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DRAFT SEISMIC STANDARD FOR FEDERAL BUILDINGS

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16. ABSTRACT *(A 200-word or less factual summary of most significant information. If document includes a significant bibliography or literature survey, mention it here.)*

This standard has been prepared for uniform use by all Federal agencies as an adaptation of existing voluntary standards, model building codes, Federal regulations, and research reports. It is closely based on the seismic requirements of the Uniform Building Code (which, in turn, are based on the Recommended Lateral Force Requirements and Commentary published by the Structural Engineers Association of California). However, there are many instances of substantive differences from the UBC. Several important provisions, including the seismic zoning map, have been adapted from the Tentative Provisions for the Development of Seismic Regulations for Buildings developed by the Applied Technology Council. A number of provisions have been added to this standard that are based on the current practices and policies of various Federal agencies. Furthermore, this standard is organized considerably differently from the UBC and many provisions are phrased differently.

17. KEY WORDS *(six to twelve entries; alphabetical order; capitalize only the first letter of the first key word unless a proper name; separated by semicolons)*

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PREFACE

This draft standard was developed within the Subcommittee on Building Standards of the Interagency Committee on Seismic Safety in Construction (ICSSC). The membership of the Subcommittee is:

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Lincoln Chang	Department of Housing and Urban Development
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* denotes Chairman

The able editorial and secretarial assistance of Ms. Sandra Berry of the National Bureau of Standards deserves recognition.

The Subcommittee has recommended that this draft be submitted to all concerned agencies with the request that they test its implementation through use in design, contract administration, and quality control, either on a trial or real basis during 1981. Following the trial implementation, the Subcommittee plans to review the draft, revise as necessary, and then recommend its adoption by the Interagency Committee as a standard for Federal Buildings. Comment on this draft standard is welcomed. Comment should be forward to:

Dr. E. V. Leyendecker
Chairman, ICCSSC Subcommittee #2
Room B168, Building 226
National Bureau of Standards
Washington, D.C. 20234

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ABSTRACT

This standard has been prepared for uniform use by all Federal agencies as an adaptation of existing voluntary standards, model building codes, Federal regulations, and research reports. It is closely based on the seismic requirements of the Uniform Building Code (which, in turn, are based on the Recommended Lateral Force Requirements and Commentary published by the Structural Engineers Assocation of California). However, there are many instances of substantive differences from the UBC. Several important provisions, including the seismic zoning map, have been adapted from the Tentative Provisions for the Development of Seismic Regulations for Buildings developed by the Applied Technology Council. A number of provisions have been added to this standard that are based on the current practices and policies of various Federal agencies. Furthermore, in the spirit of improvement, this standard is organized considerably differently from the UBC and many provisions are phrased differently.

SI Conversion Units

The following list of conversion factors for the most frequently used quantities in building design and construction may be used.

QUANTITY	INTERNATIONAL (SI) UNIT	U.S. CUSTOMARY UNIT	APPROXIMATE CONVERSION	
LENGTH	meter (m)	foot (ft)	1 m	= 3.2808 ft
	millimeter (mm)	inch (in)	1 mm	= 0.0394 in
AREA	square meter (m^2)	square yard (yd^2)	1 m^2	= 1.1960 yd^2
		square foot (ft^2)	1 m^2	= 10.764 ft^2
	square millimeter (mm^2)	square inch (in^2)	1 mm^2	= 1.55×10^{-3} in^2
VOLUME	cubic meter (m^3)	cubic yard (yd^3)	1 m^3	= 1.3080 yd^3
		cubic foot (ft^3)	1 m^3	= 35.315 ft^3
	cubic millimeter (mm^3)	cubic inch (in^3)	1 mm^3	= 61.024×10^{-6} in^3
CAPACITY	liter (L)	gallon (gal)	1 L	= 0.2642 gal
	milliliter (mL)	fluid ounce (fl oz)	1 mL	= 0.0338 fl oz
VELOCITY, SPEED	meter per second (m/s)	foot per second (ft/s or f.p.s.)	1 m/s	= 3.2808 ft/s
	kilometer per hour (km/h)	mile per hour (mile/h or m.p.h.)	1 km/h	= 0.6214 mile/h
ACCELERATION	meter per second squared (m/s^2)	foot per second squared (ft/s^2)	1 m/s^2	= 3.2808 ft/s^2
MASS	metric ton (t) [1000 kg]	short ton [2000 lb]	1 t	= 1.1023 ton
	kilogram (kg)	pound (lb)	1 kg	= 2.2046 lb
	gram (g)	ounce (oz)	1 g	= 0.0353 oz
DENSITY	metric ton per cubic meter (t/m^3)	ton per cubic yard (ton/yd^3)	1 t/m^3	= 0.8428 ton/yd^3
	kilogram per cubic meter (kg/m^3)	pound per cubic foot (lb/ft^3)	1 kg/m^3	= 0.0624 lb/ft^3
FORCE	kilonewton (kN)	ton-force (tonf)	1 kN	= 0.1124 tonf
		kip [1000 lbf]	1 kN	= 0.2248 kip
	newton (N)	pound-force (lbf)	1 N	= 0.2248 lbf
MOMENT OF FORCE, TORQUE	kilonewton meter (kN•m)	ton-force foot (tonf•ft)	1 kN•m	= 0.3688 tonf•ft
	newton meter (N•m)	pound-force inch (lbf•in)	1 N•m	= 8.8508 lbf•in
PRESSURE, STRESS	megapascal (MPa)	ton-force per square inch ($tonf/in^2$)	1 MPa	= 0.0725 $tonf/in^2$
		ton-force per square foot ($tonf/ft^2$)	1 MPa	= 10.443 $tonf/ft^2$
	kilopascal (kPa)	pound-force per square inch (lbf/in^2)	1 kPa	= 0.1450 lbf/in^2
		pound-force per square foot (lbf/ft^2)	1 kPa	= 20.885 lbf/ft^2
WORK, ENERGY, QUANTITY OF HEAT	megajoule (MJ)	kilowatthour (kWh)	1 MJ	= 0.2778 kWh
	kilojoule (kJ)	British thermal unit (Btu)	1 kJ	= 0.9478 Btu
	joule (J)	foot pound-force (ft•lbf)	1 J	= 0.7376 ft•lbf
POWER, HEAT FLOW RATE	kilowatt (kW)	horsepower (hp)	1 kW	= 1.3410 hp
	watt (W)	British thermal unit per hour (Btu/h)	1 W	= 3.4121 Btu/h
		foot pound-force per second (ft•lbf/s)	1 W	= 0.7376 ft•lbf/s
COEFFICIENT OF HEAT TRANSFER [U-value]	watt per square meter kelvin ($W/m^2 \cdot K$) [= ($W/m^2 \cdot °C$)]	Btu per square foot hour degree Fahrenheit ($Btu/ft^2 \cdot h \cdot °F$)	1 $W/m^2 \cdot K$	= 0.1761 $Btu/ft^2 \cdot h \cdot °F$
THERMAL CONDUCTIVITY [k-value]	watt per meter kelvin (W/m•K) [= (W/m•°C)]	Btu per square foot degree Fahrenheit ($Btu/ft^2 \cdot °F$)	1 W/m•K	= 0.5778 $Btu/ft^2 \cdot °F$

NOTES: (1) The above conversion factors are shown to three or four places of decimals.

(2) Unprefixed SI units are underlined. (The kilogram, although prefixed, is an SI base unit.)

REFERENCES:

NBS Guidelines for the Use of the Metric System, LC1056, Revised August 1977;

The Metric System of Measurement, Federal Register Notice of October 26, 1977, LC 1078, Revised November 1977;

NBS Special Publication 330, "The International System of Units (SI)," 1977 Edition;

NBS Technical Note 938, "Recommended Practice for the Use of Metric (SI) Units in Building Design and Construction," Revised edition June 1977;

ASTM Standard E621-78, "Standard Practice for the Use of Metric (SI) Units in Building Design and Construction," (based on NBS TN 938), March 1978;

ANSI Z210.1-1976, "American National Standard for Metric Practice."

ASTM E380-79[c], "Standard for Metric Practice."

IEEE Std. 268-1979, "Standard for Metric Practice."

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FOREWORD

This standard has been prepared by the Interagency Committee on Seismic Safety in Construction as one part of the National Earthquake Hazards Reduction Program, the President's plan to implement the Earthquake Hazards Reduction Act of 1977 (Public Law 95-124). The standard contains provisions for the reduction of earthquake hazards in Federal buildings. It is intended to be uniformly applied by all Federal agencies for the planning, design, and construction of buildings, both within and outside the United States. The Public Law also encourages agencies to define comprehensive screening and evaluation programs for existing buildings and establish priorities and programs for corrective actions. Guidelines for such programs for existing buildings are included in another document,* also prepared by the Interagency Committee.

The standard has been prepared for uniform use by all Federal agencies and is an adaptation of existing voluntary standards, model building codes, Federal regulations, and research reports. The standard is most closely related to the Uniform Building Code, published by the International Conference of Building Officials, Whittier, CA, 1979, which, in turn, is based on the Recommended Lateral Force Requirements and Commentary, published by the Structural Engineers Association of California, San Francisco, CA, 1975. The use made of the following document also deserves special note: Tentative Provisions for the Development of Seismic Regulations for Buildings, prepared by the Applied Technology Council and published by the National Bureau of Standards, Washington, DC, 1978.

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* In preparation by Subcommittee 3.

1. REGULATION

1.1 SCOPE. This standard shall be used in the planning, design, and construction of new buildings and their appurtenances and in the alteration, repair, or change of use of existing buildings. This standard also establishes criteria for the evaluation of the seismic safety of existing buildings and for the performance of corrective actions. This standard is intended to provide varying degrees of safety in buildings against the effects of seismic ground shaking.

