, shot pins, or cast iron embedments, or
- Attachments utilize spot welds, plug welds, or minimum size welds as defined by SBC 306.

f. Components mounted on vibration isolation systems shall have a bumper restraint or snubber in each horizontal direction, and vertical restraints shall be provided where required to resist overturning. Isolator housings and restraints shall not be constructed of cast iron or other materials with limited ductility. (See additional design force requirements in Table 12.3.2.) A visco-elastic pad or similar material of appropriate thickness shall be used between the bumper and equipment item to limit the impact load.

12.3.15 **Alternative Seismic Qualification Methods.** As an alternative to the analysis methods implicit in the design methodology described above, equipment testing is an acceptable method to determine seismic capacity. Thus, adaptation of a nationally recognized standard for qualification by testing that is acceptable to the authority having jurisdiction is an acceptable alternative, so long as the equipment seismic capacity equals or exceeds the demand expressed in Sections 12.1.3 and 12.1.4.

12.3.16 Elevator Design Requirements.

12.3.16.1 **Reference Document.** Elevators shall meet the force and displacement provisions of Section 12.3.2 unless exempted by either Section 9.1.2.1 or Section 12.1. Elevators designed in accordance with the seismic provisions of (Ref. 12-1) shall be deemed to meet the seismic force requirements of this Section, except as modified below.

12.3.16.2 **Elevators and Hoistway Structural System.** Elevators and hoistway structural systems shall be designed to meet the force and displacement provisions of Sections 12.1.3 and 12.1.4.

12.3.16.3 **Elevator Machinery and Controller Supports and Attachments.** Elevator machinery and controller supports and attachments shall be designed to meet with the force and displacement provisions of Sections 12.1.3 and 12.1.4.

12.3.16.4 **Seismic Controls.** Seismic switches shall be provided for all elevators addressed by Section 12.3.16.1 including those meeting the requirements of the ASME reference, provided they operate with a speed of 46 m/min or greater.

Seismic switches shall provide an electrical signal indicating that structural motions are of such a magnitude that the operation of elevators may be impaired. Upon activation of the seismic switch, elevator operations shall conform to provisions of (Ref. 12-1) except as noted below. The seismic switch shall be located at or above the highest floor serviced by the elevators. The seismic switch shall have two horizontal perpendicular axes of sensitivity. Its trigger level shall be set to 30% of the acceleration of gravity.

In facilities where the loss of the use of an elevator is a life safety issue, the elevator shall only be used after the seismic switch has triggered provided that:

1. The elevator shall operate no faster than the service speed,
2. Before the elevator is occupied, it is operated from top to bottom and back to top to verify that it is operable, and
3. The individual putting the elevator back in service shall ride the elevator from top to bottom and back to top to verify acceptable performance.

12.3.16.5 **Retainer Plates.** Retainer plates are required at the top and bottom of the car and counterweight.

CHAPTER 13
SEISMIC DESIGN REQUIREMENTS FOR NONBUILDING STRUCTURAL

SECTION 13.1
GENERAL

13.1.1 **Nonbuilding Structures.** Nonbuilding structures include all self-supporting structures that carry gravity loads and that may be required to resist the effects of earthquake, with the exception of building structures specifically excluded in Section 9.1.2, and other nonbuilding structures where specific seismic provisions have yet to be developed in Chapter 13. Nonbuilding structures supported by the earth or supported by other structures shall be designed and detailed to resist the minimum lateral forces specified in this section. Design shall conform to the applicable provisions of other sections as modified by this section. Foundation design shall comply with the requirements of Sections 10.1.1, 10.13 and Chapter 11.

13.1.2 **Design.** The design of nonbuilding structures shall provide sufficient stiffness, strength, and ductility consistent with the requirements specified herein for buildings to resist the effects of seismic ground motions as represented by these design forces:

a. Applicable strength and other design criteria shall be obtained from other portions of the seismic provisions of this Code or its reference documents.

b. When applicable strength and other design criteria are not contained in, or referenced by the seismic provisions of this Code, such criteria shall be obtained from reference documents. Where reference documents define acceptance criteria in terms of allowable stresses as opposed to strength, the design seismic forces shall be obtained from this section and used in combination with other loads as specified in Section 2.4 of this Code and used directly with allowable stresses specified in the reference documents. Detailing shall be in accordance with the reference documents.

13.1.3 **Structural Analysis Procedure Selection.** Structural analysis procedures for nonbuilding structures that are similar to buildings shall be selected in accordance with Section 10.6.

Nonbuilding structures that are not similar to buildings shall be designed using either the equivalent lateral force procedure in accordance with Section 10.9, the modal analysis procedure in accordance with Section 10.10. The linear response history analysis and the nonlinear response history analysis as per Section 10.14, or the procedure prescribed in the specific reference document.

SECTION 13.2
REFERENCE STANDARDS

13.2.1 **Consensus Standards.** The cited references (Ref. 13.2-1 through 13.2-22) are consensus standards and are to be considered part of the requirements of Chapters 9 through 12 to the extent referred to in Chapter 13.

13.2.2 **Accepted Standards.** The cited references (Ref. 13.2-23 through 13.2-31) are standards developed within the industry and represent acceptable procedures for design and construction.

13.2.3 **Industry Design Standards and Recommended Practice.** Table 13.2.3 is a cross-reference of consensus standards/accepted standards and the applicable nonbuilding structures.

SECTION 13.3
NONBUILDING STRUCTURES SUPPORTED BY OTHER STRUCTURES

Where nonbuilding structures identified in Table 13.4-2 are supported by other structures, and the nonbuilding structures are not part of the primary seismic force-resisting system, one of the following methods shall be used.

13.3.1 **Less Than 25 % Combined Weight Condition.** For the condition where the weight of the nonbuilding structure is less than 25 % of the combined weight of the nonbuilding structure and supporting structure, the design seismic forces of the nonbuilding structure shall be determined in accordance with Chapter 12 where the values of R_p and a_p shall be determined in accordance to Section 12.1. The supporting structure shall be designed in accordance with the requirements of Chapter 10 or 13.5 as appropriate with the weight of the nonbuilding structure considered in the determination of the seismic weight, W.

13.3.2 **Greater than or Equal to 25 % Combined Weight Condition.** For the condition where the weight of the nonbuilding structure is equal to or greater than 25 percent of the combined weight of the nonbuilding structure and supporting structure, an analysis combining the structural characteristics of both the nonbuilding structure and the supporting structures shall be performed to determine the seismic design forces as follows:

1. Where the nonbuilding structure has rigid component dynamic characteristics (as defined in Section 13.4.2), the nonbuilding structure shall be considered a rigid element with appropriate distribution of its seismic weight. The supporting structure shall be designed in accordance with the requirements of Chapter 10 or 13.5 as appropriate, and the R value of the combined system shall be permitted to be taken as the R value of the supporting structural system. The nonbuilding structure and attachments shall be designed for the forces using the procedures of Chapter 12 where the value of R_p shall be taken as equal to the R value of the nonbuilding structure as set forth in Table 13.4-2 and a_p shall be taken as 1.0.

2. Where the nonbuilding structure has non-rigid characteristics (as defined in Section 13.4.2), the nonbuilding structure and supporting structure shall be modeled together in a combined model with appropriate stiffness and seismic weight distributions. The combined structure shall be designed in accordance with Section 13.5 with the R value of the combined system taken as the R value of the nonbuilding structure with a maximum value of 3.0. The nonbuilding structure and attachments shall be designed for the forces determined for the nonbuilding structure in the combined analysis.

TABLE 13.2.3:
STANDARDS, INDUSTRY STANDARDS, AND REFERENCES

Application	Reference
Steel Storage Racks	RMI [25]
Piers and Wharves	NAVFAC R-939 [27], NAVFAC DM-25.1 [28]
Welded Steel Tanks for Water Storage	ACI 371R [4], ANSI/AWWA D100 [15], Army TM 5-809-10/ NAVFAC P-355/ Air Force AFM 88-3 Chapter 13 [29]
Welded Steel and Aluminum Tanks for Petroleum and Petrochemical Storage	API 620, 9th Edition, Addendum 3 [6], API 650, 10th Edition, Addendum 1 [7], ANSI/API 653, 2nd Edition, Addendum 4 [8], ASME B96.1 [14], Army TM 5-809-10/ NAVFAC P-355/ Air Force AFM 88-3 Chapter 13 [29]
Bolted Steel Tanks for Water Storage	ANSI/AWWA D103 [16], Army TM 5-809-10/ NAVFAC P-355/ Air Force AFM 88-3 Chapter 13 [29]
Bolted Steel Tanks for Petroleum and Petrochemical Storage	API Specification 12B, 14th Edition [10], Army TM 5-809-10/ NAVFAC P-355/ Air Force AFM 88-3 Chapter 13 [29]
Concrete Tanks for Water Storage	ACI 350.3/350.3R [3], ANSI/AWWA D110 [17], ANSI/AWWA D115 [18], Army TM 5-809-10/ NAVFAC P-355/ Air Force AFM 88-3 Chapter 13 [29]
Pressure Vessels	ASME [11]
Refrigerated Liquids Storage:	
Liquid Oxygen, Nitrogen, and Argon	ANSI/NFPA 50 [30], GGA [31]
Liquefied Natural Gas (LNG)	ANSI/NFPA 59A [22], DOT Title 49CFR Part 193 [26]
LPG (Propane, Butane, etc.)	ANSI/API 2510, 7th Edition [9], ANSI/NFPA 30 [19], ANSI/NFPA 58 [20], ANSI/NFPA 59 [21]
Ammonia	ANSI K61.1 [5]
Concrete Silos and Stacking Tubes	ACI 313 [2]
Impoundment Dikes and Walls:	
Hazardous Materials	ANSI K61.1 [5]
Flammable Materials	ANSI/NFPA 30 [19]
Liquefied Natural Gas	ANSI/NFPA 59A [22], DOT Title 49CFR Part 193 [26]
Gas Transmission and Distribution Piping Systems	ASME B31.8 [13]
Cast-in-Place Concrete Stacks and Chimneys	ACI 307 [1]
Steel Stacks and Chimneys	ASME STS-1 [12]
Guyed Steel Stacks and Chimneys	ASME STS-1 [12]
Brick Masonry Liners for Stacks and Chimneys	ASTM C1298 [24]
Amusement Structures	ASTM F1159 [23]

13.3.3 Architectural, Mechanical, and Electrical Components. Architectural, mechanical, and electrical components supported by nonbuilding structures shall be designed in accordance with Chapter 12 of this Code.

SECTION 13.4
STRUCTURAL DESIGN REQUIREMENTS

13.4.1 Design Basis. Nonbuilding structures having specific seismic design criteria established in reference documents shall be designed using the standards as amended herein. When reference documents are not cited herein, nonbuilding structures shall be designed in compliance with Sections 13.5 and 13.6 to resist minimum seismic lateral forces that are not less than the requirements of Section 10.9 with the following additions and exceptions:

1. The basic seismic-force-resisting system shall be selected as follows:

a. For nonbuilding structures similar to buildings, a system shall be selected from among the types indicated in Table 10.2 or Table 13.4-1 subject to the system limitations and height limits, based on seismic design category indicated in the table. The appropriate values of R, Ω_o, and C_d indicated in 13.4-1 shall be used in determining the base shear, element design forces, and design story drift as indicated in this Code.

b. For nonbuilding structures not similar to buildings, a system shall be selected from among the types indicated in Table 13.4-2 subject to the system limitations and height limits, based on seismic design category indicated in the table. The appropriate values of R, Ω_o, and C_d indicated in Table 13.4-2 shall be used in determining the base shear, element design forces, and design story drift as indicated in this Code.

c. Where neither Table 13.4-1 nor Table 13.4-2 contains an appropriate entry, applicable strength and other design criteria shall be obtained from a reference document that is applicable to the specific type of nonbuilding structure. Design and detailing requirements shall comply with the reference document.

2. For nonbuilding systems that have an R value provided in Table 13.4-2, the seismic response coefficient (C_s) shall not be taken less than:

$$C_s = 0.03 \qquad \textbf{(Eq. 13.4-1)}$$

Exception:
Tanks and vessels that are designed to the following reference documents AWWA 13.2-16, D103-97, Appendix E and Appendix L as modified by this Code, shall be subject to the larger of the minimum base shear values defined by the reference document or the following equation:

$$C_s = 0.01 \qquad \textbf{(Eq. 13.4-2)}$$

Minimum base shear requirements need not apply to the convective (sloshing) component of liquid in tanks.

3. The importance factor, I, shall be as set forth in Section 13.4.1.1.

4. The vertical distribution of the lateral seismic forces in nonbuilding structures covered by this section shall be determined:

a. Using the requirements of Section 10.9.4, or

b. Using the procedures of Section 10.10, or

c. In accordance with reference document applicable to the specific nonbuilding structure.

5. For nonbuilding structural systems containing liquids, gases, and granular solids supported at the base as defined in Section 13.7.1, the minimum seismic design force shall not be less than that required by the reference document for the specific system.

6. Irregular structures per Section 10.3.2 at sites where the S_{DS} is greater than or equal to 0.50 and that cannot be modeled as a single mass shall use the procedures of Section 10.10.

7. Where a reference document provides a basis for the earthquake resistant design of a particular type of nonbuilding structure covered by Chapter 13, such a standard shall not be used unless the following limitations are met:

a. The seismic ground accelerations, and seismic coefficients, shall be in conformance with the requirements of Section 9.4.

b. The values for total lateral force and total base overturning moment used in design shall not be less than 80 percent of the base shear value and overturning moment, each adjusted for the effects of soil structure interaction that is obtained from a substantial analysis using reference documents.

8. The base shear is permitted to be reduced in accordance with Section 10.14 to account for the effects of soil-structure interaction. In no case shall the reduced base shear be less than 0.7V.

9. Unless otherwise noted in Chapter 13, the effects on the nonbuilding structure due to gravity loads and seismic forces shall be combined in accordance with the factored load combinations as presented in Section 2.3.

10. Where specifically required by Chapter 13, the design seismic force on nonbuilding structures shall be as defined in Section 10.4.1.

13.4.1.1 **Importance Factor.** The importance factor (I) and Occupancy Category for nonbuilding structures are based on the relative hazard of the contents and the function. The value of I shall be the largest value determined by the following:

a. Applicable reference document listed in Section 13.2.

b. The largest value as selected from Table 9.5.

c. As specified elsewhere in Chapter 13.

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

$$V = 0.30\ S_{DS}\ WI \qquad \textbf{(Eq. 13.4-3)}$$

Where

V = the total design lateral seismic base shear force applied to a nonbuilding structure

S_{DS} = the site design response acceleration as determined from Section 9.4.4

W = nonbuilding structure operating weight

I = the importance factor determined in accordance with Section 13.4.1.1.

The force shall be distributed with height in accordance with Section 10.9.4.

13.4.3 **Loads.** The weight W for nonbuilding structures shall include all dead load as defined for structures in Section 10.7: For purposes of calculating design seismic forces in nonbuilding structures, W also shall include all normal operating contents for items such as tanks, vessels, bins, hoppers, and the contents of piping.

13.4.4 **Fundamental Period.** The fundamental period of the nonbuilding structure shall be determined using the structural properties and deformation characteristics of the resisting elements in a properly substantiated analysis as indicated in Section 10.9.3. Alternatively, the fundamental period T may be computed from the following equation:

$$T = 2\pi\sqrt{\frac{\sum_{i=1}^{n} w_i \delta_i^2}{g\sum_{i=1}^{n} f_i \delta_i}} \qquad \textbf{(Eq. 13.4-4)}$$

The values of f_i represent any lateral force distribution in accordance with the principles of structural mechanics. The elastic deflections, δ_i, shall be calculated using the applied lateral forces, f_i.

Equations 10.9.3.2-1, 10.9.3.2-1a, 10.9.3.2-2 and 10.9.3.2-3 shall not be used for determining the period of a nonbuilding structure.