1.2 DEFINITIONS. The following definitions provide the meaning of the terms used in this standard.

approved, as to materials and types of construction, refers to approval by the designated authority as the result of investigation and tests conducted by him, or by reason of accepted principles or tests by national authorities, technical or scientific organizations.

base is the level at which the earthquake motions are considered to be imparted to the structure.

bearing wall is any wall meeting either of the following classifications:

(1) any metal or wood stud wall which supports more than 100 pounds per lineal foot of superimposed load

(2) any masonry or concrete wall which supports more than 200 pounds per lineal foot of superimposed load, or any such wall supporting its own weight for more than one story.

box system see 3.1.2.

braced frame is a truss system or its equivalent which is provided to resist lateral seismic forces in the frame system and in which the members are subjected primarily to axial stresses by the seismic forces.

building is any structure used or intended for supporting or sheltering any human use or occupancy.

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

designated authority is the official representative of the government who is charged with the administration and enforcement of this standard, or his duly authorized representative.

diaphragm is a horizontal, or nearly horizontal, component or system, including bracing systems, that is designed to transmit lateral seismic forces to the vertical elements of the seismic force resisting system.

dual system see 3.1.2.

ductile moment-resisting space frame is a moment-resisting (unbraced) space frame with special ductility provisions to permit repeated inelastic straining.

ductile moment-resisting space frame system see 3.1.2.

essential facilities see 1.4.2.

exterior wall is any wall or element of a wall, or any member or group of members, which defines the exterior boundaries or courts of a building and which has a slope of 60 degrees or greater with the horizontal plane.

height is the vertical distance from the base to uppermost level in the structure unless otherwise indicated.

live load is the load superimposed by the use and occupancy of the building not including the wind load, earthquake load, or dead load.

moment-resisting space frame for lateral forces is an unbraced vertical load-carrying space frame in which members are restrained at the joints and arranged so that they are subjected primarily to flexural stresses.

parapet wall is that part of any wall entirely above the roof line.

seismic force-resisting system is that part of the structural system assigned to resist seismic forces.

shear wall is a wall designed to resist lateral forces parallel to the wall.

space frame is a three-dimensional structural system without bearing walls, laterally supported so as to function as a complete self-contained unit. It may or may not included horizontal diaphragms or floor-bracing systems.

structure is an edifice or building of any kind, or any piece of work artificially built up or composed of parts joined together in some definite manner.

1.3 NOTATION. The following notations are used in this standard.

C = Numerical coefficient for the vibratory response of a structure to seismic motions

$\overline{C}$ = Value of C used in calculating a lower limit for the base shear

C_m = Value of C for the m^{th} specific mode of vibration

C_p = Numerical coefficient for the vibratory response of a part of a structure to seismic motions

D = The dimension of the structure, in feet, in a direction parallel to the applied forces.

δ_i = Deflection at level i relative to the base, due to arbitrary lateral forces, Σf_i for use in determining the period of vibration, T

F_i, F_n, F_x = Lateral seismic force applied to level i, n, or x, respectively

F_p = Lateral seismic forces on a part of the structure

F_t = That portion of V considered concentrated at the top of the structure in addition to F_n

F_{xm} = Lateral seismic force applied to level x when considering the m^{th} mode of vibration

f_i = Distributed portion of an arbitrary lateral force at level i for use in determining the period of vibration, T

g = Acceleration due to gravity

h_i, h_n, h_x = Height in feet above the base to level i, n, or x, respectively

I = Numerical coefficient for occupancy hazard

K = Numerical coefficient for structural system response to seismic motions

L_D = the effect of dead load

L_E = the effect of lateral seismic load

L_L = the effect of live load

L_S = the effect of snow load

Level i, n, x:

i = Level of the structure referred to by the subscript i

n = That level which is uppermost in the main portion of the structure

x = That level which is under design consideration

i or x = 1 designates the first level above the base

M_{xm} = Overturning moment at level x when considering the m^{th} mode of vibration

N = The total number of stories above the base to level n

S = Numerical coefficient for site-structure resonance in response to seismic motions

T = Fundamental elastic period of vibration of the building or structure in seconds in the direction under consideration

$\overline{T}$ = Value of T used in calculating a lower limit for the base shear

T_m = Period of vibration for the m^{th} mode, in seconds

T_s = Characteristic site period

V = The total lateral seismic force or shear at the base

$\overline{V}$ = A lower limit of V when using modal analysis

V_m = The value of V when considering the m^{th} mode of vibration

V_t = The total lateral seismic force or shear at the base determined by means of modal analysis

V_{xm} = The total lateral seismic force at level x when considering the m^{th} mode of vibration

W = The total dead load including the partition loading where applicable; see 4.2.1

W_m = The portion of W effective for the m^{th} mode of vibration

W_p = The weight of a portion of a structure or nonstructural component

w_i, w_x = That portion of W which is located at or is assigned to level i or x, respectively

Z = Numerical coefficient for the seismic hazard zone

ϕ_{im}, ϕ_{xm} = The displacement amplitude at level i, or x, of the building for the fixed-based condition when vibrating in its m^{th} mode.

1.4 HAZARD CLASSIFICATIONS

1.4.1 Seismic Ground Shaking Hazard. The zone representing the level of seismic ground shaking hazard shall be established for each building site according to the maps shown in figure 1. For locations not included in figure 1, the zone shall be established from approved documents or from a site evaluation as indicated in the following paragraph.

For those building sites that have had a design ground acceleration established by means of an approved site evaluation (refer to 2.2.1), the zone shall be determined according to table 1:

Table 1

Zone for Sites with Approved Site Evaluation

Design ground acceleration, a	Zone
$a < 0.05g$	0
$0.05g \leq a < 0.10g$	1
$0.10g \leq a < 0.20g$	2
$0.20g \leq a < 0.40g$	3
$0.40g \leq a$	4