13.4.5 **Drift Limitations.** The drift limitations of Section 10.12 need not apply to nonbuilding structures if a rational analysis indicates they can be exceeded without adversely affecting structural stability or attached or interconnected components and elements such as walkways and piping. P-delta effects shall be considered when critical to the function or stability of the structure.

13.4.6 **Materials Requirements.** The requirements regarding specific materials in Chapter 11 shall be applicable unless specifically exempted in Chapter 13.

13.4.7 **Deflection Limits and Structure Separation.** Deflection limits and structure separation shall be determined in accordance with this Code unless specifically amended in Chapter 13.

SECTION 13.5
NONBUILDING STRUCTURES SIMILAR TO BUILDINGS

13.5.1 **General.** Nonbuilding structures similar to buildings as defined in Section 9.2 shall be designed in accordance with this Code as modified by this section and the specific reference documents.

This general category of nonbuilding structures shall be designed in accordance with the seismic provisions of this Code and 13.4.

The combination of load effects, E, shall be determined in accordance with Section 10.4.

13.5.2 Pipe Racks

13.5.2.1 **Design Basis.** In addition to the provisions of 13.5.1, pipe racks supported at the base of the structure shall be designed to meet the force requirements of Section 10.9 or 10.10.

Displacements of the pipe rack and potential for interaction effects (pounding of the piping system) shall be considered using the amplified deflections obtained from the following equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I} \qquad \textbf{(Eq.13.5-1)}$$

where

C_d = deflection amplification factor in Table 13.4-1.

δ_{xe} = deflections determined using the prescribed seismic design forces of this Code.

I = importance factor determined in accordance with Section 13.4.1.1.

See Section 12.3 for the design of piping systems and their attachments. Friction resulting from gravity loads shall not be considered to provide resistance to seismic forces.

13.5.3 **Steel Storage Racks.** In addition to the provisions of 13.5.1, steel storage racks shall be designed in accordance with the provisions of Sections 13.5.3.1 through 13.5.3.4. Alternatively, steel storage racks shall be permitted to be designed in accordance with the method defined in Section 2.7 "Earthquake Forces" of where the following changes are included:

1. The values of C_a and C_v used shall equal $S_{DS}/2.5$ and S_{D1}, respectively, where S_{DS} and S_{D1} are determined in accordance with Section 9.4.4 of this Code.

2. The importance factor for storage racks in structures open to the public, such as warehouse retail stores, shall be taken equal to 1.5.

3. For storage racks located at or below grade, the value of C_s used shall not be less than 0.14 S_{DS}. For storage racks located above grade, the value of C_s used shall not be less than the value for F_p determined in accordance with Section 12.1.3 of this Code where R_p is taken as equal to R from and a_p is taken as equal to 2.5.

13.5.3.1 **General Requirements.** Steel storage racks shall satisfy the force requirements of this section.

Exception: Steel storage racks supported at the base are permitted to be designed as structures with an R of 4, provided that the seismic requirements of this Code are met. Higher values of R are permitted to be used when the detailing requirements of reference documents listed in 11.1 are met. The importance factor for storage racks in structures open to the public, such as warehouse retail stores, shall be taken equal to 1.5.

13.5.3.2 **Operating Weight.** Steel storage racks shall be designed for each of the following conditions of operating weight W or W_p.

a. Weight of the rack plus every storage level loaded to 67 percent of its rated load capacity.

b. Weight of the rack plus the highest storage level only loaded to 100 percent of its rated load capacity. The design shall consider the actual height of the center of mass of each storage load component.

13.5.3.3 **Vertical Distribution of Seismic Forces.** For all steel storage racks, the vertical distribution of seismic forces shall be as specified in Section 10.9.4 and in accordance with the following:

a. The base shear, V, of the typical structure shall be the base shear of the steel storage rack when loaded in accordance with Section 13.5.3.2.

b. The base of the structure shall be the floor supporting the steel storage rack. Each steel storage level of the rack shall be treated as a level of the structure with heights hi and hx measured from the base of the structure.

c. The factor k shall be permitted to be taken as 1.0.

13.5.3.4 **Seismic Displacements.** Steel storage rack installations shall accommodate the seismic displacement of the storage racks and their contents relative to all adjacent or attached components and elements. The assumed total relative displacement for storage racks shall be not less than 5 percent of the height above the base.

Table 13.4-1:
Seismic Coefficients for Nonbuilding Structures Similar to Buildings.

Nonbuilding Structure Type	R	Ω_o	C_d	Structural System and Height Limits (m)[a]		
				A&B	C	D
Building frame system:						
Special steel concentrically braced frames	5	2	5	NL	NL	50
Ordinary steel concentrically braced frame	4	2	4.5	NL	NL	11
Moment resisting frame system:						
Special steel moment frames	6	3	5.5	NL	NL	NL
Special reinforced concrete moment frames	6	3	5.5	NL	NL	NL
Intermediate steel moment frames	3	3	4	NL	NL	11
Intermediate reinforced concrete moment frames	3	3	4.5	NL	NL	NP
Ordinary moment frames of steel	2.5	3	3	NL	NL	NP[c,d]
Ordinary reinforced concrete moment frames	2	3	2.5	NL	NP	NP

Notes:
a NL = no limit and NP = not permitted. Height shall be measured from the base.
b Steel ordinary braced frames are permitted in pipe racks up to 20 m.
c Steel ordinary moment frames and intermediate moment frames are permitted in pipe racks up to a height of 20 m where the moment joints of field connections are constructed of bolted end plates.
d Steel ordinary moment frames and intermediate moment frames are permitted in pipe racks up to a height of 11 m.

Table 13.4-2:
Seismic Coefficients for Nonbuilding Structures NOT Similar to Buildings.

Nonbuilding Structure Type	R	Ω_o	C_d	Structural System and Height Limits (m)[a]		
				A&B	C	D
Steel Storage Racks	3	2	3.5	NL	NL	NL
Elevated tanks, vessels, bins, or hoppers:						
On symmetrically braced legs	2	2^b	2.5	NL	NL	50
On unbraced legs or asymmetrically braced legs	1.5	2^b	2.5	NL	NL	30
Single pedestal or skirt supported	1.5	2^b	2	NL	NL	NL
- welded steel	1.5	2^b	2	NL	NL	NL
- prestressed or reinforced concrete	1.5	2^b	2	NL	NL	NL
Horizontal, saddle supported welded steel vessels.	2	2^b	2.5	NL	NL	NL
Tanks or vessels supported on structural towers similar to buildings	Use values for the appropriate structure type in the categories for building frame systems and moment resisting frame systems listed in Table 13.4-2.					
Flat bottom ground supported tanks:						
Steel or fiber-reinforced plastics:						
Mechanically anchored	2	2^b	2.5	NL	NL	NL
Self-anchored	2	2^b	2	NL	NL	NL
Reinforced or prestressed concrete:						
Reinforced nonsliding base	1.5	2^b	2	NL	NL	NL
Anchored flexible base	2.5	2^b	2	NL	NL	NL
Unanchored and unconstrained flexible base	1	1.5^b	1.5	NL	NL	NL
All other	1	1.5^b	1.5	NL	NL	NL
Cast-in-place concrete soils, stacks, and chimneys having walls continuous to the foundation	2	1.75	3	NL	NL	NL
All other reinforced masonry structures not similar to buildings	1.5	2	2.5	NL	NL	NL
All other nonreinforced masonry structures not similar to buildings	0.8	2	1.5	NL	NL	15
All other steel and reinforced concrete distribution mass cantilever structures not covered herein including stacks, chimneys, soils, and skirt-supported vertical vessels that are not similar to buildings	2	2	2.5	NL	NL	NL

Table 13.4-2:
Seismic Coefficients for Nonbuilding Structures NOT Similar to Buildings (Contd…..).

Nonbuilding Structure Type		R	Ω_o	C_d	Structural System and Height Limits (m)[a]		
					A&B	C	D
Trussed towers (freestanding or guyed), guyed stacks and chimneys		2	2	2.5	NL	NL	NL
Cooling towers:							
Concrete or steel		2.5	1.75	3	NL	NL	NL
Telecommunication towers:							
Truss:	Steel	2	1.5	3	NL	NL	NL
Pole:	Steel	1	1.5	1.5	NL	NL	NL
	Concrete	1	1.5	1.5	NL	NL	NL
Frame:	Steel	2	1.5	1.5	NL	NL	NL
	Concrete	1.5	1.5	1.5	NL	NL	NL
Amusement structure and monuments		1.5	2	2	NL	NL	NL
Inverted pendulum type structures (except elevated tanks, vessels, bins and hoppers)		1.5	2	2	NL	NL	NL
Signs and billboards		2.5	1.75	3	NL	NL	NL
All other self-supporting structures, tanks, or vessels not covered above or by reference standards that are similar to buildings.		1	2	2.5	NL	NL	15

Notes:
a NL = no limit and NP = not permitted. Height shall be measured from the base.
b See Section 13.7.3 a. for the application of the over-strength factors, Ω_o , for tank and vessels.

13.5.4 Electrical Power Generating Facilities

13.5.4.1 **General.** Electrical power generating facilities are power plants that generate electricity by steam turbines, combustion turbines, diesel generators, or similar turbo machinery.

13.5.4.2 **Design Basis.** In addition to the provisions of 13.5.1, electrical power generating facilities shall be designed using this Code and the appropriate factors contained in Section 13.4.

13.5.5 Structural Towers for Tanks and Vessels

13.5.5.1 **General.** In addition to the provisions of 13.5.1, structural towers which support tanks and vessels shall be designed to meet the provisions of Section 13.3. In addition, the following special considerations shall be included:

a. The distribution of the lateral base shear from the tank or vessel onto the supporting structure shall consider the relative stiffness of the tank and resisting structural elements.

b. The distribution of the vertical reactions from the tank or vessel onto the supporting structure shall consider the relative stiffness of the tank and resisting structural elements. When the tank or vessel is supported on grillage beams, the calculated vertical reaction due to weight and overturning shall be increased at least 20 percent to account for non-uniform support. The grillage beam and vessel attachment shall be designed for this increased design value.

c. Seismic displacements of the tank and vessel shall consider the deformation of the support structure when determining P-delta effects or evaluating required clearances to prevent pounding of the tank on the structure.

13.5.6 Piers and Wharves

13.5.6.1 **General.** Piers and wharves are structures located in waterfront areas that project into a body of water or parallel the shore line.

13.5.6.2 **Design Basis.** In addition to the provisions of 13.5.1, piers and wharves that are accessible to the general public, such as cruise ship terminals and piers with retail or commercial offices or restaurants, shall be designed to comply with this Code.

The design shall account for the effects of liquefaction and soil failure collapse mechanisms, as well as consider all applicable marine loading combinations, such as mooring, berthing, wave and current on piers and wharves as required. Structural detailing shall consider the effects of the marine environment.

SECTION 13.6
GENERAL REQUIREMENTS FOR NONBUILDING STRUCTURES NOT SIMILAR TO BUILDINGS

Nonbuilding structures that do not have lateral and vertical seismic force-resisting systems that are similar to buildings shall be designed as modified by this section and the specific reference documents. Loads and load distributions shall not be less demanding than those determined in this Code. The combination of load effects, E, shall be determined in accordance with Section 10.4.
Exception: The redundancy factor, ρ, per Section. 10.3.3 shall be taken as 1.

13.6.1 **Earth-Retaining Structures.** This section applies to all earth-retaining walls. The applied seismic forces shall be determined in accordance with Section 10.13.5.1 with a geotechnical analysis prepared by a registered design professional.
The occupancy category shall be determined by the proximity of the retaining wall to buildings and other structures. If failure of the retaining wall would affect an adjacent structure, the occupancy category shall not be less than that of the adjacent structure, as determined from Table 1.6-1. Earth-retaining walls are permitted to be designed for seismic loads as either yielding or non-yielding walls. Cantilevered reinforced concrete retaining walls shall be assumed to be yielding walls and shall be designed as simple flexural wall elements.

13.6.2 **Stacks and Chimneys.** Stacks and chimneys shall be permitted to be either lined or unlined, and shall be constructed from concrete, steel, or masonry. Steel stacks, concrete stacks, steel chimneys, concrete chimneys, and liners shall be designed to resist seismic lateral forces determined from a substantiated analysis using reference documents. Interaction of the stack or chimney with the liners shall be considered. A minimum separation shall be provided between the liner and chimney equal to C_d times the calculated differential lateral drift.

13.6.3 **Amusement Structures.** Amusement structures are permanently fixed structures constructed primarily for the conveyance and entertainment of people. Amusement structures shall be designed to resist seismic lateral forces determined from a substantiated analysis using reference documents.

13.6.4 **Special Hydraulic Structures.** Special hydraulic structures are structures that are contained inside liquid containing structures. These structures are exposed to liquids on both wall surfaces at the same head elevation under normal operating conditions. Special hydraulic structures are subjected to out-of-plane forces only during an earthquake when the structure is subjected to differential hydrodynamic fluid forces. Examples of special hydraulic structures include: separation walls, baffle walls, weirs, and other similar structures.

13.6.4.1 **Design Basis.** Special hydraulic structures shall be designed for out-of-phase movement of the fluid. Unbalanced forces from the motion of the liquid must be applied simultaneously "in front of" and "behind" these elements.

Structures subject to hydrodynamic pressures induced by earthquakes shall be designed for rigid body and sloshing liquid forces and their own inertia force. The height of sloshing shall be determined and compared to the freeboard height of the structure.

Interior elements, such as baffles or roof supports, also shall be designed for the effects of unbalanced forces and sloshing.

13.6.5 **Secondary Containment Systems.** Secondary containment systems such as impoundment dikes and walls shall meet the requirements of the applicable standards for tanks and vessels and the building official. Secondary containment systems shall be designed to withstand the effects of the maximum considered earthquake ground motion when empty and two-thirds of the maximum considered earthquake ground motion when full including all hydrodynamic forces as determined in accordance with the procedures of Section 9.4. When determined by the risk assessment required by Section 1.5.2 or by the authority having jurisdiction that the site may be subject to aftershocks of the same magnitude as the maximum considered motion, secondary containment systems shall be designed to withstand the effects of the maximum considered earthquake ground motion when full including all hydrodynamic forces as determined in accordance with the procedures of Section 9.4.

13.6.5.1 **Freeboard.** Sloshing of the liquid within the secondary containment area shall be considered in determining the height of the impound. Where the primary containment has not been designed with a reduction in the structure category (i.e., no reduction in importance factor I) as permitted by Section 1.5.2, no freeboard provision is required. Where the primary containment has been designed for a reduced structure category (i.e., importance factor I reduced) as permitted by Section 1.5.2, a minimum freeboard, δ_S, shall be provided where

$$\delta_s = 0.50\, DS_{ac} \qquad \textbf{(Eq. 13.6-1)}$$

where S_{ac} is the spectral acceleration of the convective component and is determined according to the procedures of Section 9.4 using 0.5 percent damping. For circular impoundment dikes, D shall be taken as the diameter of the impoundment dike. For rectangular impoundment dikes, D shall be taken as the longer plan dimension of the impoundment dike.

13.6.6 **Telecommunication Towers.** Self-supporting and guyed telecommunication towers shall be designed to resist seismic lateral forces determined from a substantiated analysis using reference documents.

SECTION 13.7
TANKS AND VESSELS

13.7.1 **General.** This section applies to all tanks, vessels, bins and silos, and similar containers storing liquids, gases, and granular solids supported at the base (hereafter referred to generically as tanks and vessels). Tanks and vessels covered herein include reinforced concrete, prestressed concrete, steel, aluminum, and fiber-reinforced plastic materials. Tanks supported on elevated levels in buildings shall be designed in accordance with Section 13.3.