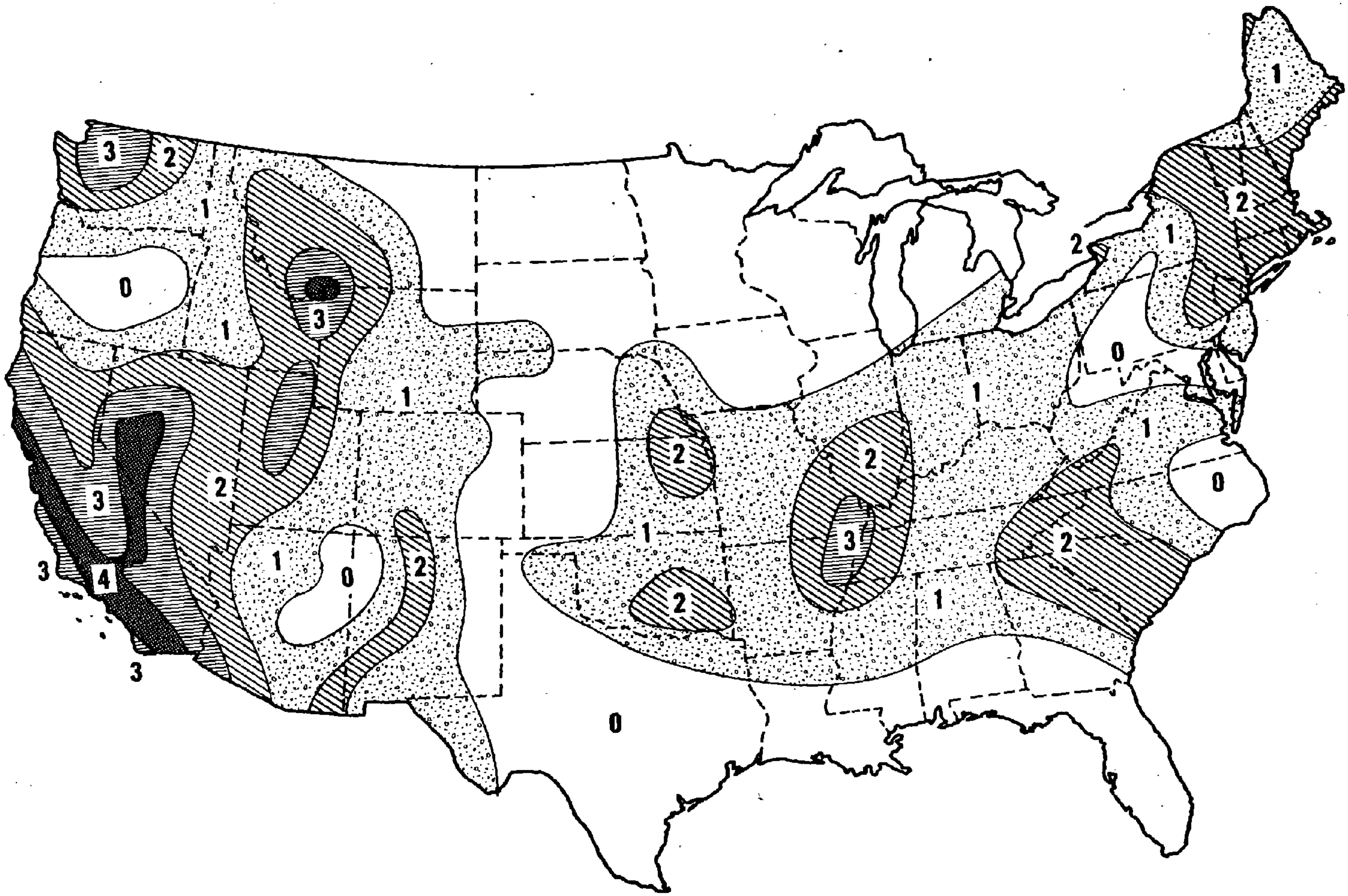

Figure 1. SEISMIC ZONE MAP

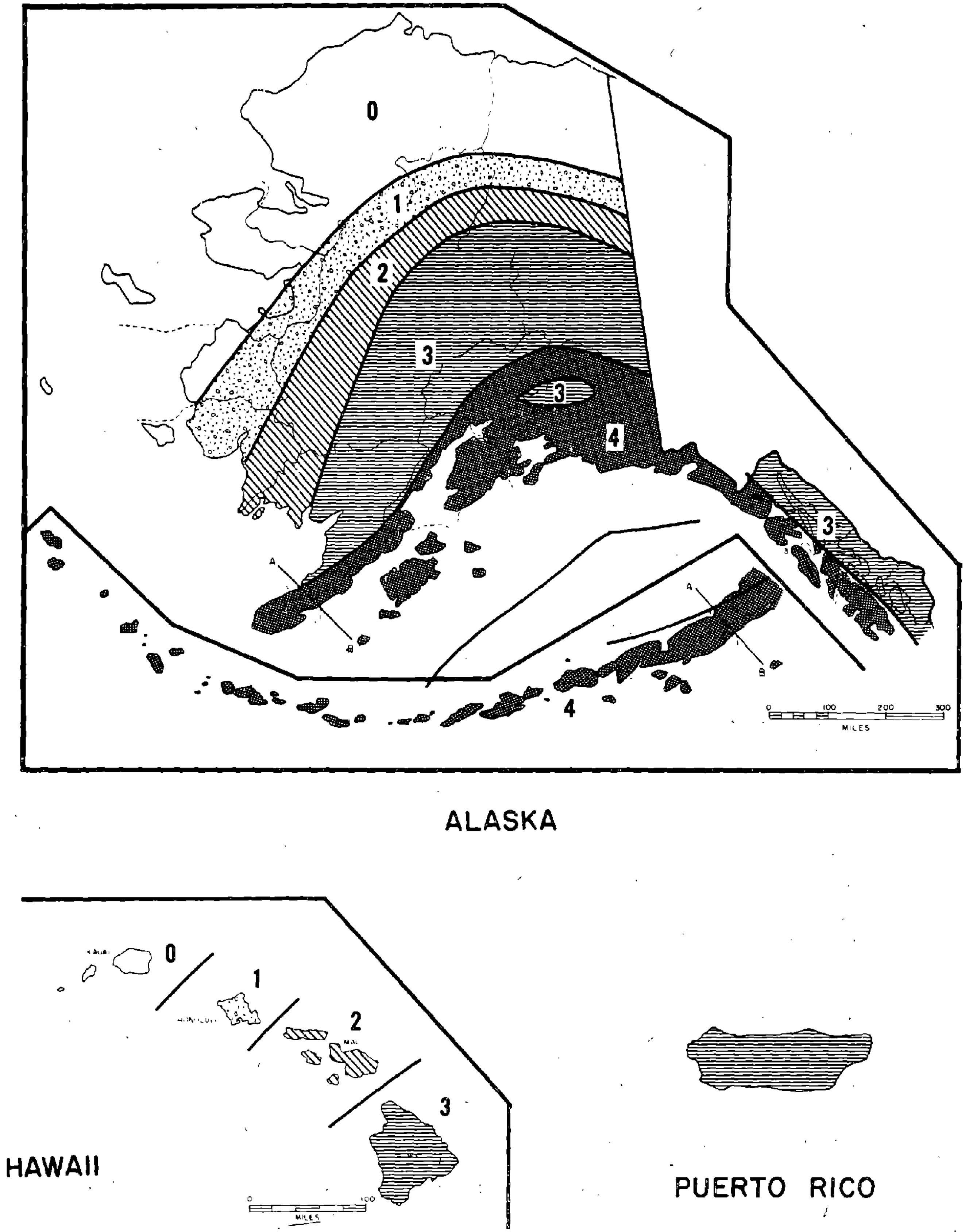

Figure 1. SEISMIC ZONE MAP

1.4.2 Occupancy Hazard. The occupancy of the building shall be classified and the occupancy importance factor I shall be established for each building according to table 2, unless otherwise specified by the designated authority:

Table 2

Values for Occupancy Importance Factor

Type of Occupancy	I
Essential Facilitites. Essential facilities are those buildings or their appurtenances which must be safe and usable for emergency purposes after a major earthquake in order to preserve the health and safety of the general public. Such facilities shall include but not be limited to: 1. Hospitals and other medical facilities having surgery or emergency treatment areas 2. Fire and police stations 3. Government disaster operation and communication centers deemed to be vital in emergencies 4. Power stations and other utilities required as emergency facilities	1.5
High Risk. Any building where the primary occupancy is for assembly use for more than 300 persons, or where the occupants' mobility is restricted or impaired, or where the contents of the building are hazardous	1.25
All Other	1.0

1.5 ALTERNATIVE PROVISIONS. Alternate materials, methods of construction, structural concepts, and analytical procedures to those prescribed in this standard may be used subject to the approval of the designated authority.

Substantiating evidence demonstrating that the proposed alternate, for the purpose intended, will be at least equal in strength, durability, seismic resistance, validity, and safety shall be submitted.

2. GENERAL REQUIREMENTS

2.1 GENERAL PERFORMANCE REQUIREMENT. Every building and the appurtenances and parts thereof shall be designed and constructed to resist the forces produced by seismic ground shaking as provided in this standard, except as may be specified by the designated authority. Buildings located in zone 0 are exempt from the requirements in this standard.

2.2 SITE PLANNING FOR NEW BUILDINGS

2.2.1 Site Evaluation Study. A site evaluation study shall be conducted for all hospitals in zones 3 or 4 and other buildings specified by the designated authority. The study shall establish the design ground motion for the site and shall evaluate the likelihood and effect of the following phenomena:

(1) surface rupture due to active fault displacement

(2) liquefaction

(3) landslide or slope stability failure

(4) subsidence

2.2.2 Site Limitations. This section applies only to those buildings for which a site evaluation study is required. Essential facilities shall not be sited such that surface rupture due to fault displacement would pass through the building. For sites that have a potential for liquefaction, landslide, or subsidence, the building, foundation, and/or subsoil shall be engineered to mitigate the hazard or the effects of the phenomena.

2.3 DESIGN OF NEW BUILDINGS

2.3.1 Application of the Provisions. The structural portion of the building shall be designed to satisfy the requirments of chapter 3, STRUCTURAL DESIGN CRITERIA. For all buildings in zones 3 or 4 and buildings with an importance factor I greater than 1.0 in zone 2, the nonstructural portion of the building shall be designed to satisfy the requirements of chapter 7, NONSTRUCTURAL DESIGN REQUIREMENTS.

2.3.2 Documentation. Drawings, specifications, basis of design, calculations, reports, certifications, and other substantiation necessary to verify compliance with the design provisions shall be submitted to the designated authority.

2.4 CONSTRUCTION. The construction quality of all buildings in zones 3 or 4 and buildings with an importance factor I greater than 1.0 in zone 2 shall be assured by satisfying the requirements of chapter 8, CONSTRUCTION QUALITY CONTROL.

2.5 EXISTING BUILDINGS

2.5.1 Alterations and Repairs. When specified by the designated authority any Federal building for which the cost of renovations or repairs, exclusive of seismic strengthening, exceeds 25 percent of the replacement cost of the improved building must be corrected to resist the appropriate level of earthquake forces, as recommended by the provisions of *__________ __________________, also prepared by the Interagency Committee.

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* In Preparation by Subcommittee 3.

Minor structural alterations may be made in existing buildings and other structures, but the resistance to lateral seismic forces shall be not less than that before such alterations were made, unless the building as altered meets the requirements of this standard.

2.5.2 Changes of Use or Location. Changes of use that increase the occupancy importance factor I, or of location that increase the seismic hazard zone shall be permitted only if the building is made to satisfy the requirements of this standard.

3. STRUCTURAL DESIGN CRITERIA

3.1 SEISMIC FORCE-RESISTING SYSTEMS

3.1.1 Integrity. The seismic force-resisting system shall include a continuous load path, or paths, to transfer all seismic forces to the final point of resistance. Connections and elements shall be provided to transfer the seismic forces from other parts of the building to the seismic force-resisting system, using the forces specified in 4.6 and 7.1, where applicable.

3.1.2 System Response Classification. Each building shall be assigned to one of the following categories based on the type of elements used to support gravity loads and on the type and ductility of elements designated to be the seismic force-resisting system:

(1) Ductile Moment-Resisting Space Frame System ($K = 0.67$) - a system in which essentially all the gravity load is supported on framing without the use of bearing walls and in which the designated seismic force-resisting system in the direction under consideration is composed entirely of reinforced concrete or structural steel ductile moment-resisting space frames (unbraced frames).

(2) Dual System ($K = 0.80$) - a system in which essentially all the total gravity load is supported on framing without the use of bearing walls and in which the designated seismic force-resisting system in the direction under consideration is composed of a combination of reinforced concrete or structural steel ductile moment-resisting space frames with shear walls or braced frames.

(3) Box System (K = 1.33) - a system in which a significant fraction of the gravity load is supported on bearing walls.

(4) Building Frame System (K = 1.0) - any other structural system.

Rigid elements that are not designated as part of the seismic force-resisting system may be incorporated into buildings provided that their effect on the action of the seismic force-resisting system is considered and provided for in the design. In particular, moment-resisting space frames and ductile moment-resisting space frames may be enclosed by or adjoined by other rigid elements if it can be shown that the action or failure of the other rigid elements will not impair the vertical and lateral load resisting ability of the space frame.

Structural steel ductile moment-resisting space frames shall satisfy the ductility provisions of 6.2, and reinforced concrete ductile moment-resisting space frames shall satisfy the ductility provisions of 6.3.

3.1.3 Strength. Members and connections shall resist the effect of combined gravity and seismic forces. The resistance shall be determined according to chapter 5, DESIGN AND CONSTRUCTION STANDARDS FOR STRUCTURAL MATERIALS. In computing the effect of seismic force in combination with vertical loads, gravity load stresses induced in members by dead load plus design live load and snow load, except roof live load, shall be considered. Consideration shall also be given to minimum gravity loads acting in combination with seismic forces.

In addition, the following special requirements apply to the ductile moment-resisting space frame system and the dual system:

(1) Ductile Moment-Resisting Space Frame Systems (K = 0.67): The ductile moment-resisting space frames shall have the capacity to resist the total required lateral seismic force by themselves.

(2) Dual Systems (K = 0.80): (i) The frames and shear walls shall resist the total lateral force in accordance with their relative rigidities considering the interaction of the shear walls and frames. (ii) The shear walls acting independently of the ductile moment-resisting portions of the space frame shall resist the total required seismic forces. (iii) The ductile moment-resisting space frame shall have the capacity to resist not less than 25 percent of the total required lateral seismic force.

3.1.4 Stiffness and Building Separations. Lateral deflections or drift of each story relative to its adjacent stories due to seismic forces as determined in 4.4.7 shall not exceed 0.015 times the story height, unless it can be demonstrated that greater drift can be tolerated. Diaphragm deformations shall be considered in the design of the supported walls.

All portions of structures shall be designed and constructed to act as an integral unit in resisting horizontal forces unless separated structurally to avoid contact under deflection from seismic action.

3.1.5 Overturning Stability. Every building shall be designed to resist the overturning effects caused by the seismic forces specified in this standard.

3.1.6 Height. All buildings in zones 3 or 4 and buildings with an importance factor I greater than 1.0 in zone 2 that are more than 160 feet in height shall have ductile moment-resisting space frames capable of resisting not less than 25 percent of the required seismic forces for the structure as a whole.

3.2 OTHER STRUCTURAL ELEMENTS

3.2.1 Strength and Anchorage. Structural elements and their anchorages shall resist the seismic forces induced by their own mass and by connected elements, as determined in 4.6.1. Resistance shall be determined in accordance with chapter 5, DESIGN AND CONSTRUCTION STANDARDS FOR STRUCTURAL MATERIALS.

3.2.2 Compatibility. For all buildings in zones 3 or 4 and buildings with an importance factor I greater than 1.0 in zone 2, all framing elements not required by design to be part of the seismic force-resisting system shall be investigated and shown to be adequate for vertical load-carrying capacity and induced moment due to the force effects of the distortions calculated according to 4.4.7. The rigidity of other elements shall be considered in accordance with 4.4.1.

3.3 FOUNDATIONS

3.3.1 Soil and Foundation Capacity. In the determination of the foundation design criteria, recognition shall be given to the dynamic nature of the forces, the expected ground motions, and the design basis for strength and ductility of the structure.

3.3.2 Structural Ductility. For all buildings in zones 3 or 4 and buildings with an importance factor I greater than 1.0 in zone 2 that have a dual system or a ductile moment-resisting space frame system, the special ductility requirements for structural steel or reinforced concrete specified in 6.2 and 6.3 shall apply to all structural elements below the base which are required to transmit forces to the foundation resulting from the application of the design lateral seismic forces to the building.

4. STRUCTURAL ANALYSIS PROCEDURES

Stresses shall be calculated as the effect of a force applied horizontally at each floor and roof level above the base. The force shall be assumed to come from any horizontal direction; buildings shall be analyzed for the force on each principal axis.

4.1 REQUIRED METHOD FOR SEISMIC-RESISTING SYSTEMS. The seismic loads for all buildings shall be analyzed according to 4.2, as a minimum, except for the following buildings, which shall be analyzed according to 4.3 as a minimum:

(1) buildings specified by the designated authority

(2) hospitals in zones 3 or 4 that have shapes or framing systems irregular in a vertical sense

(3) buildings over 4 stories in zone 4 that have shapes or framing systems irregular in a vertical sense.

For buildings with irregular shapes or framing systems in a horizontal sense, special attention should be given to the distribution of forces; the designated authority may require the use of a more sophisticated analysis for such buildings.

The designated authority may require or approve the use of a soil-structure interaction analysis that will modify the seismic forces and displacements determined in this chapter. The seismic load effects shall be analyzed according to 4.4. The seismic loads and effects may be analyzed by other methods according to the limitations given in 4.5.

4.2 ELASTIC STATIC LOAD ANALYSIS

4.2.1 Base Shear. The total lateral seismic force assumed to act at the base of the structure shall be determined as follows for each main axis:

$$V = ZIKCSW \qquad \text{(Eq. 1)}$$

where the terms are as follows:

Z shall be determined from table 3 for buildings that do not have an approved site evaluation:

Table 3

Zone Coefficient Z

Zone	Z
4	1
3	3/4
2	3/8
1	3/16
0	0

For buildings that have a design ground acceleration established according to an improved site evaluation study, Z = 2.5 times the design ground acceleration (see 2.2.1) expressed as a fraction of the acceleration of gravity.

I is specified in 1.4.2

K shall be determined from table 4 for buildings:

Table 4

K for Buildings

System Response Classification (see 3.1.2)	K
Ductile Moment-Resisting Space Frame	0.67
Dual System	0.80
Box System	1.33
Building Frame System	1.0

For other structures and appurtenances associated with buildings K shall be determined from table 5:

Table 5

K for Other Structures

Type of Structure	K
Elevated tanks plus full contents, on four or more cross-braced legs and not supported by a building	2.5
Structures other than buildings and other than those set forth in 4.6.1	2.00

For elevated tanks, the minimum value of KC shall be 0.12 and the maximum value of KCS need not exceed 0.30. Elevated tanks which are supported by buildings or are not supported on four or more cross-braced legs as described above shall be designed in accordance with 4.6, using $C_p = 0.3$.

C shall be determined as:

$$C = \frac{1}{15\sqrt{T}} \qquad \text{(Eq. 2)}$$

The value of C need not exceed 0.12. The value of T is specified in 4.2.2.

S shall be determined in accordance with 4.2.3, except that the product CS need not exceed 0.14.

W shall be equal to the total dead load. For storage and warehouse occupancies, 25 percent of the floor live load shall be included in W. Where the design snow load is 30 psf or less, no part need be included in the value of W. Where the snow load is greater than 30 psf, the snow load shall be included; however, the snow load may be reduced by up to 75 percent, where the snow load duration warrants and the designated authority approves the reduction.

4.2.2 Period of Vibration. The value of T for buildings shall be determined as:

$$T = \frac{0.05h_n}{\sqrt{D}} \quad \text{(Eq. 3)}$$

except for buildings in which the lateral force-resisting system consists of ducile moment-resisting space frames capable of resisting 100 percent of the required lateral forces and such system is not enclosed by or adjoined by more rigid elements tending to prevent the frame from resisting lateral forces, in which case T shall be determined as:

$$T = 0.10N \quad \text{(Eq. 4)}$$

where N is the total number of stories above the base.

The period T may be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis, which could make use of an equation such as:

$$T = 2\pi \sqrt{\left(\sum_{i=1}^{n} w_i \delta_i^2\right) \div \left(g \sum_{i=1}^{n} f_i \delta_i\right)} \qquad \text{(Eq. 5)}$$

where the values of f_i represent any lateral force distributed approximately in accordance with the principles of 4.2.4 or any other rational distribution. The elastic deflections, δ_i, shall be calculated using the applied lateral forces, f_i. However, the value of T so determined shall not exceed the value calculated by the appropriate equation 3 or 4 by more than 20 percent.

4.2.3 Site Coefficient. The value of S shall be determined by one of the following two methods.

Method I

The effects of site conditions on building response shall be established based on soil profile types defined as follows:

(1) Soil Profile Type 1 is a profile with:

(a) rock of any characteristic, either shale-like or crystalline in nature. Such material may be characterized by a shear wave velocity greater than 2,500 feet per second; or,

(b) stiff soil conditions where the soil depth is less than 200 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

(2) Soil Profile Type 2 is a profile with deep cohesionless or stiff clay conditions, including sites where the soil depth exceeds 200 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

(3) Soil Profile Type 3 is a profile with soft- to medium-stiff clays and sands, characterized by 30 feet or more of soft- to medium-stiff clays with or without intervening layers of sand or other cohesionless soils.

In locations where the soil properties are not known in sufficient detail to determine the soil profile type or where the profile does not fit any of the three types, Soil Profile Type 2 shall be used.

S is a coefficient for the effects of site conditions on building response given in table 6:

Table 6

Site Coefficient

Soil Profile Type	S
1	1.0
2	1.2
3	1.5

In zones 3 and 4, the product CS for Soil Profile Type 3 need not exceed 0.11.

Method II

The value of S shall be determined by the following equations, but shall not be less than 1.0:

For T/T_s = 1.0 or less

$$S = 1.0 + \left(\frac{T}{T_s}\right) - 0.5 \left(\frac{T}{T_s}\right)^2 \qquad \text{(Eq. 6)}$$

For T/T_s greater than 1.0

$$S = 1.2 + 0.6 \frac{T}{T_s} - 0.3 \left(\frac{T}{T_s}\right)^2 \qquad \text{(Eq. 7)}$$

where

T in equations 6 and 7 shall be as determined in 4.2.2 but T shall not be less than 0.3 second.