13.7.2 **Design Basis.** Tanks and vessels storing liquids, gases, and granular solids shall be designed in accordance with this Code and shall be designed to meet the requirements of the applicable reference documents listed in Table 13.2. Resistance to seismic forces shall be determined from a substantiated analysis based on the applicable reference documents listed in Table 13.2.

a. Damping for the convective (sloshing) force component shall be taken as 0.5 percent

b. Impulsive and convective components shall be combined by the direct sum or the square root of the sum of the squares (SRSS) method when the modal periods are separated. If significant modal coupling may occur, the complete quadratic combination (CQC) method shall be used.

c. Vertical earthquake forces shall be considered in accordance with the applicable reference document. If the reference document permits the user the option of including or excluding the vertical earthquake force, to comply with this Code, it shall be included. For tanks and vessels not covered by a reference document, the forces due to the vertical acceleration shall be defined as follows:

(1) Hydrodynamic vertical and lateral forces in tank walls: The increase in hydrostatic pressures due to the vertical excitation of the contained liquid shall correspond to an effective increase in density, γ_L, of the stored liquid equal to 0.2 $S_{DS}I\gamma_L$.

(2) Hydrodynamic hoop forces in cylindrical tank walls: In a cylindrical tank wall, the hoop force per unit height, N_h, at level y from the base, associated with the vertical excitation of the contained liquid, shall be computed in accordance with Equation 15.7-1.

$$N_h = 0.2\, S_{DS}I x\ \gamma_L\, (H_L - y)\, (D_i/2) \qquad \textbf{(Eq. 13.7-1)}$$

where

D_i = inside tank diameter (m)

H_L = liquid height inside the tank (m).

y = distance from base of the tank to level being investigated (m).

γ_L = unit weight of stored liquid (kN/m^3)

(3) Vertical inertia forces in cylindrical and rectangular tank walls: Vertical inertia forces associated with the vertical acceleration of the structure itself shall be taken equal to 0.2 $S_{DS}IW$.

13.7.3 **Strength and Ductility.** Structural components and members that are part of the lateral support system shall be designed to provide the following:

a. Connections and attachments for anchorage and other lateral force-resisting components shall be designed to develop the strength of the anchor (e.g., minimum published yield strength, F_y in direct tension, plastic bending moment), or Ω_o times the calculated element design force. The over-strength provisions of Section 10.4.1, and the Ω_o values tabulated in Table 13.4-2, do not apply to the design of walls, including interior walls, of tanks or vessels.

b. Penetrations, manholes, and openings in shell components shall be designed to maintain the strength and stability of the shell to carry tensile and compressive membrane shell forces.

c. Support towers for tanks and vessels with irregular bracing, unbraced panels, asymmetric bracing, or concentrated masses shall be designed using the provisions of Section 10.3.2 for irregular structures. Support towers using chevron or eccentric braced framing shall comply with the seismic requirements of this Code requirement. Support towers using tension only bracing shall be designed such that the full cross-section of the tension element can yield during overload conditions.

d. In support towers for tanks and vessels, compression struts that resist the reaction forces from tension braces shall be designed to resist the lesser of the yield load of the brace (Ag Fy), or Ω_o times the calculated tension load in the brace.

e. The vessel stiffness relative to the support system (foundation, support tower, skirt, etc.) shall be considered in determining forces in the vessel, the resisting components and the connections.

f. For concrete liquid-containing structures, system ductility, and energy dissipation under unfactored loads shall not be allowed to be achieved by inelastic deformations to such a degree as to jeopardize the serviceability of the structure. Stiffness degradation and energy dissipation shall be allowed to be obtained either through limited microcracking, or by means of lateral force resistance mechanisms that dissipate energy without damaging the structure.

13.7.4 **Flexibility of Piping Attachments.** Design of piping systems connected to tanks and vessels shall consider the potential movement of the connection points during earthquakes and provide sufficient flexibility to avoid release of the product by failure of the piping system. The piping system and supports shall be designed so as not to impart significant mechanical loading on the attachment to the tank or vessel shell. Mechanical devices which add flexibility such as bellows, expansion joints, and other flexible apparatus may be used when they are designed for seismic displacements and defined operating pressure.

Unless otherwise calculated, the minimum displacements in Table 13.7-1 shall be assumed. For attachment points located above the support or foundation elevation, the displacements in Table 13.7-1 shall be increased to account for drift of the tank or vessel relative to the base of support.

The piping system and tank connection shall also be designed to tolerate C_d times the displacements given in Table 13.7-1 without rupture, although permanent deformations and inelastic behavior in the piping supports and tank shell is permitted. For attachment points located above the support or foundation elevation, the displacements in Table 13.7-1 shall be increased to account for drift of the tank or vessel.

The values given in Table 13.7-1 do not include the influence of relative movements of the foundation and piping anchorage points due to foundation movements (e.g., settlement, seismic displacements). The effects of the foundation movements shall be included in the piping system design including the determination of the mechanical loading on the tank or vessel, and the total displacement capacity of the mechanical devices intended to add flexibility.

The anchorage ratio, J, for self-anchored tanks shall comply with the criteria shown in Table 13.7-2 and is defined as:

$$J = \frac{M_{rw}}{D^2 \left(w_t + w_a\right)} \quad \textbf{(Eq. 13.7-2)}$$

where

$$w_t = \frac{W_s}{\pi D} + w_r \quad \textbf{(Eq. 13.7-3)}$$

w_r = roof load acting on the shell in pounds per foot of shell circumference. Only permanent roof loads shall be included. Roof live load shall not be included.

w_a = maximum weight of the tank contents that may be used to resist the shell overturning moment in pounds per foot of shell circumference. Usually consists of an annulus of liquid limited by the bending strength of the tank bottom or annular plate.

M_{rw} = the overturning moment applied at the bottom of the shell due to the seismic design loads in foot-pounds (also known as the ring-wall moment)

D = tank diameter in feet

W_s = total weight of tank shell in pounds

13.7.5 **Anchorage.** Tanks and vessels at grade shall be permitted to be designed without anchorage when they meet the requirements for unanchored tanks in reference documents. Tanks and vessels supported above grade on structural towers or building structures shall be anchored to the supporting structure.

The following special detailing requirements shall apply to steel tank anchor bolts in seismic regions where $S_{DS} > 0.5$, or where the structure is classified as Occupancy Category IV.

a. Hooked anchor bolts (L or J shaped embedded bolts) or other anchorage systems based solely on bond or mechanical friction shall not be used when $S_{DS} \geq 0.33$. Post-installed anchors may be used provided that testing validates their ability to develop yield load in the anchor under cyclic loads in cracked concrete.

b. When anchorage is required, the anchor embedment into the foundation shall be designed to develop the minimum specified yield strength of the anchor.

Table 13.7-1:
Minimum Design Displacements for Piping Attachments.

Condition	Displacements (mm)
Mechanically-Anchored Tanks and Vessels	
Upward vertical displacement relative to support or foundation	25
Downward vertical displacement relative to support or foundation	12
Range of horizontal displacement (radial and tangential) relative to support or foundation	12
Self-anchored Tanks or Vessels (at grade)	
Upward vertical displacement relative to support or foundation	
If designed in accordance with a reference document as modified by this Code:	
Anchorage ratio less than or equal to 0.785 (indicates no uplift)	25
Anchorage ratio greater than 0.785 (indicates uplift)	100
If designed for seismic loads in accordance with this Code but not covered by a reference document:	
For tanks and vessels with a diameter less than 12 m	200
For tanks and vessels with a diameter equal to or greater than 12 m	300
Downward vertical displacement relative to support or foundation	
For tanks with a ring-wall/mat foundation	12
For tanks with a beam foundation	25
Range of horizontal displacement (radial and tangential) relative to support or foundation	50

Table 13.7-2: Anchorage Ratio

J Anchorage Ratio	Criteria
$J < 0.785$	No uplift under the design seismic overturning moment. The tank is self anchored.
$0.785 < J < 1.54$	Tank is uplifting, but the tank is stable for the design load providing the shell compression requirements are satisfied. Tank is self-anchored.
$J > 1.54$	Tank is not stable and shall be mechanical mechanically anchored for the design load.

13.7.6 Ground-Supported Storage Tanks for Liquids

13.7.6.1 General. Ground-supported, flat bottom tanks storing liquids shall be designed to resist the seismic forces calculated using one of the following procedures:

a. The base shear and overturning moment calculated as if tank and the entire contents are a rigid mass system per Section 13.4.2 of this Code, or

b. Tanks or vessels storing liquids in Occupancy Category IV, or with a diameter greater than 6 m shall be designed to consider the hydrodynamic pressures of the liquid in determining the equivalent lateral forces and lateral force distribution per the applicable reference documents listed in

Section 13.2 and the requirements of Section 13.7 of this Code requirement.

c. The force and displacement provisions of Section 13.4 of this Code requirement.

The design of tanks storing liquids shall consider the impulsive and convective (sloshing) effects and their consequences on the tank, foundation, and attached elements. The impulsive component corresponds to the high frequency amplified response to the lateral ground motion of the tank roof, shell, and portion of the contents that moves in unison with the shell. The convective component corresponds to the low frequency amplified response of the contents in the fundamental sloshing mode. Damping for the convective component shall be 0.5 percent for the sloshing liquid unless otherwise defined by the reference document. The following definitions shall apply:

D_i = inside diameter of tank or vessel

H_L = design liquid height inside tank or vessel

L = inside length of a rectangular tank, parallel to the direction of the earthquake force being investigated

N_h = hydrodynamic hoop force per unit height in the wall of a cylindrical tank or vessel

T_c = natural period of the first (convective) mode of sloshing

T_i = fundamental period of the tank structure and impulsive component of the content

V_i = base shear due to impulsive component from weight of tank and contents

V_c = base shear due to the convective component of the effective sloshing mass

y = distance from base of the tank to level being investigated.

γ_L = unit weight of stored liquid

The seismic base shear is the combination of the impulsive and convective components:

$$V = V_i + V_c \qquad \textbf{(Eq. 13.7-4)}$$

where

$$V_i = \frac{S_{ai} W_i}{(R/I)} \qquad \textbf{(Eq. 13.7-5)}$$

$$V_c = \frac{S_{ac} I}{1.5} W_c \qquad \textbf{(Eq. 13.7-6)}$$

S_{ai} = the spectral acceleration as a multiplier of gravity including the site impulsive components at period T_i and 5 percent damping

For $T_i \le T_s$:

$$S_{ai} = S_{DS} \qquad \textbf{(Eq. 13.7-7)}$$

For $T_s < T_i \le T_L$

$$S_{ai} = \frac{S_{D1}}{T_i} \qquad \textbf{(Eq. 13.7-8)}$$

For $T_i > T_L$

$$S_{ai} = \frac{S_{D1} T_L}{T_i^2} \qquad \textbf{(Eq. 13.7-9)}$$

Notes:

a. When a reference document is used in which the spectral acceleration for the tank shell, and the impulsive component of the liquid is independent of T_i, then $S_{ai} = S_{DS}$.

b. Eq. 13.7-8 and Eq. 13.7-9 shall not be less than the minimum values required in Section 13.4.1 Item 2 multiplied by R/I.

c. For tanks in Occupancy Category IV, the value of the importance factor (I) used for freeboard determination only shall be taken as 1.0.

d. For tanks in Occupancy Categories I, II and III, the value of T_L used for freeboard determination shall be permitted to be set equal to 4 seconds. The value of the importance factor (I) used for freeboard determination for tanks in Occupancy Categories I, II and III shall be the value determined from Table 9.5.

e. Impulsive and convective seismic forces for tanks are permitted to be combined using the square root of the sum of the squares (SRSS) method in lieu of the direct sum method shown in Section 13.7.6 and its related subsections.

S_{ac} = the spectral acceleration of the sloshing liquid based on the sloshing period T_c and 0.5 percent damping

For $T_c \leq T_L$:

$$S_{ac} = \frac{1.5\, S_{D1}}{T_c} \qquad \textbf{(Eq. 13.7-10)}$$

For $T_c > T_L$:

$$S_{ac} = \frac{1.5\, S_{D1} T_L}{T_c^2} \qquad \textbf{(Eq. 13.7-11)}$$

where

$$T_c = 2\pi \sqrt{\frac{D}{3.68\, g\, tan\, h\left(\frac{3.68H}{D}\right)}} \qquad \textbf{(Eq. 13.7-12)}$$

and where

D = the tank diameter in meters, H = liquid height in m, and g = acceleration due to gravity in consistent units

W_i = impulsive weight (impulsive component of liquid, roof and equipment, shell, bottom, and internal components)

W_c = the portion of the liquid weight sloshing

13.7.6.1.1 **Distribution of Hydrodynamic and Inertia Forces.** Unless otherwise required by the appropriate reference document listed in Table 13.2, the method given in ACI 530.3-01 may be used to determine the vertical and horizontal distribution of the hydrodynamic and inertia forces on the walls of circular and rectangular tanks.

13.7.6.1.2 **Sloshing.** Sloshing of the stored liquid shall be taken into account in the seismic design of tanks and vessels in accordance with the following provisions:

a. The height of the sloshing wave, δ_s, shall be computed using Eq. 13.7-13 as follows:

$$\delta_s = 0.5\, D_i I S_{ac} \qquad \textbf{(Eq. 13.7-13)}$$

For cylindrical tanks, D_i shall be the inside diameter of the tank; for rectangular tanks, the term D_i shall be replaced by the longer longitudinal plan dimension of the tank, L.

b. The effects of sloshing shall be accommodated by means of one of the following:

1. A minimum freeboard in accordance with Table 13.7-3.
2. A roof and supporting structure designed to contain the sloshing liquid in accordance with subsection c below.
3. For open-top tanks or vessels only, an overflow spillway around the tank or vessel perimeter.

c. If the sloshing is restricted because the freeboard is less than the computed sloshing height, then the roof and supporting structure shall be designed for an equivalent hydrostatic head equal to the computed sloshing height less the freeboard. In addition, the design of the tank shall use the confined portion of the convective (sloshing) mass as an additional impulsive mass.

13.7.6.1.3 Equipment and Attached Piping. Equipment, piping, and walkways or other appurtenances attached to the structure shall be designed to accommodate the displacements imposed by seismic forces. For piping attachments, see Section 13.7.4.

13.7.6.1.4 Internal Components. The attachments of internal equipment and accessories which are attached to the primary liquid or pressure retaining shell or bottom, or provide structural support for major components (e.g., a column supporting the roof rafters) shall be designed for the lateral loads due to the sloshing liquid in addition to the inertial forces by a substantiated analysis method.

13.7.6.1.5 Sliding Resistance. The transfer of the total lateral shear force between the tank or vessel and the subgrade shall be considered:

a. For unanchored flat bottom steel tanks, the overall horizontal seismic shear force is permitted to be resisted by friction between the tank bottom and the foundation or subgrade. Unanchored storage tanks shall be designed such that sliding will not occur when the tank is full of stored product. The maximum calculated seismic base shear, V, shall not exceed:

$$V < W \tan 30^{o} \qquad \textbf{(Eq. 13.7-14)}$$

W shall be determined using the effective weight of the tank, roof, and contents after reduction for coincident vertical earthquake. Lower values of the friction factor shall be used if the design of the tank bottom to supporting foundation does not justify the friction value above (e.g., leak detection membrane beneath the bottom with a lower friction factor, smooth bottoms, etc).

Table 13.7- 3: Minimum Required Freeboard

Value of S_{DS}	Occupancy Category		
	I or II	III	IV
$S_{DS} < 0.167$ g	a	a	δ_s^c
0.167 g $\leq S_{DS} < 0.33$ g	a	a	δ_s^c
0.33 g $\leq S_{DS} < 0.50$ g	a	$0.7\delta_s^b$	δ_s^c
$S_{DS} \geq$ 0.50 g	a	$0.7\delta_s^b$	δ_s^c

a No minimum freeboard is required.

b A freeboard equal to 0.7 δ_s is required unless one of the following alternatives is provided:
1. Secondary containment is provided to control the product spill.
2. The roof and supporting structure are designed to contain the sloshing liquid.

c Freeboard equal to the calculated wave height, δ_s , is required unless one of the following alternatives is provided:
1. Secondary containment is provided to control the product spill.
2. The roof and supporting structure are designed to contain the sloshing liquid.

b. No additional lateral anchorage is required for anchored steel tanks designed in accordance with reference documents.

c. The lateral shear transfer behavior for special tank configurations (e.g., shovel bottoms, highly crowned tank bottoms, tanks on grillage) can be unique and are beyond the scope of this Code.