T_s The range of values of T_s may be established from properly substantiated geotechnical data, except that T_s shall not be taken as less than 0.5 second nor more than 2.5 seconds. T_s shall be that value within the range of site periods, as determined above, that is nearest to T.

When T_s is not properly established, the value of S shall be 1.5.

Exception: Where T has been established by a properly substantiated analysis and exceeds 2.5 seconds, the value of S may be determined by assuming a value of 2.5 seconds for T_s.

4.2.4 Vertical Distribution of Forces. The distribution of the seismic lateral forces in structures which have highly irregular shapes, large differences in lateral resistance or stiffness between adjacent stories, or other unusual structural features, shall be determined considering the dynamic characteristics of the structure. Buildings having setbacks wherein the plan dimension of the tower in each direction is at least 75 percent of the corresponding plan dimension of the lower part may be considered as uniform buildings without setbacks for the purpose of this section, provided other irregularities do not exist.

For uniform buildings, the distribution of the lateral seismic forces shall be as follows:

The total lateral force V shall be distributed over the height of the structure as follows:

$$V = F_t + \sum_{i=1}^{n} F_i \qquad \text{(Eq. 8)}$$

The concentrated force at the top shall be determined as:

$$F_t = 0.07TV \qquad \text{(Eq. 9)}$$

F_t need not exceed 0.25V and may be considered as zero where T is 0.7 second or less. The remaining portion of the total base shear V shall be distributed over the height of the structure, including level n, as follows:

$$F_x = \frac{(V - F_t)\ w_x h_x}{\sum_{i=1}^{n} w_i h_i} \qquad \text{(Eq. 10)}$$

4.2.5 Overturning Moment. The overturning moment at each story shall be calculated as follows:

$$M_x = F_t\ (h_n - h_x) + \sum_{i=x}^{n} F_i\ (h_i - h_x) \qquad \text{(Eq. 11)}$$

For structures in which the seismic resisting system acts essentially as an isolated cantilever, the overturning moment shall be modified to account for the dynamic characteristics of the structure.

4.3 ELASTIC DYNAMIC LOAD ANALYSIS. The lateral seismic forces shall be determined in accordance with this section using all modes of vibration with a period greater than 0.4 second, but in no case less than the first three modes in either principal direction.

4.3.1 Mode Shapes and Periods. The mode shape, ϕ_{im}, and the modal period, T_m shall be determined for each mode in accordance with the principles of mechanics.

4.3.2 Modal Base Shear. The total lateral seismic force for a mode shall be determined as follows:

$$V_m = ZIKC_mSW_m \qquad \text{(Eq. 12)}$$

where

Z, I, K and S are as defined in 4.2

C_m shall be determined for each mode as:

$$C_m = \frac{1}{15\sqrt{T_m}} \qquad \text{(Eq.13)}$$

The value of C_m need not exceed 0.12 and the product of C_mS need not exceed 0.14. T_m is the period of vibration for the m^{th} mode, in seconds. Subject to the approval of the designated authority, a site specific spectral shape may be used in lieu of Eq. 13.

W_m shall be determined for each mode as:

$$W_m = \frac{\left(\sum_{i=1}^{n} w_i\phi_{im}\right)^2}{\sum_{i=1}^{n} w_i\phi^2_{im}} \qquad \text{(Eq. 14)}$$

4.3.3 Design Values. The following quantities shall be determined for each mode according to the principles of mechanics:

(1) F_{xm} - the equivalent lateral seismic force applied to each level

(2) ϕ_{xm} - the lateral displacement at each level

(3) V_{xm} - the lateral seismic shear force at each level

(4) M_{xm} - the overturning moment at each level

(5) the story drift at each level

The total value for each of the above quantities and the total base shear V_t shall not be less than that obtained as the square root of the sum of the squares of the quantity for each mode. V_t shall be compared with the quantity $\overline{V}$,

where

$\overline{V}$ is determined from equation 1 by substituting $\overline{C}$ for C

$\overline{C}$ is determined from equation 2 by substituting $\overline{T}$ for T

$\overline{T}$ is determined as 1.4 times the value determined for T in equation 3 or 4, as appropriate

Where V_t is less than $\overline{V}$, the design value for each quantity shall be the product of the total value and the following factor, A:

$$A = \overline{V}/V_t \qquad \text{(Eq. 15)}$$

V_t need not exceed V determined from equation 1.

4.4 ELASTIC LOAD EFFECT ANALYSIS

4.4.1 Shear. Total shear in any horizontal plane shall be distributed to the various elements of the seismic force-resisting system in proportion to their rigidities considering the rigidity of the horizontal bracing system or diaphragm.

4.4.2 Horizontal Torsion. At each level designated as x, the force F_x shall be applied over the area of the building in accordance with the mass distribution on that level.

Provisions shall be made for the increase in shear resulting from the horizontal torsion due to an eccentricity between the center of mass and the center of rigidity. Negative torsional shears shall be neglected. Where the vertical resisting elements depend on diaphragm action for shear distribution at any level, the shear-resisting elements shall be capable of resisting a torsional moment assumed to be equivalent to the story shear acting with an eccentricity of not less than 5 percent of the maximum building dimension at that level.

4.4.3 Diaphragm Forces. Floor and roof diaphragms shall be designed to resist the effects of forces determined as follows:

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \qquad \text{(Eq. 16)}$$

where

F_i = the lateral force applied to level i

w_i = the portion of W at level i

w_{px} = the weight of the diaphragm and the elements tributary thereto at level x, including 25 percent of the floor live load in storage and warehouse occupancies.

The force F_{px} determined from equation 16 need not exceed $0.30ZIw_{px}$.

When the diaphragm is required to transfer lateral forces 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 stiffness in the vertical elements, these forces shall be added to those determined from equation 16.

However, in no case shall lateral force on the diaphragm be less than $0.14ZIw_{px}$.

4.4.4 Overturning. The increment of overturning moment at each story shall be distributed to the resisting elements in the same proportion as the distribution of the horizontal shears. For tall buildings analyzed according to 4.2, the overturning moment effect in any element may be multiplied by the factor k depending on the location of the element as follows:

(1) k = 1.0 for the top 10 stories

(2) k = 0.8 for the 20th story from the top and below

(3) k = a value between 1.0 and 0.8 determined by a straight line interpolation for stories between the 10th and 20th stories below the top.

Where other vertical members are provided which are capable of partially resisting the overturning moments, a redistribution may be made to these members if framing members of sufficient strength and stiffness to transmit the required loads are provided.

Where a vertical resisting element is discontinuous, provision shall be made to carry the overturning moment carried by the lowest story of that element to the foundation.

At the soil-foundation interface, the design overturning moment may be modified as follows:

(1) for buildings analyzed according to 4.2, 75 percent of the value determined in 4.2.5 may be used.

(2) for buildings analyzed according to 4.3, 90 percent of the value determined in 4.3.3 may be used.

4.4.5 Orthogonal Effect. For all buildings in zones 1 or 2, the design seismic load effects may be determined assuming separate application of the seismic loads on each of the main axes of the structure. For all buildings in zones 3 or 4, the seismic load effect shall be determined from the most critical direction of application of the seismic loads, which may be assumed to be satisfied if the following combination is taken: 100 percent of the force effect for one direction of motion plus 30 percent of the force effect for the perpendicular direction of motion. The combination requiring the maximum element strength shall be used. Exception: diaphragms, and components of the seismic force-resisting system utilized in only one of the two orthogonal directions need not be designed for the combined effects.

4.4.6 Vertical Motion Effect. When combining gravity and seismic load effects for buildings in zones 3 or 4, vertical seismic loads shall be accounted for by increasing or decreasing the dead load as given in this section:

$$\text{additive load combination} = 1.2\,L_D + L_L + L_S \pm L_E \qquad \text{(Eq. 17)}$$

$$\text{counteracting load combination} = 0.8\,L_D \pm L_E \qquad \text{(Eq. 18)}$$

where

L_D = dead load effect

L_L = live load effect

L_S = snow load effect

L_E = lateral seismic load effect

The combination requiring the larger resistance shall be used. For brittle elements, such as welded steel column splices, etc., or for horizontal prestressed components, the coefficient 0.8 in equation 18 shall be changed to 0.5. For horizontal cantilever components, the coefficient 0.8 in equation 18, shall be changed to -0.2.

4.4.7 Displacements. The displacement calculated from the application of the required lateral forces shall be multiplied by (3.0/K) to obtain the design displacement (and drift).

4.5 INELASTIC ANALYSIS. Any analytical procedure based on the principles of mechanics may be used subject to the approval of the designated authority. In no case shall the force effects for which the building is designed be less than that determined using 4.3.3.

4.6 ANALYSIS OF OTHER STRUCTURAL ELEMENTS

Parts or portions of structures, nonstructural components and their anchorage to the main structural system shall resist the following lateral seismic forces:

$$F_p = ZIC_pW_p \qquad \text{(Eq. 19)}$$

where

Z and I are the coefficients used for the building (see 4.2.1 and 1.4.2)

W_p is the weight of the part or portion

C_p is set forth in table 7.

Interconnection forces between two parts of the structure shall not be less than 0.133ZI times the weight of the smaller portion.