13.7.6.1.6 **Local Shear Transfer.** Local transfer of the shear from the roof to the wall and the wall of the tank into the base shall be considered. For cylindrical tanks and vessels, the peak local tangential shear per unit length shall be calculated by:

$$V_{max} = \frac{2V}{\pi D} \qquad \textbf{(Eq. 13.7-15)}$$

a. Tangential shear in flat bottom steel tanks shall be transferred through the welded connection to the steel bottom. This transfer mechanism is deemed acceptable for steel tanks designed in accordance with the reference documents where $S_{DS} < 1.0$ g.

b. For concrete tanks with a sliding base where the lateral shear is resisted by friction between the tank wall and the base, the friction coefficient value used for design shall not exceed tan 30 degrees.

c. Fixed-base or hinged-base concrete tanks, transfer the horizontal seismic base shear shared by membrane (tangential) shear and radial shear into the foundation. For anchored flexible-base concrete tanks, the majority of the base shear is resisted by membrane (tangential) shear through the anchoring system with only insignificant vertical bending in the wall. The connection between the wall and floor shall be designed to resist the maximum tangential shear.

13.7.6.1.7 **Pressure Stability.** For steel tanks, the internal pressure from the stored product stiffens thin cylindrical .shell structural elements subjected to membrane compression forces. This stiffening effect may be considered in resisting seismically induced compressive forces if permitted by the reference document or the building official.

13.7.6.1.8 **Shell Support.** Steel tanks resting on concrete ring walls or slabs shall have a uniformly supported annulus under the shell. Uniform support shall be provided by one of the following methods:

- **a.** Shimming and grouting the annulus
- **b.** Using fiberboard or other suitable padding
- **c.** Using butt-welded bottom or annular plates resting directly on the foundation
- **d.** Using closely spaced shims (without structural grout) provided that the localized bearing loads are considered in the tank wall and foundation to prevent local crippling and spalling.

Anchored tanks shall be shimmed and grouted. Local buckling of the steel shell for the peak compressive force due to operating loads and seismic overturning shall be considered.

13.7.6.1.9 **Repair, Alteration, or Reconstruction.** Repairs, modifications, or reconstruction (i.e., cut down and re-erect) of a tank or vessel shall conform to industry standard practice and this Code. For welded steel tanks storing liquids (see ANSI/API 653-01) and the applicable reference document listed in Section 13.2. Tanks that are relocated shall be re-evaluated for the seismic loads for the new site and the requirements of new construction in accordance with the appropriate reference document and this Code.

13.7.7 Water and Water Treatment Tanks and Vessels

13.7.7.1 **Welded Steel.** Welded steel water storage tanks and vessels shall be designed in accordance with the seismic requirements of ANSI/ AWWA D100-96 except that the design input forces shall be modified as follows: The equations for base shear and overturning moment are defined by the following equations for allowable stress design procedures:

For $T_s < T_c \le T_L$

$$V_{ACT} = \frac{S_{DS}}{1.4\left(R/I\right)}\left[\left(W_s + W_r + W_f + W_1\right) + 1.5\frac{T_s}{T_c}W_2\right] \quad \textbf{(Eq 13.7-16)}$$

$$M = \frac{S_{DS}}{1.4\left(R/I\right)}\left[\left(W_s X_s + W_r H_t + W_1 X_1\right) + 1.5\frac{T_s}{T_c}W_2 X_2\right] \quad \textbf{(Eq 13.7-17)}$$

For $T_c > T_L$

$$V_{ACT} = \frac{S_{DS} I}{1.4\,R}\left[\left(W_s + W_r + W_f + W_1\right) + 1.5\frac{T_s T_L}{T_c^2}W_2\right] \quad \textbf{(Eq 13.7-18)}$$

$$M = \frac{S_{DS} I}{1.4R}\left[\left(W_s X_s + W_r H_t + W_1 X_1\right) + 1.5\frac{T_s T_L}{T_c^2}W_2 X_2\right] \quad \textbf{(Eq 13.7-19)}$$

- **a.** Substitute the above equations for Eqs. 13-4 and 13-8 of ANSI/AWWA D100-96 where S_{DS} and T_s are defined in Section 9.4.4, T_L is defined in Section 9.4.5, and R is defined in Table l3.4-2.
- **b.** The hydrodynamic seismic hoop tensile stress is defined in Eqs. 13-20 through 13-25 in ANSI/AWWA D100-96. When using these equations, substitute Eq. 13.7-20 for $\frac{ZI}{R_w}$ directly into the equations.

$$\frac{S_{DS}}{2.5\left[1.4\left(R/I\right)\right]}$$ **(Eq. 13.7-20)**

c. Sloshing height shall be calculated per Section 13.7.6.1.2 instead of Eq. 13-26 of ANSI/AWWA D100-96.

13.7.7.2 **Bolted Steel.** Bolted steel water storage structures shall be designed in accordance with the seismic requirements of ANSI/AWWA D103-97 except that the design input forces shall be modified in the same manner shown in Section 13.7.7.1 of this Code requirement.

13.7.7.3 **Reinforced and Prestressed Concrete.** Reinforced and prestressed concrete tanks shall be designed in accordance with the seismic requirements of ACI 530.3-01 except that the design input forces for allowable stress design procedures shall be modified as follows:

a. For $T_I < T_o$, and $T_I > T_s$ substitute the term $S_a/[1.4\ (R/I)]$ where S_a is defined in Section 9.4.5, Subsections 1, 2, or 3, for the terms in the appropriate equations as shown below:

For $\frac{ZC_I}{\left(R_1/I\right)}$ shear and overturning moment equations of ANSI/AWWA D110-95

For $\frac{ZC_I}{\left(R_W/I\right)}$ shear and overturning moment equations of ANSI/AWWA D115-95

For $\frac{ZSC_i}{\left(R_i/I\right)}$ in the base shear and overturning moment equations of ACI 530.3-01

b. For $T_o \le T_I \le T_s$, substitute the terms $\frac{S_{DS}}{1.4\left(R/I\right)}$ for terms $\frac{ZC_I}{\left(R_i/I\right)}$ and $\frac{ZSC_i}{\left(R_i/I\right)}$

c. For all values of T_c (or T_w), $\frac{ZC_c}{\left(R_c/I\right)}$, $\frac{ZC_c}{\left(R_W/I\right)}$ and $\frac{ZSC_c}{\left(R_c/I\right)}$ are replaced by $\frac{1.5\ S_{D1} I\ T_L}{T_c^2}$ or $\frac{1.5\ S_{DS} I T_S T_L}{T_c^2}$

where

S_a, S_{D1}, S_{DS}, T_o, T_s and T_L are defined in Section 9.4.5 of this Code.

13.7.8 Petrochemical and Industrial Tanks and Vessels Storing Liquids

13.7.8.1 **Welded Steel.** Welded steel petrochemical and industrial tanks and vessels storing liquids shall be designed in accordance with the seismic requirements of API 620-02 and API 650-01 except that the design input forces for allowable stress design procedures shall be modified as follows:

a. When using the equations in Section E.3 of API 650-01, substitute into the equation for overturning moment M (where S_{DS}, T_s and T_L are defined in Section 9.4.5 of this Code requirement. Thus,

In the range $Ts < T_c \le T_L$,

$$M = S_{DS} I[0.24(W_s X_s + W_t H_t + W_1 X_1) + 0.80\ C_2 T_s W_2 X_2]$$

(Eq. 13.7-21)

where

$$C_2 = \frac{0.75\,S}{T_c} \quad \text{and S = 1.0}$$

In the range $T_w > T_L$, and

$$M = S_{DS}I[0.24(W_sX_s + W_tH_t + W_1X_1) + 0.71\ C_2T_sW_2X_2] \qquad \textbf{(Eq. 13.7-22)}$$

where

$$C_2 = \frac{0.8438\,ST_L}{T_c^2} \quad \text{and S = 1.0}$$

13.7.8.2 **Bolted Steel.** Bolted steel tanks used for storage of production liquids. API 12B-95 covers the material, design, and erection requirements for vertical, cylindrical, aboveground bolted tanks in nominal capacities of 100 to 10,000 barrels for production service. Unless required by the building official having jurisdiction, these temporary structures need not be designed for seismic loads. If design for seismic load is required, the loads may be adjusted for the temporary nature of the anticipated service life.

13.7.8.3 **Reinforced and Prestressed Concrete.** Reinforced concrete tanks for the storage of petrochemical and industrial liquids shall be designed in accordance with the force requirements of Section 13.7.7.3.

13.7.9 Ground-Supported Storage Tanks for Granular Materials

13.7.9.1 **General.** The inter-granular behavior of the material shall be considered in determining effective mass and load paths, including the following behaviors:

a. Increased lateral pressure (and the resulting hoop stress) due to loss of the inter-granular friction of the material during the seismic shaking.

b. Increased hoop stresses generated from temperature changes in the shell after the material has been compacted.

c. Inter-granular friction, which can transfer seismic shear directly to the foundation.

13.7.9.2 **Lateral Force Determination.** The lateral forces for tanks and vessels storing granular materials at grade shall be determined by the requirements and accelerations for short period structures (i.e., S_{DS}).

13.7.9.3 **Force Distribution to Shell and Foundation**

13.7.9.3.1 **Increased Lateral Pressure.** The increase in lateral pressure on the tank wall shall be added to the static design lateral pressure but shall not be used in the determination of pressure stability effects on the axial buckling strength of the tank shell.

13.7.9.3.2 **Effective Mass.** A portion of a stored granular mass will act with the shell (the effective mass). The effective mass is related to the physical characteristics of the product, the height-to-diameter (H/D) ratio of the tank, and the intensity of the seismic event. The effective mass shall be used to determine the shear and overturning loads resisted by the tank.

13.7.9.3.3 **Effective Density.** The effective density factor (that part of the total stored mass of product which is accelerated by the seismic event) shall be determined in accordance ACI 313-97.

13.7.9.3.4 **Lateral Sliding.** For granular storage tanks that have a steel bottom and are supported such that friction at the bottom to foundation interface can resist lateral shear loads, no additional anchorage to prevent sliding is required. For tanks without steel bottoms (i.e., the material rests directly on the foundation), shear anchorage shall be provided to prevent sliding.

13.7.9.3.5 **Combined Anchorage Systems.** If separate anchorage systems are used to prevent over-turning and sliding, the relative stiffness of the systems shall be considered in determining the load distribution.

13.7.9.4 **Welded Steel Structures.** Welded steel granular storage structures shall be designed in accordance with the seismic provisions of this Code. Component allowable stresses and materials shall be per ANSI/AWWA D100-96, except the allowable circumferential membrane stresses and material requirements in API 650-01 shall apply.

13.7.9.5 **Bolted Steel Structures.** Bolted steel granular storage structures shall be designed in accordance with the seismic provisions of this section. Component allowable stresses and materials shall be per ANSI/AWWA D103-97.

13.7.9.6 **Reinforced Concrete Structures.** Reinforced concrete structures for the storage of granular materials shall be designed in accordance with the seismic force requirements of this Code and the requirements of ACI 313-97.

13.7.9.7 **Prestressed Concrete Structures.** Prestressed concrete structures for the storage of granular materials shall be designed in accordance with the seismic force provisions of this Code and the requirements of ACI 313-97.

13.7.10 Elevated Tanks and Vessels for Liquids and Granular Materials

13.7.10.1 **General.** This section applies to tanks, vessels, bins, and hoppers that are elevated above grade where the supporting tower is an integral part of the structure, or where the primary function of the tower is to support the tank or vessel. Tanks and vessels that are supported within buildings, or are incidental to the primary function of the tower, are considered mechanical equipment and shall be designed in accordance with Chapter 12 of this Code requirement.

Elevated tanks shall be designed for the force and displacement requirements of the applicable reference document, or Section 13.4.

13.7.10.2 **Effective Mass.** The design of the supporting tower or pedestal, anchorage, and foundation for seismic over-turning shall assume the material stored is a rigid mass acting at the volumetric center of gravity. The effects of fluid-structure interaction may be considered in determining the forces, effective period, and mass centroids of the system if the following requirements are met:

- **a.** The sloshing period, T_c is greater than 3T where T = natural period of the tank with confined liquid (rigid mass) and supporting structure.
- **b.** The sloshing mechanism (i.e., the percentage of convective mass and centroid) is determined for the specific configuration of the container by detailed fluid-structure interaction analysis or testing.

Soil-structure interaction may be included in determining T Section 10.14.

13.7.10.3 **P-Delta Effects.** The lateral drift of the elevated tank shall be considered as follows:

a. The design drift, the elastic lateral displacement of the stored mass center of gravity shall be increased by the factor, C_d for evaluating the additional load in the support structure.

b. The base of the tank shall be assumed to be fixed rotationally and laterally.

c. Deflections due to bending, axial tension, or compression shall be considered. For pedestal tanks with a height-to-diameter ratio less than 5, shear deformations of the pedestal shall be considered.

d. The dead load effects of roof mounted equipment or platforms shall be included in the analysis.

e. If constructed within the plumbness tolerances specified by the reference document, initial tilt need not be considered in the P-delta analysis.

13.7.10.4 **Transfer of Lateral Forces into Support Tower.** For post supported tanks and vessels which are cross-braced:

a. The bracing shall be installed in such a manner as to provide uniform resistance to the lateral load (e.g., pretensioning or tuning to attain equal sag).

b. The additional load in the brace due to the eccentricity between the post to tank attachment and the line of action of the bracing shall be included.

c. Eccentricity of compression strut line of action (elements that resist the tensile pull from the bracing rods in the seismic force-resisting systems) with their attachment points shall be considered.

d. The connection of the post or leg with the foundation shall be designed to resist both the vertical and lateral resultant from the yield load in the bracing assuming the direction of the lateral load is oriented to produce the maximum lateral shear at the post to foundation interface. Where multiple rods are connected to the same location, the anchorage shall be designed to resist the concurrent tensile loads in the braces.

13.7.10.5 **Evaluation of Structures Sensitive to Buckling Failure.** Shell structures that support substantial loads may exhibit a primary mode of failure from localized or general buckling of the support pedestal or skirt during seismic loads. Such structures may include single pedestal water towers, skirt-supported process vessels, and similar single member towers. Where the structural assessment concludes that buckling of the support is the governing primary mode of failure, structures and components in Occupancy Category IV shall be designed to resist the seismic forces as follows:

a. The seismic response coefficient for this evaluation shall be per Section 10.9.2.1 of this Code with I/R set equal to 1.0. Soil-structure and fluid-structure interaction may be utilized in determining the Structural response. Vertical or orthogonal combinations need not be considered.

b. The resistance of the structure or component shall be defined as the critical buckling resistance of the element; i.e., a factor of safety set equal to 1.0.

c. The anchorage and foundation shall be designed to resist the load determined in (a). The foundation shall be proportioned to provide a stability ratio of at least 1.2 for the overturning moment. The maximum toe pressure under the foundation shall not exceed the lesser of the ultimate bearing capacity or 3 times the allowable bearing capacity. All structural components and elements of the foundation shall be designed to resist the combined loads with a load factor of 1.0 on all loads including dead load, live load, and earthquake load. Anchors shall be permitted to yield.

13.7.10.6 Welded Steel Water Storage Structures. Welded steel elevated water storage structures shall be designed and detailed in accordance with the seismic requirements of ANSI/AWWA D100-96 and this Code except that the design input forces for allowable stress design procedures shall be modified by substituting the following terms for $\frac{ZC}{(R_w / I)}$ into Eqs. 13-1 and 13-3 of ANSI/AWWA D100-96 and set the value for S = 1.0.

For $T \leq T_s$ substitute the term

$$\frac{S_{DS}}{1.4(R/I)} \qquad \textbf{(Eq. 13.7-22)}$$

For $T_s < T \leq 4.0$ sec, substitute the term

$$\frac{S_{D1}}{T[1.4(R/I)]} \qquad \textbf{(Eq. 13.7-23)}$$

13.7.10.6.1 Analysis Procedures. The equivalent lateral force procedure shall be permitted. A more rigorous analysis shall also be permitted. Analysis of single pedestal structures shall be based on a fixed-base, single degree-of-freedom model. All mass, including the liquid, shall be considered rigid unless the sloshing mechanism (i.e., the percentage of convective mass and centroid) is determined for the specific configuration of the container by detailed fluid-structure interaction analysis or testing. The inclusion of soil-structure interaction shall be permitted.

13.7.10.6.2 Structure Period. The fundamental period of vibration of the structure shall be established using the structural properties and deformational characteristics of the resisting elements in a substantiated analysis. The period used to calculate the seismic response coefficient shall not exceed 4.0 sec. See ANSI/AWWA D100-96 for guidance on computing the fundamental period of cross-braced structures.

13.7.10.7 Concrete Pedestal (Composite) Tanks. Concrete pedestal (composite) elevated water storage structures shall be designed in accordance with the requirements of ACI 371R-98 and except that the design input forces shall be modified as follows:

In Eq. 4-8a of ACI 371R-98,

For $T_s < T \leq 2.5$ sec, replace the term $\frac{1.2\,C_v}{RT^{2/3}}$ with

$$\frac{S_{D1}}{T(R/I)} \qquad \textbf{(Eq. 13.7-24)}$$

In Eq. 4-8b of ACI 371R-98, replace the term $\frac{2.5\,C_a}{R}$ with

$$\frac{S_{DS}}{(R/I)} \qquad \textbf{(Eq. 13.7-25)}$$

In Eq. 4-9 of ACI 371R-98, replace the term $0.5C_a$ with

$$0.2\,S_{DS} \qquad \textbf{(Eq. 13.7-26)}$$

13.7.10.7.1 Analysis Procedures. The equivalent lateral force procedure shall be permitted for all concrete pedestal tanks and. shall be based on a fixed-base, single degree-of-freedom model. All mass, including the liquid, shall be considered rigid unless the sloshing mechanism (i.e., the percentage of convective mass and centroid) is determined for the specific configuration of the container by detailed fluid-structure interaction analysis or testing. Soil structure interaction may be included as per Section 10.14. A more rigorous analysis shall be permitted.

13.7.10.7.2 **Structure Period.** The fundamental period of vibration of the structure shall be established using the uncracked structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The period used to calculate the seismic response coefficient shall not exceed 2.5 sec.

13.7.11 Boilers and Pressure Vessels

13.7.11.1 **General.** Attachments to the pressure boundary, supports, and lateral force-resisting anchorage systems for boilers and pressure vessels shall be designed to meet the force and displacement requirements of Sections 15.3 or 15.4 and the additional requirements of this section. Boilers and pressure vessels categorized as Occupancy Category III or IV shall be designed to meet the force and displacement requirements of Sections 13.3 or 13.4.

13.7.11.2 **ASME Boilers and Pressure Vessels.** Boilers or pressure vessels designed and constructed in accordance with ASME BPVC-03 shall be deemed to meet the requirements of this section provided that the force and displacement requirements of Sections 13.3 or 13.4 are used with appropriate scaling of the force and displacement requirements to the working stress design basis.

13.7.11.3 **Attachments of Internal Equipment and Refractory.** Attachments to the pressure boundary for internal and external ancillary components (refractory, cyclones, trays, etc.) shall be designed to resist the seismic forces specified in this Code to safeguard against rupture of the pressure boundary. Alternatively, the element attached may be designed to fail prior to damaging the pressure boundary provided that the consequences of the failure do not place the pressure boundary in jeopardy. For boilers or vessels containing liquids, the effect of sloshing on the internal equipment shall be considered if the equipment can damage the integrity of the pressure boundary.

13.7.11.4 **Coupling of Vessel and Support Structure.** Where the mass of the operating vessel or vessels supported is greater than 25 percent of the total mass of the combined structure, the structure and vessel designs shall consider the effects of dynamic coupling between each other. Coupling with adjacent, connected structures such as multiple towers shall be considered if the structures are interconnected with elements that will transfer loads from one structure to the other.

13.7.11.5 **Effective Mass.** Fluid-structure interaction (sloshing) shall be considered in determining the effective mass of the stored material providing sufficient liquid surface exists for sloshing to occur and the T_c is greater than 3T. Changes to or variations in material density with pressure and temperature shall be considered.

13.7.11.6 **Other Boilers and Pressure Vessels.** Boilers and pressure vessels designated as Occupancy Category IV but are not designed and constructed in accordance with the requirements of ASME BPVC-03 shall meet the following requirements:

The seismic loads in combination with other service loads and appropriate environmental effects shall not exceed the material strength shown in Table 13.7-4.

Consideration shall be made to mitigate seismic impact loads for boiler or vessel components constructed of non-ductile materials or vessels operated in such a way that material ductility is reduced (e.g., low temperature applications).

Table 13.7- 4: Maximum Material Strength

Material	Minimum Ratio F_u/F_y	Max. Material Strength Vessel Material	Max. Material Strength Threaded Material[a]
Ductile (e.g., steel, aluminum, copper)	1.33[b]	90%[d]	70%[d]
Semi-ductile	1.2[c]	70%[d]	50%[d]
Nonductile (e.g., cast iron, ceramics, fiberglass)	NA	25%[e]	20%[e]

a Threaded connection to vessel or support system.
b Minimum 20% elongation per the ASTM material specification.
c Minimum 15% elongation per the ASTM material specification.
d Based on material minimum specified yield strength.
e Based on material minimum specified tensile strength.

13.7.11.7 **Supports and Attachments for Boilers and Pressure Vessels.** Attachments to the pressure boundary and support for boilers and pressure vessels shall meet the following requirements:

a. Attachments and supports transferring seismic loads shall be constructed of ductile materials suitable for the intended application and environmental conditions.

b. Seismic anchorages embedded in concrete shall be ductile and detailed for cyclic loads.

c. Seismic supports and attachments to structures shall be designed and constructed so that the support or attachment remains ductile throughout the range of reversing seismic lateral loads and displacements.

d. Vessel attachments shall consider the potential effect on the vessel and the support for uneven vertical reactions based on variations in relative stiffness of the support members, dissimilar details, non-uniform shimming, or irregular supports. Uneven distribution of lateral forces shall consider the relative distribution of the resisting elements, the behavior of the connection details, and vessel shear distribution.

The requirements of Sections 13.4 and 13.7.10.5 shall also be applicable to this section.

13.7.12 Liquid and Gas Spheres

13.7.12.1 **General.** Attachments to the pressure or liquid boundary, supports, and lateral force-resisting anchorage systems for liquid and gas spheres shall be designed to meet the force and displacement requirements of Sections 13.3 or 13.4 and the additional requirements of this section. Spheres categorized as Occupancy Category III or IV shall themselves be designed to meet the force and displacement requirements of Sections 13.3 or 13.4.

13.7.12.2 **ASME Spheres.** Spheres designed and constructed in accordance with Section VIII of ASME BPVC-03 shall be deemed to meet the requirements of this section providing the force and displacement requirements of Sections 13.3 or 13.4 are used with appropriate scaling of the force and displacement requirements to the working stress design basis.

13.7.12.3 **Attachments of Internal Equipment and Refractory.** Attachments to the pressure or liquid boundary for internal and external ancillary components (refractory, cyclones, trays, etc.) shall be designed to resist the seismic forces specified in this Code to safeguard against rupture of the pressure boundary. Alternatively, the element attached to the sphere could be designed to fail prior to damaging the pressure or liquid boundary providing the consequences of the failure does not place the pressure boundary in jeopardy. For spheres containing liquids, the effect of sloshing on the internal equipment shall be considered if the equipment can damage the pressure boundary.

13.7.12.4 **Effective Mass.** Fluid-structure interaction (sloshing) shall be considered in determining the effective mass of the stored material providing sufficient liquid surface exists for sloshing to occur and the T_c is greater than 3T. Changes to or variations in fluid density shall be considered.

13.7.12.5 **Post and Rod Supported.** For post supported spheres that are cross-braced:

- **a.** The requirements of Section 13.7.10.4 shall also be applicable to this section.
- **b.** The stiffening effect of (reduction in lateral drift) from pretensioning of the bracing shall be considered in determining the natural period.
- **c.** The slenderness and local buckling of the posts shall be considered.
- **d.** Local buckling of the sphere shell at the post attachment shall be considered.
- **e.** For spheres storing liquids, bracing connections shall be designed and constructed to develop the minimum published yield strength of the brace. For spheres storing gas vapors only, bracing connection shall be designed for Ω_0 times the maximum design load in the brace. Lateral bracing connections directly attached to the pressure or liquid boundary are prohibited.

13.7.12.6 **Skirt Supported.** For skirt-supported spheres, the following requirements shall apply:

- **a.** The provisions of Section 13.7.10.5 shall also apply.
- **b.** The local buckling of the skirt under compressive membrane forces due to axial load and bending moments shall be considered.
- **c.** Penetration of the skirt support (manholes, piping, etc.) shall be designed and constructed to maintain the strength of the skirt without penetrations.

13.7.13 Refrigerated Gas Liquid Storage Tanks and Vessels

13.7.13.1 **General.** The seismic design of the tanks and facilities for the storage of liquefied hydrocarbons and refrigerated liquids is beyond the scope of this section. The design of such tanks is addressed in part by various reference documents listed in

Section 13.2.

Exception: Low pressure, welded steel storage tanks for liquefied hydrocarbon gas (e.g., LPG, butane, etc.) and refrigerated liquids (e.g., ammonia) shall be designed in accordance with .the requirements of Section 13.7.8 and API 620-02.

13.7.14 Horizontal, Saddle Supported Vessels for Liquid or Vapor Storage

13.7.14.1 General. Horizontal vessels supported on saddles (sometimes referred to as blimps) shall be designed to meet the force and displacement requirements of Sections 13.3 or 13.4.

13.7.14.2 Effective Mass. Changes to or variations in material density shall be considered. The design of the supports, saddles, anchorage, and foundation for seismic overturning shall assume the material stored is a rigid mass acting at the volumetric center of gravity.

13.7.14.3 Vessel Design. Unless a more rigorous analysis is performed,

a. Horizontal vessels with a length-to-diameter ratio of 6 or more may be assumed to be a simply supported beam spanning between the saddles for determining the natural period of vibration and global bending moment.

b. Horizontal vessels with a length-to-diameter ratio of less than 6, the effects of "deep beam shear" shall be considered when determining the fundamental period and stress distribution.

c. Local bending and buckling of the vessel shell at the saddle supports due to seismic load shall be considered. The stabilizing effects of internal pressure shall not be considered to increase the buckling resistance of the vessel shell.

d. If the vessel is a combination of liquid and gas storage, the vessel and supports shall be designed both with and without gas pressure acting (assume piping has ruptured and pressure does not exist).

CHAPTER 14
SITE CLASSIFICATION PROCEDURE FOR SEISMIC DESIGN

SECTION 14.1
GENERAL

14.1.1 **Site Class Definitions.** The site shall be classified as one of the following classes:

A = Hard rock with measured shear wave velocity, $\overline{\upsilon_s}$ > 1500 m/s

B = Rock with 760 m/s < $\overline{\upsilon_s}$ ≤ 1500 m/s

C = Very dense soil and soft rock with 370 m/s ≤ $\overline{\upsilon_s}$ ≤ 760 m/s or $\overline{N}$ or $\overline{N}_{ch}$ > 50 or $\overline{s_u}$ ≥ 100 kPa

D = Stiff soil with 180 m/s ≤ $\overline{\upsilon_s}$ ≤ 370 m/s or with 15 ≤ $\overline{N}$ or $\overline{N}_{ch}$ ≤ 50 or 50 kPa ≤ $\overline{s_u}$ ≤ 100 kPa

E = A soil profile with $\overline{\upsilon_s}$ < 180 m/s or any profile with more than 3 m of soft clay. Soft clay is defined as soil with PI > 20, w ≥ 40%, and $\overline{s_u}$ < 25 kPa

F = Soils requiring site-specific evaluations:

1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils.

   Exception: For structures having fundamental periods of vibration equal to or less than 0.5-sec, site-specific evaluations are not required to determine spectral accelerations for liquefiable soils. Rather, the Site Class may be determined in accordance with Section 14.1.2 and the corresponding values of F_a and F_v determined from Tables 9.4.3a and 9.4.3b.
2. Peats and/or highly organic clays (H > 3 m of peat and/or highly organic clay where *H* = thickness of soil).
3. Very high plasticity clays *(H* > 8 m with *PI > 75)*.
4. Very thick soft/medium stiff clays (H > 37 m).

Exception: When the soil properties are not known in sufficient detail to determine the Site Class, Class D shall be used. Site Class E shall be used when the authority having jurisdiction determines that Site Class E is present at the site or in the event that Site E is established by geotechnical data.

14.1.1.1 **Referenced Standards.** The following standards are referenced in the provisions for determining the seismic coefficients:

[1] ASTM. "Test Method for Penetration Test and Split-Barrel Sampling of Soils." ASTM D1586-84, 1984.

[2] ASTM. "Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils." ASTM D4318-93, 1993.

[3] ASTM. "Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock." ASTM D2216-92, 1992.

[4] ASTM. "Test Method for Unconfined Compressive Strength of Cohesive Soil." ASTM D2166-91, 1991.

[5] ASTM. "Test Method for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression." ASTM D2850-87, 1987.

TABLE 14.1.1: SITE CLASSIFICATION

Site Class	$\overline{\upsilon}_s$	$\overline{N}$ or $\overline{N}_{ch}$	$\overline{s}_u$
A Hard rock	> 1500 m/s	not applicable	not applicable
B Rock	760 to 1500 m/s	not applicable	not applicable
C Very dense soil and soft rock	370 to 760 m/s	> 50	> 100 kPa
D Stiff soil	180 to 370 m/s	15 to 50	50 to 100 kPa
E Soil	<180 m/s	< 15	<50 kPa
	Any profile with more than 3 m of soil having the following characteristics: - Plasticity index PI > 20, - Moisture content w ≥ 40%, and - Undrained shear strength $\overline{s}_u$ < 25 kPa		
F Soils requiring site-specific evaluation	1. Soils vulnerable to potential failure or collapse 2. Peats and/or highly organic clays 3. Very high plasticity clays 4. Very thick soft/medium clays		

Note: When the soil properties are not known in sufficient detail to determine the Site Class, Class D or E shall be used.

14.1.2 **Steps for Classifying a Site.** The Site Class of a site shall be determined using the following steps:

Step 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.

Step 2: Check for the existence of a total thickness of soft clay > 3 m where a soft clay layer is defined by $\overline{s}_u$ < 25 kPa, $w \geq 40\%$, and $PI > 20$. If this criterion is satisfied, classify the site as Site Class E.

Step 3: Categorize the site using one of the following three methods with $\overline{\upsilon}_s$, $\overline{N}$, and $\overline{s}_u$ computed in all cases as specified by the definitions in Section 14.1.3.

a. The $\overline{\upsilon_s}$ method:

Determine $\overline{\upsilon_s}$ for the top 30 m of soil. Compare the value of $\overline{\upsilon_s}$ with those given in Section 14.1.1 and Table 14.1.1 and assign the corresponding Site Class.

$\overline{\upsilon_s}$ for rock, Site Class B, shall be measured on-site or estimated by a geotechnical engineer or engineering geologist/seismologist for competent rock with moderate fracturing and weathering.