Table 7

Horizontal Force Factor C_p for Elements of Structures

Part or Portion of Buidings	Direction of Horizontal Forces	Value of C_p
Exterior bearing and nonbearing walls, interior bearing walls and partitions, interior non-bearing walls and partitions. Masonry or concrete fences over 6 feet high.	Normal to flat surface	0.3
Cantilever elements, chimneys or stacks	Any direction	0.8
Connections for prefabricated structural elements other than walls, with force applied at center of gravity of assembly.	Any direction	0.3

5. DESIGN AND CONSTRUCTION STANDARDS FOR STRUCTURAL MATERIALS

This chapter specifies the commonly accepted standards for materials, design, and construction that are presumed as a basis for this standard. The designated authority may require or approve the use of a different edition or standard than specified here. In overseas construction, where local materials of grades other than those herein are used, the working stresses, grades, and other requirements of this standard shall be modified as applicable in accordance with good engineering practice.

5.1 STEEL. The quality and testing of steel materials and the design and construction of steel components which resist seismic forces shall conform to the following references, except as modified by other provisions of this standard.

(1) Specification for the Design, Fabrication and Erection of Structural Steel for Buildings, American Institute of Steel Construction (AISC), 1978.

(2) Structural Welding Code (AWS D1.1-80), American Welding Society (AWS), 1980.

(3) Specification for Structural Joints Using ASTM A325 or A490 Bolts, AISC, 1976.

(4) Specification for the Design of Cold-formed Steel Structural Members, American Iron and Steel Institute (AISI), 1980 Edition.

(5) Standard Specification for Open Web Steel Joists, J- and H-Series, adopted by the Steel Joist Institute (SJI) and AISC on October 1, 1974.

(6) Standard Specification for Longspan Steel Joists LJ- and LH-Series and Deep Longspan Steel Joists, DLJ- and DLH-Series, adopted by SJI and AISC on October 1, 1974.

(7) Criteria for Structural Applications for Steel Cables for Buildings, AISI, 1973 Edition.

(8) Design Manual for Floor Decks and Roof Decks (SDI #23), Steel Deck Institute.

5.2 CONCRETE. The quality and testing of concrete and steel materials and the design and construction of reinforced concrete components which resist seismic forces shall conform to the following references except as modified by other provisions of this standard:

(1) Building Code Requirements for Reinforced Concrete, (ACI 318-77, excluding Appendix A), American Concrete Institute (ACI), 1977.

(2) Specifications for Structural Concrete for Buildings (ACI 301-72), ACI, revised 1975.

5.3 WOOD. The quality, design, and construction of members and their fastenings in wood systems which resist seismic forces shall conform to the following references except as modified by other provisions of this standard:

(1) National Design Specification for Stress Grade Lumber and Its Fastenings, National Forest Products Association, 1977, with 1980 Supplement.

(2) American Softwood Lumber Standard, Voluntary Product Standard, PS 20-70, U.S. Department of Commerce, 1970.

(3) Plywood Design Specification, American Plywood Association, 1977.

(4) Plywood Diaphragm Construction, American Plywood Association, 1977.

(5) Construction and Industrial Plywood, Voluntary Product Standard, PS 1-74, U.S. Department of Commerce, 1974.

(6) Standards for the Design of Structural Timber Framing, AITC 102-76, American Institute of Timber Construction, 1976.

(7) Standard Specifications for Structural Glued Laminated Timber of Douglas Fir, Western Larch, Southern Pine, and California Redwood, AITC 117-76, American Institute of Timber Construction, 1976.

(8) Structural Glued-Laminated Timber, Voluntary Product Standard, PS 56-73, U.S. Department of Commerce, 1973.

(9) Part III of the One- and Two-Family Dwelling Code, 1979 editions, published by International Conference of Building Officials, Building Officials and Code Administrators, and Southern Building Code Congress.

(10) Section 4713: "Shear-resisting Construction with Wood Frame," Uniform Building Code, International Conference of Building Officials, 1979.

5.4 MASONRY. The quality and testing of masonry and steel materials and the design and construction of masonry and reinforced masonry components that resist seismic forces shall conform to one or more of the following references, except as modified by other provisions of this standard:

(1) Chapter 24: Masonry, in the Uniform Building Code, International Conference of Buiding Officials, 1979.

(2) Section 4: Reinforced Masonry, in Seismic Design for Buildings, TM 5-809-10, Departments of the Army, Navy and Air Force, 1973.

(3) Building Code Requirements for Concrete Masonry Structures, ACI 531-79, American Concrete Institute, 1979.

(4) American Standard Building Code Requirements for Masonry, A41.1-1953, National Bureau of Standards Miscellaneous Publication, 1954.

(5) Building Code Requirements for Reinforced Masonry, A41.2-1960 (R 1970), American National Standards Institute.

(6) Specification for the Design and Construction of Load Bearing Concrete Masonry, National Concrete Masonry Association, 1970.

(7) Building Code Requirements for Engineered Brick Masonry, Brick Institute of America, 1969.

(8) Part III of the One- and Two-Family Dwelling Code, 1979 edition, published by International Conference of Building Officials, Building Officials and Code Administrators, and Southern Building Code Congress.

5.5 ALUMINUM. The quality, testing, design, and construction of aluminum members which resist seismic forces shall conform to the following reference, except as modified by other provisions of this standard:

Specifications for Aluminum Structures, 3rd Edition, The Aluminum Association, 1976.

5.6 GYPSUM. The quality, testing, design, and construction of gypsum components which resist seismic forces shall conform to the following references, except as modified by other provisions of this standard:

(1) Section 2407: Reinforced Gypsum Concrete, in Uniform Building Code, International Conference of Building Officials, 1979.

(2) Section 4711: Gypsum Wallboard, in Uniform Building Code, International Conference of Building Officials, 1979.

(3) Section 5-05: Gypsum Diaphragms, Cast-in-Place, in Seismic Design for Buildings, TM 5-809-10, Departments of the Army, Navy, and Air Force, 1973.

(4) Standard Specification for Gypsum Concrete, ASTM C317-76.

(5) Standard Specification for Lightweight Aggregate for Insulating Concrete, ASTM C332-77a.

(6) USA Standard Specification for Reinforced Gypsum Concrete, The Gypsum Association, 1968.

(7) Recommended Specifications for the Application and Finishing of Gypsum Board, The Gypsum Association, 1979.

(8) Using Gypsum Board for Walls and Ceilings, The Gypsum Association, 1977.

6. STRUCTURAL DESIGN DETAILS

Where a single building includes framing systems that have different values of the coefficient for structural system response, K, each component common to systems having different K values shall satisfy the more stringent detailing requirements.

Portions of the following documents are frequently cited in the provisions of this chapter by a reference number enclosed in square brackets:

[1] Uniform Building Code, International Conference of Building Officials, 1979.

[2] Recommended Lateral Force Requirements and Commentary, Structural Engineers Association of California, 1978.

[3] Seismic Design for Buildings, Departments of the Army, Navy and Air Force (TM 5-809-10), 1973.

[4] Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings, American Institute of Steel Construction, 1979.

[5] Building Code Requirements for Reinforced Concrete, American Concrete Institute, 1977 (ACI 318-77).

6.1 MOMENT-RESISTING FRAMES

6.1.1 Ductile Moment-Resisting Space Frames. Ductile moment-resisting space frames shall be structural steel complying with 6.2 or reinforced concrete complying with 6.3.

6.1.2 Concrete Frames. In zones 2, 3 and 4, all concrete space frames required by design to be part of the lateral force-resisting system and all concrete frames located in the perimeter line of vertical support shall be ductile moment-resisting space frames. EXCEPTION: Frames in the perimeter line of the vertical support of buildings designed with shear walls taking 100 percent of the design lateral forces need only conform with 3.2.2.

6.2 STEEL DUCTILE MOMENT-RESISTING SPACE FRAMES. The design and construction of moment-resistant joints for steel ductile moment-resisting space frames shall comply with all applicable requirements for "Type 1 Construction" as defined in reference [4], unless it can be shown that adequate ductility can be obtained by deformations of the connection materials and that the added drift is accounted for.

In addition, steel ductile moment-resisting space frames for all buildings in zones 3 and 4 and buildings with an importance factor I greater than 1.0 in zone 2 shall comply with the requirements of Section 2722: "Steel Ductile Moment-Resisting Space Frames" of reference [1], or Section 4 "Steel Ductile Moment-Resisting Space Frames" of reference [2], with the following clarification: the requirements for "plastic design sections" are the minimum thickness and lateral bracing requirements (sections 2.7 and 2.9) of Part II of reference [4].

6.3 REINFORCED CONCRETE DUCTILE MOMENT-RESISTING SPACE FRAMES. The design and construction of concrete ductile moment-resisting space frames for all buildings in zone 3 or 4 and buildings with an importance factor I greater than 1.0 in zone 2 shall comply with the requirements of one of the following:

(1) Section 2626: "Ductile Moment-Resisting Space Frames - Seismic Zones Nos. 3 and 4" of reference [1].

(2) Section 2: "Concrete Ductile Moment-Resisting Space Frames" of reference [2].

(3) Paragraph 7-04: "Concrete Ductile Moment-Resisting Space Frames" of reference [3].

Concrete ductile moment-resisting space frames in other buildings shall comply with either the previous requirements or the requirements of reference [5], excluding Appendix A, plus the following requirements of this section.

The terms "web reinforcement," "tie," and "spiral" and the symbol "d" in the following shall be used as defined in reference [5].

6.3.1 Flexural Members. Web reinforcement shall be required throughout the length of the member. It shall be designed according to Chapter 11 of reference [5], except that such web reinforcement shall be not less than 0.15 percent of the area computed as the product of the width of the web and the spacing of web reinforcement along the longitudinal axis of the member. The first stirrup shall be located 2 inches from the column face. The next six stirrups shall be spaced not over d/4.

Positive moment reinforcement at all supports of flexural members subject to reversal of moments shall be anchored by bond or mechanical anchors in or through the supporting member to develop the yield strength of the bar.

Lapped splices located in a region of tension or reversing stress shall be confined by at least two closed ties at each splice.

6.3.2 Columns. The spacing of ties at the ends of tied columns shall not exceed 4 inches for a distance equal to the maximum column dimension but not less than one-sixth of the clear height of the column from the face of the joint. The first tie shall be located 2 inches from the face of the joint. Joints of exterior and corner columns shall be confined by lateral reinforcement through the joint. Such lateral reinforcement shall consist of spirals or ties as required at the ends of columns.

6.4 BRACED FRAMES. In zones 3 and 4 and for buildings having an occupancy importance factor, I, greater than 1.0 located in zone 2, braced frames shall satisfy the following requirements.

6.4.1 Required Capacity. All members in braced frames shall be designed for 1.25 times the force determined in accordance with chapter 4, STRUCTURAL ANALYSIS PROCEDURES. Connections shall be designed to develop the full capacity of the members or shall be based on the above forces without the one-third increase usually permitted for stresses resulting from earthquake forces.

6.4.2 Steel Braced Frames. Braced frames shall be composed of axially loaded bracing members of ASTM A36, A440, A441, A500, A501, A572 (except Grades 60 and 65) or A588 structural steel. A500 steel shall not be welded.

6.4.3 Reinforced Concrete Braced Frames. Reinforced concrete members of braced frames subjected primarily to axial stresses shall have special transverse reinforcing as specified for axially loaded frame members in the requirements cited by reference in 6.3 throughout the full length of the member. Tension members additionally shall meet the requirements for compression members.

6.5 REINFORCED CONCRETE SHEAR WALLS. Reinforced concrete shear walls for all buildings in zones 3 or 4 and buildings with an importance factor I greater than 1.0 in zone 2 shall comply with the applicable requirements of one of the following:

(1) Section 2627: "Earthquake Resisting Concrete Shear Walls and Braced Frames" of reference [1].

(2) Section 3: "Concrete Shear Walls and Braced Frames" of reference [2].

6.6 REINFORCED MASONRY WALLS. Masonry walls required to be reinforced masonry by 6.9 or 7.5 shall comply with the minimum amount and maximum spacing of reinforcement specified in table 8.

Table 8

Minimum Reinforcement and Maximum Spacing for Reinforced Masonry Walls

Type of Wall	Total Area[1] of Reinforcement as a Percent of Gross Area of Wall (nominal dimensions)			Maximum Spacing of Bars (inches)					
				Vertical Bars			Horizontal Bars		
	Seismic Zones			Seismic Zone			Seismic Zone		
	4&3	2	1	4&3	2	1	4&3	2	1
Structural (i.e., bearing or shear)	0.20	0.20	0.15	24	36	60	48	60	72
Nonstructural	0.15	0.15	0.15	48	60	72	84	84	96

(1) The total minimum reinforcement is the sum of the vertical and horizontal reinforcement; not less than 1/3 of the prescribed total minimum reinforcement shall be used in each direction.

Splices may be made only at such points and in such manner that the structural strength of the member will not be reduced. Lapped splices shall provide sufficient lap to transfer the working stress of the reinforcement by bond and shear, but in no case shall the lap be less than 30 bar diameters. Welded or mechanical connections shall develop the strength of the reinforcement.

6.7 DIAPHRAGMS

6.7.1 Ties Between Chords. Diaphragms providing lateral support to concrete or masonry walls by means of anchor bolts or similar connections shall have ties to distribute the anchorage forces into the diaphragm.

6.7.2 Anchorages to Wood Diaphragms. In zones 2, 3 or 4 where wood diaphragms are used to laterally support concrete or masonry walls, the anchorage shall not be accomplished by use of toenails or nails subjected to withdrawal; nor shall wood framing be used in cross-grain bending or cross-grain tension.

6.8 OPENINGS IN SHEAR WALLS AND DIAPHRAGMS.

Where steps in the edges or openings occur in shear walls or diaphragms or other plate-like elements, chords shall be provided at the edges of the discontinuity to resist the local stresses created by the presence of the discontinuity. These chords shall extend into the body of the wall or diaphragm a distance sufficient to develop and distribute the stress of the chord member.

6.9 CONCRETE AND MASONRY ELEMENTS

6.9.1 Reinforcement. All elements within structures located in zones 2, 3, or 4 which are of masonry or concrete shall be reinforced so as to qualify as reinforced masonry or concrete under the provisions of 6.6 for masonry and reference [5] for concrete. Principal reinforcement in masonry shall be spaced 2 feet maximum on center in buildings using a moment-resisting space frame.

6.9.2 Anchorage of Walls. Concrete or masonry walls shall be anchored to all floors and roofs which provide lateral support for the wall. Such anchorage shall provide a positive direct connection capable of resisting the horizontal forces specified in this standard or a minimum force of 200 pounds per linear foot of wall, whichever is greater. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 feet. Required anchors in masonry walls of hollow units or cavity walls shall be embedded in a reinforced grouted structural element of the wall.

6.10 FOUNDATIONS

6.10.1 Ties Between Foundation Units. Unless it can be demonstrated that equivalent restraint can be provided by other approved methods, ties at approximately right angles shall be provided between foundation units as follows:

(1) in zones 2, 3, or 4, between individual pile caps and caissons in all buildings

(2) in zones 3 or 4, between isolated spread footings in buildings over three stories tall or in buildings constructed over crawl spaces.

The ties shall resist the induced lateral seismic forces, but not less than a minimum horizontal force of 0.10ZI times the vertical load on the pile, cap, caisson, or footing.

6.10.2 Pile Cap Connections. In zones 2, 3, or 4, all piles shall be connected to the pile cap to resist an uplift force of not less than 0.15ZI times the vertical load on the pile.

6.10.3 Concrete Piles. In zones 2, 3, or 4, all concrete piles shall be provided with longitudinal reinforcement sufficient to resist the uplift force specified in 6.10.2 throughout the entire length of the pile, except that the reinforcement only need be provided in the upper one-third of the pile in zone 2.

Furthermore, in zones 2, 3, or 4, all concrete piles shall be provided with transverse ties spaced no further apart than 4 inches over the top 4 feet of the pile.

Properly bonded metal casing, such as steel pipe, may be used to satisfy these reinforcement requirements.

7. NONSTRUCTURAL DESIGN REQUIREMENTS

The requirements of this section apply to all buildings in zones 3 or 4 and to buildings with an importance factor I greater than 1.0 in zone 2.

7.1 ANCHORAGE FOR INERTIAL FORCES.

Nonstructural components and their anchorage to the main structural system shall resist the following lateral seismic forces:

$$F_p = ZIC_pW_p \qquad \text{(Eq. 19)}$$

where

Z is specified in 4.2.1

I is specified in 1.4.2 EXCEPT:

1. For connectors of precast or prefabrication panels, the value of I shall be as given in 7.2.
2. For the anchorage of machinery and equipment required for life safety systems, the value of I shall be 1.5.

C_p is given in table 9.

W_p is the weight of the component, EXCEPT:

1. For storage racks, W_p shall be the weight of the rack plus contents.
2. For ceilings, W_p shall include all light fixtures and other equipment which is laterally supported by the ceiling, and shall be taken as not less than 4 pounds per square foot.

Table 9

Horizontal Force Factors C_p for Nonstructural Elements of Buildings

Component	Direction of Horizontal Forces	Value of C_p[1]
Exterior and interior walls and partitions Masonry or concrete fences over 6 feet high	Normal to flat surface	0.3
Cantilever elements:		
a. Parapets	Normal to flat surface	0.8
b. Chimneys or stacks	Any direction	0.8
Exterior and interior ornamentations and appendages	Any direction	0.8
When connected to, part of, or housed within a building: a. Penthouses, anchorage and supports for chimneys and stacks and tanks, including contents b. Storage racks with upper storage level at more than 8 feet in height, plus contents c. All equipment or machinery d. Large ducts, large pipes, and critical or hazardous pipes	Any direction	0.3[2,3]
Suspended ceiling framing systems	Any direction	0.3

1. C_p for elements laterally self-supported only at the ground level may be two-thirds of value shown.

2. For flexible and flexibly mounted equipment and machinery, the appropriate values of C_p shall be determined with consideration given to both the dynamic proportion of the equipment and machinery and to the building or structure in which it is placed but shall be not less than the listed values. The design of the equipment and machinery and their anchorage is an integral part of the design and specification of such equipment and machinery.

3. The value of C_p for racks over two storage support levels in height shall be 0.24 for the levels below the top two levels.

The distribution of these forces shall be according to the gravity loads pertaining thereto.

In lieu of this section, steel storage racks may be designed in accordance with ANSI Standard MH 16.1-1974, or where a number of storage rack units are interconnected so that there are a minimum of four vertical elements in each direction on each column line designed to resist horizontal forces, the racks may be designed as a structure in accordance with 4.2.1 with the design coefficients CS = 0.2 and W equal to the total dead load plus 50 percent of the rack-rated capacity.

7.2 DISTORTION COMPATIBILITY FOR EXTERIOR PANELS. Precast or prefabricated nonbearing, nonshear wall panels or similar elements which are attached to or enclose the exterior shall be designed to resist forces determined from Eq. 19 and shall accommodate movements of the structure resulting from lateral forces. The concrete panels or other similar elements shall be supported by means of cast-in-place concrete or mechanical connections and fasteners in accordance with the following provisions:

Connections and panel joints shall allow for a relative movement between stories of not less than the drift calculated in 4.4.7 or 1/2 inch, whichever is greater. Connections to permit movement in the plane of the panel for story drift shall be properly designed sliding connections using slotted or oversized holes or may be connections which permit movement by bending of steel or other connections providing equivalent sliding and ductility capacity.