$\overline{\upsilon_s}$ for softer and more highly fractured and weathered rock shall be measured on-site or shall be classified as Site Class C.

The classification of hard rock, Site Class A, shall be supported by on-site measurements of $\overline{\upsilon_s}$ 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 at least 30 m, surficial measurements of υ_s are not prohibited from being extrapolated to assess $\overline{\upsilon_s}$.

The rock categories, Site Classes A and B, shall not be assigned to a site if there is more than 3 m of soil between the rock surface and the bottom of the spread footing or mat foundation.

b. The $\overline{N}$ method:

Determine $\overline{N}$ for the top 30 m of soil. Compare the value of $\overline{N}$ with those given in Section 14.1.1 and Table 14.1.1 and assign the corresponding Site Class.

c. The $\overline{s_u}$ method:

For cohesive soil layers, determine $\overline{s_u}$ for the top 30 m of soil. For cohesionless soil layers, determine $\overline{N}_{ch}$ for the top 30 m of soil. Cohesionless soil is defined by a PI < 20 where cohesive soil is defined by a PI > 20. Compare the values of $\overline{s_u}$ and $\overline{N}_{ch}$ with those given in Section 14.1.1 and Table 14.1.1 and assign the corresponding Site Class. When the $\overline{N}_{ch}$ and $\overline{s_u}$ criteria differ, assign the category with the softer soil. (Site Class E soil is softer than D).

14.1.3 **Definitions of Site Class Parameters.** The definitions presented below apply to the upper 30 m of the site profile. Profiles containing distinctly different soil layers shall be subdivided into those layers designated by a number that ranges from 1 to *n* at the bottom where there are a total of *n* distinct layers in the upper 30 m. Where some of the *n* layers are cohesive and others are not, *k* is the number of cohesive layers and *m* is the number of cohesionless layers. The symbol *i* refers to any one of the layers between 1 and *n*.

v_{si} is the shear wave velocity in m/s.

d_i is the thickness of any layer between 0 and 30 m.

$\overline{\upsilon_s}$ is

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{\upsilon_{si}}} \quad \textbf{(Eq. 14.1.3-1)}$$

whereby $\sum_{i=1}^{n} d_i$ is equal to 30 m

N_i is the standard penetration resistance, ASTM D1586-84 not to exceed 100 blows/300 mm as directly measured in the field without corrections.

$\overline{N}$ is:

$$\overline{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \quad \textbf{(Eq. 14.1.3-2)}$$

$\overline{N}_{ch}$ is:

$$\overline{N}_{ch} = \frac{d_s}{\sum_{i=1}^{m} \frac{d_i}{N_i}} \quad \textbf{(Eq. 14.1.3-3)}$$

whereby $\sum_{i=1}^{m} d_i = d_s$. (Use only d_i and N_i for cohesionless soils.)

d_s is the total thickness of cohesionless soil layers in the top 30 m.

$\overline{s_{ui}}$ is the undrained shear strength in kPa, not to exceed 240 kPa, ASTM D2166-91 or ASTM D2850-87.

$\overline{s_u}$ is

$$\bar{s}_u = \frac{d_c}{\sum_{i=1}^{k} \frac{d_i}{s_{ui}}} \quad \textbf{(Eq. 14.1.3-4)}$$

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

d_c is the total thickness (30 - d_s) of cohesive soil layers in the top 30 m

PI is the plasticity index, ASTM D4318-93

ω is the moisture content in percent, ASTM D2216-92

CHAPTER 15
QUALITY ASSURANCE AND SUPPLEMENTAL PROVISIONS

SECTION 15.1
PURPOSE

These provisions are not directly related to computation of earthquake loads, but they are deemed essential for satisfactory performance in an earthquake when designing with the loads determined from Chapters 9 through 13, due to the substantial cyclic inelastic strain capacity assumed to exist by the load procedures given in the aforementioned chapters. These supplemental provisions form an integral part of SBC 301.

SECTION 15.2
QUALITY ASSURANCE

This section provides minimum requirements for quality assurance for seismic force-resisting systems and other designated seismic systems. These requirements supplement the testing and inspection requirements contained in the reference standards given in Chapters 11 through 13.

15.2.1 **Scope.** As a minimum, the quality assurance provisions apply to the following:

1. The seismic force-resisting systems in structures assigned to Seismic Design Categories C and D.
2. Other designated seismic systems in structures assigned to Seismic Design Category D that are required in Table 12.1.7.

   Exception: Structures that comply with the following criteria are exempt from the preparation of a quality assurance plan but those structures are not exempt from special inspection(s) or testing requirements:

   a. The structure is constructed of light-gauge cold-formed steel framing, S_{DS} does not exceed 0.50 g, the height of the structure does not exceed 10 m above grade, and the structure meets the requirements in Items c and d below,

   or

   b. The structure is constructed using a reinforced masonry structural system or reinforced concrete structural system, S_{DS} does not exceed 0.50 g, the height of the structure does not exceed 8 m above grade, and the structure meets the requirements in Items c and d below.

   c. The structure is classified as Occupancy Category I.

   d. The structure does not have any of the following plan irregularities as defined in Table 10.3.2.1 or any of the following vertical irregularities as defined in Table 10.3.2.2:

      1. Torsional irregularity
      2. Extreme torsional irregularity
      3. Nonparallel systems

4. Stiffness irregularity, soft story
5. Stiffness irregularity, extreme soft story
6. Discontinuity in capacity, weak story

15.2.2 Reference Standards.
The following standards (Ref. 15.2.2-1 through Ref. 15.2.2-3) are referenced in the provisions for inspection and testing. See Reference Chapter SBC 301.

15.2.3 Quality Assurance Plan. A quality assurance plan shall be submitted to the authority having jurisdiction.

15.2.3.1 Details of Quality Assurance Plan. The quality assurance plan shall specify the designated seismic systems or seismic force-resisting system in accordance with Section 15.2 that are subject to quality assurance. The registered design professional in responsible charge of the design of a seismic force-resisting system and a designated seismic system shall be responsible for the portion of the quality assurance plan applicable to that system. The special inspections and special tests needed to establish that the construction is in conformance with these provisions shall be included in the portion of the quality assurance plan applicable to the designated seismic system. The quality assurance plan shall include:

1. The seismic force-resisting systems and designated seismic systems in accordance with this chapter that are subject to quality assurance.
2. The special inspections and testing to be provided as required by these provisions and the reference standards in Chapter 11 through 13.
3. The type and frequency of testing.
4. The type and frequency of special inspections.
5. The frequency and distribution of testing and special inspection reports.
6. The structural observations to be performed.
7. The frequency and distribution of structural observation reports.

15.2.3.2 Contractor Responsibility. Each contractor responsible for the construction of a seismic force-resisting system, designated seismic system, or component listed in the quality assurance plan shall submit a written contractor's statement of responsibility to the regulatory authority having jurisdiction and to the owner prior to the commencement of work on the system or component. The contractor's statement of responsibility shall contain the following:

1. Acknowledgment of awareness of the special requirements contained in the quality assurance plan.
2. Acknowledgment that control will be exercised to obtain conformance with the design documents approved by the authority having jurisdiction.
3. Procedures for exercising control within the contractor's organization, the method and frequency of reporting, and the distribution of the reports.
4. Identification and qualifications of the person(s) exercising such control and their position(s) in the organization.

15.2.4 **Special Inspection.** The building owner shall employ a special inspector(s) to observe the construction of all designated seismic systems in accordance with the quality assurance plan for the following construction work:

15.2.4.1 **Foundations.** Continuous special inspection is required during driving of piles and placement of concrete in piers or piles. Periodic special inspection is required during construction of drilled piles, piers, and caisson work, the placement of concrete in shallow foundations, and the placement of reinforcing steel.

15.2.4.2 **Reinforcing Steel.**

15.2.4.2.1 Periodic special inspection during and on completion of the placement of reinforcing steel in intermediate and special moment frames of concrete and concrete shear walls.

15.2.4.2.2 Continuous special inspection during the welding of reinforcing steel resisting flexural and axial forces in intermediate and special moment frames of concrete, in boundary members of concrete shear walls, and welding of shear reinforcement.

15.2.4.3 **Structural Concrete.** Periodic special inspection during and on completion of the placement of concrete in intermediate and special moment frames, and in boundary members of concrete shear walls.

15.2.4.4 **Prestressed Concrete.** Periodic special inspection during the placement and after the completion of placement of prestressing steel and continuous special inspection is required during all stressing and grouting operations and during the placement of concrete.

15.2.4.5 **Structural Masonry.**

15.2.4.5.1 Periodic special inspection during the preparation of mortar, the laying of masonry units, and placement of reinforcement; and prior to placement of grout.

15.2.4.5.2 Continuous special inspection during welding of reinforcement, grouting, consolidation and reconsolidation, and placement of bent-bar anchors as required by Section 15.6.

15.2.4.6 **Structural Steel.**

15.2.4.6.1 Continuous special inspection is required for all structural welding.

Exception: Periodic special inspection for single-pass fillet or resistance welds and welds loaded to less than 50% of their design strength shall be the minimum requirement, provided the qualifications of the welder and the welding electrodes are inspected at the beginning of the work and all welds are inspected for compliance with the approved construction documents at the completion of welding.

15.2.4.6.2 Periodic special inspection is required for installation and tightening of fully tensioned high-strength bolts in slip-critical connections and in connections subject to direct tension. Bolts in connections identified as not being slip-critical or subject to direct tension need not be inspected for bolt tension other than to ensure that the plies of the connected elements have been brought into snug contact.

15.2.4.7 Cold-Formed Steel Framing.

15.2.4.7.1 Periodic special inspection is required during all welding operations of elements of the seismic force-resisting system.

15.2.4.7.2 Periodic special inspection is required for screw attachment, bolting, anchoring, and other fastening of components within the seismic force-resisting system including struts, braces, and hold-downs.

15.2.4.8 Architectural Components. Special inspection for architectural components shall be as follows:

1. Periodic special inspection during the erection and fastening of exterior cladding, interior and exterior nonbearing walls, and interior and exterior veneer in Seismic Design Category D.

   Exceptions:

   a. Architectural components less than 9 m above grade or walking surface.

   b. Cladding and veneer weighing 250 N/ m^2 or less.

   c. Interior nonbearing walls weighing 700 N/m^2 or less.

2. Periodic special inspection during the anchorage of access floors, suspended ceilings, and storage racks 2.5 m or greater in height in Seismic Design Category D.

3. Periodic special inspection during erection of glass 9 m or more above an adjacent grade or walking surface in glazed curtain walls, glazed storefronts, and interior glazed partitions in Seismic Design Category D.

15.2.4.9 Mechanical and Electrical Components. Special inspection for mechanical and electrical components shall be as follows:

1. Periodic special inspection during the anchorage of electrical equipment for emergency or standby power systems in Seismic Design Categories C and D.

2. Periodic special inspection during the installation for flammable, combustible, or highly toxic piping systems and their associated mechanical units in Seismic Design Categories C, and D.

3. Periodic special inspection during the installation of HVAC ductwork that will contain hazardous materials in Seismic Design Categories C and D.

4. Periodic special inspection during the installation of vibration isolation systems when the construction documents indicate a maximum clearance (air gap) between the equipment support frame and restraint less than or equal to 6 mm.

15.2.5 Testing. The special inspector(s) shall be responsible for verifying that the special test requirements are performed by an approved testing agency for the types of work in designated seismic systems listed below.

15.2.5.1 Reinforcing and Prestressing Steel. Special testing of reinforcing and prestressing steel shall be as follows:

15.2.5.1.1 Examine certified mill test reports for each shipment of reinforcing steel used to resist flexural and axial forces in reinforced concrete intermediate and special moment frames and boundary members of reinforced concrete shear walls or reinforced masonry shear walls and determine conformance with construction documents.

15.2.5.1.2 Where ASTM A615 or Saudi equivalent reinforcing steel is used to resist earthquake-induced flexural and axial forces in special moment frames and in wall boundary elements of shear walls in structures of Seismic Design Category D, verify that the requirements of Section 21.2.5.1 of SBC 304 have been satisfied.

15.2.5.1.3 Where ASTM A615 reinforcing steel or Saudi equivalent is to be welded, verify that chemical tests have been performed to determine weldability in accordance with Section 3.5.2 of SBC 304.

15.2.5.2 **Structural Concrete.** Samples of structural concrete shall be obtained at the project site and tested in accordance with the requirements of SBC 304.

15.2.5.3 **Structural Masonry.** Quality assurance testing of structural masonry shall be in accordance with the requirements of Ref. 11.4-1.

15.2.5.4 **Structural Steel.** The testing needed to establish that the construction is in conformance with these provisions shall be included in a quality assurance plan. The minimum testing contained in the quality assurance plan shall be as required in Ref. 11.1-1 and the following requirements:

15.2.5.4.1 **Base Metal Testing.** Base metal thicker than 38 mm, when subject to through-thickness weld shrinkage strains, shall be ultrasonically tested for discontinuities behind and adjacent to such welds after joint completion. Any material discontinuities shall be accepted or rejected on the basis of ASTM A435, *Specification for Straight Beam Ultrasound Examination of Steel Plates,* or ASTM A898, *Specification for Straight Beam Ultrasound Examination for Rolled Steel Shapes (Level 1 Criteria),* and criteria as established by the registered design professional(s) in responsible charge and the construction documents.

15.2.5.5 **Mechanical and Electrical Equipment.** As required to ensure compliance with the seismic design provisions herein, the registered design professional in responsible charge shall clearly state the applicable requirements on the construction documents. Each manufacturer of these designated seismic system components shall test or analyze the component and its mounting system or anchorage as required and shall submit a certificate of compliance for review and acceptance by the registered design professional in responsible charge of the design of the designated seismic system and for approval by the authority having jurisdiction. The basis of certification shall be by actual test on a shaking table, by three-dimensional shock tests, by an analytical method using dynamic characteristics and forces, by the use of experience data (i.e., historical data demonstrating acceptable seismic performance), or by more rigorous analysis providing for equivalent safety. The special inspector shall examine the designated seismic system and shall determine whether its anchorages and label conform with the certificate of compliance.

15.2.6 **Structural Observations.** Structural observations shall be provided for those structures included in Seismic Design Category D, when one or more of the following conditions exist:

1. The structure is included in Occupancy Category III or IV, or
2. The height of the structure is greater than 25 m above the base.

   Observed deficiencies shall be reported in writing to the owner and the authority having jurisdiction.

15.2.7 **Reporting and Compliance Procedures.** Each special inspector shall furnish to the authority having jurisdiction, the registered design professional in responsible charge, the owner, the persons preparing the quality assurance plan, and the contractor copies of regular weekly progress reports of his observations, noting therein any uncorrected deficiencies and corrections of previously reported deficiencies. All deficiencies shall be brought to the immediate attention of the contractor for correction. At completion of construction, each special inspector shall submit a final report to the authority having jurisdiction certifying that all inspected work was completed substantially in accordance with approved construction documents. Work not in compliance shall be described in the final report. At completion of construction, the building contractor shall submit a final report to the authority having jurisdiction certifying that all construction work incorporated into the seismic force-resisting system and other designated seismic systems was constructed substantially in accordance with the approved construction documents and applicable workmanship requirements. Work not in compliance shall be described in the final report. The contractor shall correct all deficiencies as required.

SECTION 15.3
SUPPLEMENTARY FOUNDATION REQUIREMENTS

15.3.1 **Special Pile Requirements for Category C.** All concrete piles and concrete-filled pipe piles shall be connected to the pile cap by embedding the pile reinforcement in the pile cap for a distance equal to the development length as specified in SBC 304 as modified by Section 15.5 of this Code or by the use of field-placed dowels anchored in the concrete pile. 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.

Hoops, spirals, and ties shall be terminated with seismic hooks as defined in Section 21.1 of SBC 304.

Where required 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.

Where a minimum length for reinforcement or the extent of closely spaced confinement reinforcement is specified at the top of the pile, provisions shall be made so that those specified lengths or extents are maintained after pile cutoff.

15.3.1.1 **Uncased Concrete Piles.** Reinforcement shall be provided where required by analysis. As a minimum, longitudinal reinforcement ratio of 0.0025 shall be provided for uncased cast-in-place concrete drilled or augered piles, piers, or caissons in the top one-third of the pile length or a minimum length of 3 m below the ground. There shall be a minimum of four longitudinal bars with closed ties (or equivalent spirals) of a minimum of a 10 mm diameter provided at 16 longitudinal-bar-diameter maximum spacing. Transverse confinement reinforcing with a maximum spacing of 150 mm or 8 longitudinal-bar-diameters, whichever is less, shall be provided in the pile within three pile diameters of the bottom of the pile cap.

15.3.1.2 **Metal-Cased Concrete Piles.** Reinforcement requirements are the same as for uncased concrete piles.

Exception: Spiral welded metal casing of a thickness not less than No. 14 gauge can be considered as providing concrete confinement equivalent to the closed ties or equivalent spirals required in an uncased concrete pile, provided that the metal casing is adequately protected against possible deleterious action due to soil constituents, changing water levels, or other factors indicated by boring records of site conditions.

15.3.1.3 **Concrete-Filled Pipe.** Minimum reinforcement 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.

15.3.1.4 **Precast Nonprestressed Concrete Piles.** A minimum longitudinal steel reinforcement ratio of 0.01 shall be provided for precast nonprestressed concrete piles. The longitudinal reinforcing shall be confined with closed ties or equivalent spirals of a minimum 10 mm diameter. Transverse confinement reinforcing shall be provided at a maximum spacing of eight times the diameter of the smallest longitudinal bar, but not to exceed 150 mm, within three pile diameters of the bottom of the pile cap. Outside of the confinement region, closed ties or equivalent spirals shall be provided at a 16 longitudinal-bar-diameter maximum spacing, but not greater than 200 mm. Reinforcement shall be full length.

15.3.1.5 **Precast Prestressed Piles.** For the upper 6 m of precast prestressed piles, the minimum volumetric ratio of spiral reinforcement shall not be less than 0.007 or the amount required by the following formula:

$$\rho_s = \frac{0.12 f_c'}{f_{yh}} \qquad \textbf{(Eq. 15.3.1.5-1)}$$

ρ_s = volumetric ratio (vol. spiral/vol. core)

f_c' = specified compressive strength of concrete, (MPa)

f_{yh} = specified yield strength of spiral reinforcement, which shall not be taken greater than 580 MPa

A minimum of one-half of the volumetric ratio of spiral reinforcement required by Eq. 15.3.1.5-1 shall be provided for the remaining length of the pile.

15.3.2 **Special Pile Requirements for Category D.**

15.3.2.1 **Uncased Concrete Piles.** A minimum longitudinal reinforcement ratio of 0.005

shall be provided for uncased cast-in-place drilled or augered concrete piles, piers, or caissons in the top one-half of the pile length, or a minimum length of 3m below ground, or throughout the flexural length of the pile, whichever length is greatest. The flexural length shall be taken as the length of pile to a point where the concrete section cracking moment multiplied by the resistance factor 0.4 exceeds the required factored moment at that point. There shall be a minimum of four longitudinal bars with transverse confinement reinforcement in the pile in accordance with Sections 21.4.4.1, 21.4.4.2, and 21.4.4.3 of SBC 304. Such transverse confinement reinforcement shall extend the full length of the pile in Site Classes E or F, a minimum of seven times the least pile dimension above and below the interfaces of soft to medium stiff clay or liquefiable strata, and three times the least pile dimension below the bottom of the pile cap in Site Classes other than E or F.

In other than Site Classes E or F, it shall be permitted to use a transverse spiral reinforcement ratio of not less than one-half of that required in Section 21.4.4.1(a) of SBC 304 throughout the remainder of the pile length. Tie spacing throughout the remainder of the pile length shall not exceed 12 longitudinal-bar-diameters, one-half the diameter of the section, or 300 mm. Ties shall be a minimum of 10 mm bars for up to diameter 500 mm piles and 12 mm bars for piles of larger diameter.

15.3.2.2 **Metal-Cased Concrete Piles.** Reinforcement requirements are the same as for uncased concrete piles.

Exception: Spiral welded metal-casing of a thickness not less than No. 14 gauge can be considered as providing concrete confinement equivalent to the closed ties or equivalent spirals required in an uncased concrete pile, provided that the metal casing is adequately protected against possible deleterious action due to soil constituents, changing water levels, or other factors indicated by boring records of site conditions.

15.3.2.3 **Precast Concrete Piles.** Transverse confinement reinforcement consisting of closed ties or equivalent spirals shall be provided in accordance with Sections 21.4.4.1, 21.4.4.2, and 21.4.4.3 of SBC 304 for the full length of the pile.

Exception: In other than Site Classes E or F, the specified transverse confinement reinforcement shall be provided within three pile diameters below the bottom of the pile cap, but it shall be permitted to use a transverse reinforcing ratio of not less than one-half of that required in Section 21.4.4.1(a) of SBC 304 throughout the remainder of the pile length.

15.3.2.4 **Precast Prestressed Piles.** In addition to the requirements for Seismic Design Category C, the following requirements shall be met:

1. Requirements of SBC 304, Chapter 21, need not apply.

2. Where the total pile length in the soil is 10 m or less, the lateral transverse reinforcement in the ductile region shall occur through the length of the pile. Where the pile length exceeds 10 m, the ductile pile region shall be taken as the greater of 10 m 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 200 mm, whichever is smaller.

4. Spiral reinforcement shall be spliced by lapping one full turn by welding or by the use of a mechanical connector. Where spiral reinforcement is lap spliced, the ends of the spiral shall terminate in a seismic hook in accordance with SBC 304, except that the bend shall be not less than 135 degrees. Welded splices and mechanical connectors shall comply with Section 12.14.3 of SBC 304.

5. Where the transverse reinforcement consists of circular spirals, the volumetric ratio of spiral transverse reinforcement in the ductile region shall comply with:

$$\rho_s = 0.25 \frac{f'_c}{f_{yh}} \left[\frac{A_g}{A_{ch}} - 1.0 \right] \times \left[0.5 + \frac{1.4\,P}{f'_c\, A_g} \right]$$

but not less than

$$\rho_s = 0.12 \frac{f'_c}{f_{yh}} \left[0.5 + \frac{1.4\,P}{f'_c\, A_g} \right]$$

and not to exceed $\rho_s = 0.021$

where

ρ_s = volumetric ratio (vol. spiral/vol. core

$f'_c \leq$ 40 MPa

f_{yh} = yield strength of spiral reinforcement 580 MPa

A_g = pile cross-sectional area, mm^2

A_{ch} = core area defined by spiral outside diameter, mm^2

P = axial load on pile resulting from the load combination 1.2D + 0.5L + 1.0E, kN

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 spacings, and perpendicular to dimension, h_c, shall conform to:

$$A_{sh} = 0.3\, s h_c \frac{f'_c}{f_{yh}} \left[\frac{A_g}{A_{ch}} - 1.0 \right] \times \left[0.5 + \frac{1.4\,P}{f'_c\, A_g} \right]$$

but not less than

$$A_{sh} = 0.12\, s h_c \frac{f'_c}{f_{yh}} \left[0.5 + \frac{1.4\,P}{f'_c\, A_g} \right]$$

where

s = spacing of transverse reinforcement measured along length of pile, mm

h_c = cross-sectional dimension of pile core measured center-to-center of hoop reinforcement, mm

$f_{yh} \leq 480$ 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 reinforcement, the spiral or hoop reinforcement with a volumetric ratio not less than one-half of that required for transverse confinement reinforcement shall be provided.

15.3.2.5 **Steel Piles.** The connection between the pile cap and steel piles or unfilled steel pipe piles shall be designed for a tensile force equal to 10% of the pile compression capacity.

> **Exceptions:** Connection tensile capacity need not exceed the strength required to resist the special seismic loads of Section 10.4. Connections need not be provided where the foundation or supported structure does not rely on the tensile capacity of the piles for stability under the design seismic forces.

SECTION 15.4
SUPPLEMENTARY PROVISIONS FOR STEEL

15.4.1 **General.** The design, construction, and quality of steel components that resist seismic forces shall conform to the requirements of the references listed in Section 11.1 except as modified by the requirements of this section.

15.4.2 **Seismic Requirements for Steel Structures.** Steel structures and structural elements therein that resist seismic forces shall be designed in accordance with the requirements of Sections 15.4.3 and 15.4.4 for the appropriate Seismic Design Category.

15.4.3 **Seismic Design Categories A, B, and C.** Steel structures assigned to Seismic Design Categories A, B, and C shall be of any construction permitted by the references in Section 15.4.1. An R factor as set forth in Table 10.2 shall be permitted when the structure is designed and detailed in accordance with the requirements of Ref. 11.1-1 for structural steel buildings as modified by this chapter and Section 15.4.6 for light-framed walls. Systems not detailed in accordance with Ref. 11.1-1 shall use the R factor designated for "Systems not detailed for seismic."

15.4.4 **Seismic Design Category D.** Steel structures assigned to Seismic Design Category D shall be designed and detailed in accordance with Ref. 11.1-1 Part I or Section 15.4.6 for light-framed cold-formed steel wall systems.

15.4.5 **Cold-Formed Steel Seismic Requirements.** The design of cold-formed carbon or low-alloy steel to resist seismic loads shall be in accordance with the provisions of Ref. 11.1-2, and the design of cold-formed stainless steel structural members to resist seismic loads shall be in accordance with the provisions of Ref. 11.1-3, except as modified by this section. The references to section and paragraph numbers are to those of the particular specification modified.

15.4.5.1 Ref. 11.1-2-Revised Section A5.1.3 of Ref. 11.1-2 by deleting the reference to earthquake or seismic loads in the sentence permitting the 0.75 factor. Seismic load combinations shall be as determined by this Code.

15.4.6 **Light-Framed Wall Requirements.** Cold-formed steel stud wall systems designed in accordance with Ref. 11.1-2 or 11.1-3 shall, when required by the provisions of Sections 15.4.3 or 15.4.4, also comply with the requirements of this section.

15.4.6.1 **Boundary Members.** All boundary members, chords, and collectors shall be designed to transmit the axial force induced by the specified loads of Chapters 9 through 13.

15.4.6.2 **Connections.** Connections of diagonal bracing members, top chord splices, boundary members, and collectors shall have a design strength equal to or greater than the nominal tensile strength of the members being connected or Ω_o times the design seismic forces. The pullout resistance of screws shall not be used to resist seismic forces.

15.4.6.3 **Braced Bay Members.** In stud systems where the lateral forces are resisted by braced frames, the vertical and diagonal members of braced bays shall be anchored such that the bottom tracks are not required to resist tensile forces by bending of the track or track web. Both flanges of studs in a bracing bay shall be braced to prevent lateral torsional buckling. In braced shear walls, the vertical boundary members shall be anchored so the bottom track is not required to resist uplift forces by bending of the track web.

15.4.6.4 **Diagonal Braces.** Provision shall be made for pre-tensioning or other methods of installation of tension-only bracing to prevent loose diagonal straps.

15.4.6.5 **Shear Walls.** Nominal shear values for wall sheathing materials are given in Table 15.4.6.5. Design shear values shall be determined by multiplying the nominal values therein by a factor of 0.55. In structures over 1 story in height, the assemblies in Table 15.4.6.5 shall not be used to resist horizontal loads contributed by forces imposed by masonry or concrete construction.

Panel thicknesses shown in Table 15.4.6.5 shall be considered to be minimums. No panels less than 50 mm. wide shall be used. Plywood or oriented strand board structural panels shall be of a type that is manufactured using exterior glue. Framing members, blocking, or strapping shall be provided at the edges of all sheets. Fasteners along the edges in shear panels shall be placed not less than 10 mm in from panel edges. Screws shall be of sufficient length to ensure penetration into the steel stud by at least two full diameter threads.

The height-to-length ratio of wall systems listed in Table 15.4.6.5 shall not exceed 2:1.

Perimeter members at openings shall be provided and shall be detailed to distribute the shearing stresses. Wood sheathing shall not be used to splice these members.

Wall studs and track shall have a minimum uncoated base thickness of not less than 0.85 mm and shall not have an uncoated base metal thickness greater than 1.22 mm. Panel end studs and their uplift anchorage shall have the design strength to resist the forces determined by the seismic loads.

15.4.7 **Seismic Requirements for Steel Deck Diaphragms.** Steel deck diaphragms shall be made from materials conforming to the requirements of Ref. 11.1-2 or 11.1-3. Nominal strengths shall be determined in accordance with approved analytical procedures or with test procedures prepared by a registered design professional experienced in testing of cold-formed steel assemblies and approved by the authority having jurisdiction. Design strengths shall be determined by multiplying the nominal strength by a resistance factor, ϕ equal to 0.60 for mechanically connected diaphragms and equal to 0.50 for welded diaphragms. The steel deck installation for the building, including fasteners, shall comply with the test assembly arrangement. Quality standards established for the nominal strength test shall be the minimum standards required for the steel deck installation, including fasteners.

15.4.8 **Steel Cables.** The design strength of steel cables shall be determined by the provisions of Ref. 11.1-5 except as modified by this section. Ref. 11.1-5, Section 5d, shall be modified by substituting $1.5(T_4)$ when T_4 is the net tension in cable due to dead load, prestress, live load, and seismic load. A load factor of 1.1 shall be applied to the prestress force to be added to the load combination of Section 3.1.2 of Ref. 11.1-5.

SECTION 15.5
SUPPLEMENTAL PROVISIONS FOR CONCRETE

The supplemental provisions for Concrete have already been incorporated in SBC 304.

SECTION 15.6
SUPPLEMENTARY PROVISIONS FOR MASONRY

15.6.1 For the purposes of design of masonry structures using the earthquake loads given in this Code, several amendments to the reference standard are necessary.

15.6.2 The references to "Seismic Performance Category" in Section 1.13 and elsewhere of the reference standard shall be replaced by "Seismic Design Category".

15.6.3 The anchorage forces given in Section 1.13.3.2 of the reference standard shall not be interpreted to replace the anchorage forces given in Chapters 9 through 13 of this Code.

15.6.4 To qualify for the R factors given in Chapters 9 through 13 of this Code, the requirements of the reference standard shall be satisfied and amended as follows: Ordinary and detailed plain masonry shear walls shall be designed according to Sections 2.1 and 2.2 of the reference standard. Detailed plain masonry shear walls shall be reinforced as a minimum as required in Section 1.13.5.3.3 of the reference standard.

Ordinary, intermediate, and special reinforced masonry shear walls shall be designed according to Sections 2.1 and 2.3 of the reference standard. Ordinary and intermediate reinforced masonry shear walls shall be reinforced as a minimum as required in Section 1.13.5.3.3 of the reference standard. In addition, intermediate reinforced masonry shear walls shall have vertical reinforcing bars spaced no farther apart than 1200 mm. Special reinforced masonry shear walls shall be reinforced as a minimum as required in Sections 1.13.6.3 and 1.13.6.4 of the reference standard.

Special reinforced masonry shear walls shall not have a ratio of reinforcement ρ, that exceeds that given by either Method A or B below:

15.6.4.1 **Method A**. Method A is permitted to be used where the story drift does not exceed 0.010h, as given in Table 10.12 and if the extreme compressive fiber strains are less than 0.0035 mm/mm for clay masonry and 0.0025 mm/mm for concrete masonry.

1. When walls are subjected to in-plane forces, and for columns and beams, the critical strain condition corresponds to a strain in the extreme tension reinforcement equal to five times the strain associated with the reinforcement yield stress, f_y.
2. When walls are subjected to out-of-plane forces, the critical strain condition corresponds to a strain in the reinforcement equal to 1.3 times the strain associated with reinforcement yield stress, f_y.

The strain at the extreme compression fiber shall be assumed to be 0.0035 mm/mm for clay masonry and 0.0025 mm/mm for concrete masonry.

The calculation of the maximum reinforcement ratio shall include factored gravity axial loads. The stress in the tension reinforcement shall be assumed to be 1.25 f_y. Tension in the masonry shall be neglected. The strength of the compressive zone shall be calculated as 80% of the area of the compressive zone. Stress in reinforcement in the compression zones shall be based on a linear strain distribution.

15.6.4.2 **Method B.** Method B is permitted to be used where story drift does not exceed $0.013h_{sx}$ as given in Table 10.12.

1. Boundary members shall be provided at the boundaries of shear walls when the compressive strains in the wall exceed 0.002. The strain shall be determined using factored forces and R equal to 1.5.
2. The minimum length of the boundary member shall be three times the thickness of the wall, but shall include all areas where the compressive strain per Section 15.6.4.2 item 1, is greater than 0.002.
3. Lateral reinforcement shall be provided for the boundary elements. The lateral reinforcement shall be a minimum of 10 mm closed ties at a maximum spacing of 200 mm on center within the grouted core, or equivalent approved confinement, to develop an ultimate compressive strain of at least 0.006.
4. The maximum longitudinal reinforcement ratio shall not exceed $0.15\frac{f'_m}{f_y}$.

15.6.5 Where allowable stress design is used for load combinations including earthquake, Eq. 2.4.1-3 and Eq. 2.4.1-5 of Section 2.4.1 of this Code shall replace combinations (c) and (e) of Section 2.1.1.1.1 of the reference standard.

TABLE 15.4.6.5: NOMINAL SHEAR VALUES FOR SEISMIC FORCES FOR SHEAR WALLS FRAMED WITH COLD-FORMED STEEL STUDS (IN kN/m)[a,b]

Assembly Description	Fastener Spacing at Panel Edges[c] (mm)				Framing Spacing (mm o.c.)
	150	100	75	50	
12 mm rated structural I sheathing (4-ply) plywood one side[d]	11.4	14.4	21.4	23.7	600
12 mm oriented strand board one side[d]	10.2	13.3	18.6	23.7	600

[a] Nominal shear values shall be multiplied by the appropriate strength reduction factor to determine design strength as set forth in 15.4.6.5.

[b] Studs shall be a minimum 40 mm x 90 mm with a 10 mm return lip. Track shall be a minimum 30 mm x 90 mm. Both studs and track shall have a minimum uncoated base metal thickness of 0.85 mm and shall be ASTM A653 SS Grade 33, ASTM A792 SS Grade 33, or ASTM A875 SS Grade 33. Framing screws shall be No. 8 x 16 mm wafer head self-drilling. Plywood and OSB screws shall be a minimum No. 8 x 25 mm bugle head. Where horizontal straps are used to provide blocking, they shall be a minimum 40 mm wide and of the same material and thickness as the stud and track.

[c] Screws in the field of the panel shall be installed 300 mm on center unless otherwise shown.

[d] Both flanges of the studs shall be braced in accordance with Section 15.4.6.3.

CHAPTER 16
EXISTING BUILDING PROVISIONS

SECTION 16.1
ADDITIONS TO EXISTING STRUCTURES

16.1.0 Additions shall be made to existing structures only as follows:

16.1.1 Additions shall not be permitted unless the existing structures have been designed according to this code.

16.1.2 An addition that is structurally independent from an existing structure shall be designed and constructed in accordance with the seismic requirements for new structures.

16.1.3 An addition that is not structurally independent from an existing structure shall be designed and constructed such that the entire structure conforms to the seismic force-resistance requirements for new structures unless the following three conditions are complied with:

1. The addition shall comply with the requirements for new structures.
2. The addition shall not increase the seismic forces in any structural element of the existing structure by more than 5% unless the capacity of the element subject to the increased forces is still in compliance with these provisions.
3. The addition shall not decrease the seismic resistance of any structural element of the existing structure unless the reduced resistance is equal to or greater than that required for new structures.

SECTION 16.2
CHANGE OF USE

When a change of use results in a structure being reclassified to a higher Occupancy Category, the structure shall conform to the seismic requirements for new construction.

Exceptions:

1. When a change of use results in a structure being reclassified from Occupancy Category I or II to Occupancy Category III and the structure is located in a seismic map area where S_{DS} < 0.33, compliance with these provisions is not required.
2. Specific seismic detailing provisions of Chapter 15 required for a new structure are not required to be met when it can be shown that the level of performance and seismic safety is equivalent to that of a new structure. Such analysis shall consider the regularity, over-strength, redundancy, and ductility of the structure within the context of the existing and retrofit (if any) detailing provided.

SECTION 16.3
MULTIPLE USES

Structures having multiple uses shall be assigned the classification of the use having the highest Occupancy Category except in structures having two or more portions which are structurally separated in accordance with Section 10.12, each portion shall be separately classified. Where a structurally separated portion of a structure provides access to, egress from, or shares life safety components with another portion having a higher Occupancy Category, both portions shall be assigned the higher Occupancy Category.

REFERENCED STANDARDS

These are the standards referenced within SBC 301. The standards are listed herein by the promulgating agency of the standard, the standard identification, the effective date and title. The application of the referenced standards shall be as specified in SBC.

Ref. 4-1 ANSI. (1988). "American National Standard Practice for the Inspection of Elevators, Escalators, and Moving Walks (Inspectors' Manual)." ANSI A17.2.

Ref. 4-2 ANSI/ASME. (1993). "American National Standard Safety Code for Elevators and Escalators." ANSI/ASME A17.1.

Ref. 5-1 ASCE. (1998). "Flood Resistant Design and Construction." SEI/ASCE 24-98.

Ref. 7.4-1 Standard Test Method for Performance of Exterior Windows, Curtain Walls, Doors and Storm Shutters Impacted by Missile(s) and Exposed to Cyclic Pressure Differentials, ASTM E1886-97, ASTM Inc., West Conshohocken, PA, 1997.

Ref. 7.4-2 Specification Standard for Performance of Exterior Windows, Glazed Curtain Walls, Doors and Storm Shutters Impacted by Windborne Debris in Hurricanes, ASTM E 1996-99, ASTM Inc., West Conshohocken, PA, 1999.

Ref. 11.1-1 American Institute of Steel Construction, Seismic Provisions for Structural Steel Buildings, Part I, 1997, including Supplement 2, November 10, 2000.

Ref. 11.1-2 American Iron and Steel Institute (AISI), Specification for the Design of Cold-Formed Steel Structural Members, 1996, including Supplement No. 1, July 30, 1999.

Ref. 11.1-3 ASCE, Specification for the Design of Cold-Formed Stainless Steel Structural Members, ASCE 8-90, 1990.

Ref. 11.1-4 Steel Joist Institute, Standard Specification, Load Tables and Weight Tables for Steel Joists and Joist Girders, 1994.

Ref. 11.1-5 ASCE, Structural Applications for Steel Cables for Buildings, ASCE 19-95, 1995.

Ref. 11.3-1 American Institute of Steel Construction (AISC), Seismic Provisions for Structural Steel Buildings, Including Supplement No. 1 (February 15, 1999, July 1997, Parts I and II.

Ref. 11.3-2 American Iron and Steel Institute (AISI), Specification for the Design of Cold-Formed Steel Structural Members, 1996, including Supplement 2000.

Ref. 11.4-1 American Concrete Institute, Building Code Requirements for Masonry Structures, ACI 530-99/ASCE 5-99/TMS 402-99, 1999 and Specifications for Masonry Structures, ACI 530.1-99/ ASCE 6-99/TMS 602-99, 1999.

Ref. 12-1 American Society of Mechanical Engineers (ASME), ASME A17.1, Safety Code For Elevators and Escalators, 1996.

Ref. 12-2 American Society of Mechanical Engineers (ASME), Boiler And Pressure Vessel Code, including addendums through 1997.

Ref. 12-3 American Society For Testing and Materials (ASTM), ASTM C635, Standard Specification for the Manufacture, Performance, and Testing of Metal Suspension Systems For Acoustical Tile And Lay-in Panel Ceilings, 1997.

Ref. 12-4 American Society For Testing And Materials (ASTM), ASTM C636, Standard Practice for Installation of Metal Ceiling Suspension Systems for Acoustical Tile And Lay-in Panels, 1996.

Ref. 12-5 American National Standards Institute/American Society :of Mechanical Engineers, ASME B31.1-98, Power Piping.

Ref. 12-6 American Society of Mechanical Engineers, ASME B31.3-96, Process Piping.

Ref. 12-7 American Society of Mechanical Engineers, ASME B31.4-92, Liquid Transportation Systems for Hydrocarbons, Liquid Petroleum Gas, Anhydrous Ammonia, and Alcohols.

Ref. 12-8 American Society of Mechanical Engineers, ASME B31.5-92, Refrigeration Piping.

Ref. 12-9 American Society of Mechanical Engineers, ASME B31.9-96, Building Services Piping.

Ref. 12-10 American Society of Mechanical Engineers, ASME B31.11-89 (Reaffirmed 1998), Slurry Transportation Piping Systems.

Ref. 12-11 American Society of Mechanical Engineers, ASME B31.8-95, Gas Transmission and Distribution Piping Systems.

Ref. 12-12 Institute of Electrical and Electronic Engineers (IEEE), Standard 344, Recommended Practice for Seismic Qualification of Class 1E Equipment for Nuclear Power Generating Stations, 1987.

Ref. 12-13 National Fire Protection Association (NFPA), NFPA-13, Standard for the Installation of Sprinkler Systems, 1999.

Ref. 12-14 American Society of Heating, Ventilating, and Air Conditioning (ASHRAE), "Seismic Restraint Design," 1999.

Ref. 12-15 Manufacturer's Standardization Society of the Valve and Fitting Industry (MSS). SP-58, "Pipe-hangers and Supports - Materials, Design, and Manufacture," 1988.

Ref. 12-16 Ceilings and Interior Systems Construction Association (CISCA), "Recommendations for Direct-Hung Acoustical Tile and Lay-in Panel Ceilings," Seismic Zones 0-2, 1991.

Ref. 12-17 Ceilings and Interior Systems Construction Association (CISCA), "Recommendations for Direct-Hung Acoustical Tile and Lay-in Panel Ceilings," Seismic Zones 3-4, 1991.

Ref. 12-18 Sheet Metal and Air Conditioning Contractors National Association (SMACNA), HVAC Duct Construction Standards, Metal and Flexible, 1995.

Ref. 12-19 Sheet Metal and Air Conditioning Contractors National Association (SMACNA), Rectangular Industrial Duct Construction Standards, 1980.

Ref. 12-20 Sheet Metal and Air Conditioning Contractors National Association (SMACNA), Seismic Restraint Manual Guidelines for Mechanical Systems, 1991, including Appendix B, 1998.

Ref. 12-21 American Architectural Manufacturers Association (AAMA), "Recommended Dynamic Test Method for Determining the Seismic Drift Causing Glass Fallout from a Wall System," Publication No. AAMA 501.6-2001.

Ref. 13.2-1 ACI. (1995). "Standard Practice for the Design and Construction of Cast-In-Place Reinforced Concrete Chimneys." ACI 307.

Ref. 13.2-2 ACI. (1997). "Standard Practice for the Design and Construction of Concrete Silos and Stacking Tubes for Storing Granular Materials." ACI 313.

Ref. 13.2-3 ACI. (1999). "Standard Practice for the Seismic Design of Liquid-Containing Concrete Structures." ACI 350.3/350.3R.

Ref. 13.2-4 ACI. (1998). "Guide to the Analysis, Design, and Construction of Concrete-Pedestal Water Towers." ACI 371R.

Ref. 13.2-5 ANSI "Safety Requirements for the Storage and Handling of Anhydrous

Ammonia." ANSI K61.1.

Ref. 13.2-6 American Petroleum Institute (API). (December 1998). "Design and Construction of Large, Welded, Low Pressure Storage Tanks." API 620, 9th Edition, Addendum 3.

Ref. 13.2-7 API. (March 2000). "Welded Steel Tanks For Oil Storage." API 650, 10th Edition, Addendum 1.

Ref. 13.2-8 API. (December 1999). "Tank Inspection, Repair, Alteration, and Reconstruction." ANSI/API 653, 2nd Edition, Addendum 4.

Ref. 13.2-9 API. (May 1995). "Design and Construction of Liquefied Petroleum Gas Installation." ANSI/API 2510, 7th Edition.

Ref. 13.2-10 API. (February 1995). "Bolted Tanks for Storage of Production Liquids." Specification 12B, 14th Edition.

Ref. 13.2-11 ASME. (2000). "Boiler and Pressure Vessel Code." Including Addenda through 2000.

Ref. 13.2-12 ASME. (1992). "Steel Stacks." ASME STS-1.

Ref. 13.2-13 ASME. (1995). "Gas Transmission and Distribution Piping Systems" ASME B31.8.

Ref. 13.2-14 ASME. (1999). "Welded Aluminum-Alloy Storage Tanks." ASME B96.1.

Ref. 13.2-15 American Water Works Association (AWWA). (1997). "Welded Steel Tanks for Water Storage." ANSI/AWWA D100.

Ref. 13.2-16 AWWA. (1997). "Factory-Coated Bolted Steel Tanks for Water Storage." ANSI/AWWA D103.

Ref. 13.2-17 AWWA. (1995). "Wire- and Strand-Wound Circular for Prestressed Concrete Water Tanks." ANSI/AWWA D110.

Ref. 13.2-18 AWWA. (1995). "Circular Prestressed Concrete Tanks with Circumferential Tendons." ANSI/AWWA DI15.

Ref. 13.2-19 National Fire Protection Association (NFPA). (2000). "Flammable and Combustible Liquids Code." ANSI/NFPA 30.

Ref. 13.2-20 NFPA. (2001). "Storage and Handling of Liquefied Petroleum Gas." ANSI/NFPA 58.

Ref. 13.2-21 NFPA. (2001). "Storage and Handling of Liquefied Petroleum Gases at Utility Gas Plants." ANSI/NFPA 59.

Ref. 13.2-22 NFPA. (2001). "Production, Storage, and Handling of Liquefied Natural Gas (LNG)." ANSI/NFPA 59A.

Ref. 13.2-23 ASTM. (1992). "Standard Practice for the Design and Manufacture of Amusement Rides and Devices." ASTM F1159.

Ref. 13.2-24 ASTM. (1998). "Standard Guide for Design and Construction of Brick Liners for Industrial Chimneys." ASTM C1298

Ref. 13.2-25 Rack Manufacturers Institute (RMI). (1997). "Specification for the Design, Testing, and Utilization of Industrial Steel Storage Racks."

Ref. 13.2-26 U.S. Department of Transportation (DOT). "Pipeline Safety Regulations." Title 49CFR Part 193.

Ref. 13.2-27 U.S. Naval Facilities Command (NAVFAC). "The Seismic Design of Waterfront Retaining Structures." NAVFAC R-939.

Ref. 13.2-28 U.S. Naval Facilities Engineering Command. "Piers and Wharves." NAVFAC DM-25.1

Ref. 13.2-29 U.S. Army Corps of Engineers (USACE). (1992). "Seismic Design for Buildings." Army TM 5-809-10/ NAVFAC P-355/Air Force AFM 88-3, Chapter 13.

Ref. 13.2-31 Compressed Gas Association (CGA). (1999). "Guide for Flat-Bottomed LOX/LIN/LAR Storage Tank Systems." 1st Edition.

Ref. 15.2.2-1 ANSI. (1998). "Structural Welding Code-Steel." ANSI/AWS D1.1-98.

Ref. 15.2.2-2 ASTM. (1990). "Specification for Straight Beam Ultrasound Examination of Steel Plates." ASTM A435-90.

Ref. 15.2.2-3 ASTM. (1991). "Specification for Straight Beam Ultrasound Examination for Rolled Steel Shapes." ASTM A898-91.

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