Bodies of connectors shall have sufficient ductility and rotation capacity so as to preclude fracture of the concrete or brittle failures at or near welds.

The body of the connector shall be designed for one and one-third times the force determined by Eq. 19. Fasteners attaching the connector to the panel or the structure such as bolts, inserts, welds, dowels, etc., shall be designed to ensure ductile behavior of the connector or shall be designed for four times the load determined from Eq. 19.

Fasteners embedded in concrete shall be attached to or hooked around reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel.

The value of the coefficient I shall be 1.0 for the entire connector assembly in Eq. 19.

7.3 PROTECTION AGAINST SECONDARY HAZARDS. Hazardous contents and services shall not present undue hazard to life in the event of seismic ground shaking. As a minimum, for Essential Facilities and High Risk occupancies (I = 1.5 and 1.25, respectively) in zones 3 and 4, the utility and service interface of all gas, high-temperature energy, and electrical supply shall be provided with shutoff devices or special protection.

7.4 FUNCTIONALITY OF ESSENTIAL ELEMENTS. The design and detailing of equipment which must remain in place and be functional following a major earthquake shall be based upon the requirements of 7.1. In addition, their design and detailing shall consider effects induced by the structural drift calculated in 4.4.7. Special consideration shall also be given to relative movement at separation joints.

7.5 REINFORCEMENT OF CONCRETE AND MASONRY. All nonstructural elements within structures located in zones 3 or 4 which are of masonry or concrete shall be reinforced so as to qualify as reinforced masonry or concrete as defined in chapter 5, DESIGN AND CONSTRUCTION STANDARDS FOR STRUCTURAL MATERIALS and chapter 6, STRUCTURAL DESIGN DETAILS. Principal reinforcement in masonry shall be spaced 2 feet maximum on center in buildings using a moment-resisting space frame.

8. CONSTRUCTION QUALITY CONTROL

In accordance with agency quality assurance procedures, all buildings in zones 3 or 4 and buildings with an importance factor I greatter than 1.0 in zone 2 shall be subject to inspection by the designated authority, and certain types of construction shall have special inspection to assure the quality and performance of the seismic resisting systems, as specified in this section.

8.1 SPECIAL INSPECTOR. A special inspector shall be employed by the designated authority during construction to observe the work specified in 8.2 to be certain it conforms to the design drawings and specifications. The designated authority may waive the requirement for a special inspector if he finds that the construction is of minor nature.

8.1.1 Qualifications. The special inspector shall be a qualified person who shall demonstrate his competence, to the satisfaction of the designated authority, for inspection of the particular type of construction or operation requiring special inspection.

8.1.2 Inspection Reports. The special inspector shall furnish inspection reports to the designated authority, the engineer or architect of record, and other designated persons. All discrepancies shall be brought to the immediate attention of the contractor for correction, then, if uncorrected, to the engineer or architect of record and to the designated authority.

8.1.3 Final Report. The special inspector shall submit a final signed report stating whether the work requiring special inspection was, to the best of his knowledge, in conformance with the approved plans and specifications and the applicable workmanship provisions of these codes.

8.2 REQUIRED SPECIAL INSPECTION. The following objects and operations shall be subject to continuous inspection by a special inspector. Some inspections may be made on a periodic basis and satisfy the requirements of continuous inspection, provided this periodic scheduled inspection is performed as outlined in the project plans and specifications and approved by the designated authority. Special inspections required by this standard shall not be required where the work is done on the premises of a fabricator approved by the designated authority to perform such work without special inspection.

8.2.1 Piling, Drilled Piers and Caissons. During driving and testing of piles and construction of cast-in-place drilled piles or caissons. See 8.2.3 and 8.2.4 for concrete and reinforcing steel inspection.

8.2.2 Excavation and Filling. During earthwork excavations, grading, and filling operations inspection to satisfy the requirements of the plans and specifications.

8.2.3 Concrete. During the taking of test specimens and placing of all reinforced concrete and pneumatically placed concrete. EXCEPTIONS:

(1) For foundation concrete when the structural design is based on f'_c no greater than 2000 psi.

(2) Nonstructural slabs on grade, including prestressed slabs on grade when effective prestress in concrete is less than 150 pounds per square inch, unless the slab is used as a tie to satisfy 6.10.1.

(3) Site work concrete full-supported on earth and concrete where no special hazard exists.

8.2.4 Reinforcing Steel and Prestressing Steel. During all stressing and grouting of prestressed concrete, and during placing of reinforcing steel, placing of tendons and prestressing steel for all concrete required to have special inspection by 8.2.3. EXCEPTION: The special inspector need not be present during entire reinforcing steel and prestressing steel-placing operations, provided he has inspected for conformance with the approved plans prior to the closing of forms or the delivery of concrete to the job site.

8.2.5 Ductile Moment-Resisting Concrete Frame. Continuous inspection by a specially qualified inspector under the supervision of the person responsible for the structural design during the placement of reinforcement and concrete.

8.2.6 Welding

(1) All structural welding, including welding of reinforcing steel. EXCEPTIONS: When welding is done in an approved fabricator's shop; or when approved by the designated authority, single-pass fillet welds when stressed to less than 50 percent of the allowable stresses and floor and roof deck welding and welded studs when used for structural diaphragm or composite systems may have periodic inspections. For periodic inspection, the inspector shall check qualifications of welders at start of work and then make final inspection of all welds for compliance prior to completion of welding.

(2) Ductile moment-resisting steel frames shall receive the following nondestructive testing:

Welded connections between the primary members of ductile moment-resisting space frames shall be tested by nondestructive methods for compliance with the AWS Structural Welding Code (D1.1-80) and job specifications. A program for this testing shall be established by the person responsible for structural design and as shown on plans and specifications. As a minimum, this program shall include the following:

(a) All complete penetration groove welds contained in joints and splices shall be tested. EXCEPTION: When approved, the non-destructive testing rate for an individual welder or welding operator may be reduced to 25 percent, provided the reject rate is demonstrated to be five percent or less of the welds tested for the welder or welding operator. A sampling of at least 40 completed welds for a job shall be made for such reduction evaluation. Reject rate is defined as the number of welds completed. For evaluating the reject rate of continuous welds over 3 feet in length where the effective size is 1 inch or less, each 12-inch increment or fraction thereof shall be considered as one weld. For evaluating the reject rate on continuous welds over 3 feet in length where the effective size is greater than 1 inch, each 6 inches of length or fraction thereof shall be considered one weld.

When approved by the designated authority and outlined in the project plans and specifications, this nondestructive testing may be performed in the shop of an approved fabricator utilizing qualified test techniques in the employment of the fabricator.

(b) All partial penetration groove welds when used in column splices shall be tested when required by the plans and specifications.

(c) Base metal thicker than 1-1/2 inches, when subjected to through-thickness weld shrinkage strains, shall be inspected for discontinuities directly behind such welds after joint completion. Any material discontinuities shall be accepted or rejected on the basis of the defect rating in accordance with the criteria of the AWS Structural Welding Code (D1.1-80).

8.2.7 High-Strength Bolting. During all bolt installations and tightening operations. EXCEPTIONS:

(1) The special inspector need not be present during the entire installation and tightening operation, provided he:

(a) inspects the surfaces and bolt type for conformance to plans and specifications prior to start of bolting, and

(b) will verify the minimum specified bolt tension for 10 percent of the bolts for each "type" of connection, for a representative number of total connections established by the plans and specifications.

(2) In bearing-type connections when threads are not required by design to be excluded from the shear plane, inspection prior to or during installation will not be required.

8.2.8 Structural Masonry. During preparation of masonry wall prisms, sampling and placing of all masonry units, placement of reinforcement, inspection of grout space, immediately prior to closing of cleanouts, and during all grouting operations. Where the f'_m is less than 2600 psi and special inspection stresses are used, test specimens may consist of either one prism test for each 5000 square feet of wall area or a series of tests based on both grout and mortar for the first 3 consecutive days and each third day thereafter. EXCEPTION: Special inspection will not be required for structures designed in accordance with the values in appropriate tables for noncontinuous inspection.

8.2.9 Reinforced Gypsum Concrete Used as a Diaphragm. When cast-in-place Class "B" (1000 psi minimum compressive strength) gypsum concrete is being mixed and placed.

8.2.10 Insulating Concrete Used as a Diaphragm. During the application of insulating concrete. EXCEPTION: The special inspection may be limited to an initial inspection to check the deck surface and placement of reinforcing. The special inspector shall supervise the preparation of compression test specimens during this initial inspection.

8.2.11 Special Cases. Any other work which, in the opinion of the designated authority requires special inspection.

COMMENTARY

This standard contains provisions for the reduction of earthquake hazards in Federal buildings. One purpose is to provide a uniform standard for use by all Federal agencies for the planning, design, and construction of buildings, both within and outside the United States. As with the voluntary standards and codes upon which this standard is based, the requirements

> "are intended to provide criteria to fulfill life safety concepts. It is emphasized that the recommended design levels are not directly comparable to recorded or estimated peak ground accelerations from earthquakes. They are however, related to the effective peak accelerations to be expected in seismic events. More specifically with regard to earthquakes, structures designed in conformance with the provisions and principles set forth therein should, in general, be able to:
>
> 1. Resist minor earthquakes without damage;
> 2. Resist moderate earthquakes without structural damage, but with some nonstructural damage;
> 3. Resist major earthquakes, of the intensity of severity of the strongest experienced in California, without collapse, but with some structural as well as nonstructural damage.
>
> In most structures it is expected that structural damage, even in a major earthquake, could be limited to repairable damage. This, however, depends upon a number of factors, including the type of construction selected for the structure."[1]

The standard has been prepared for the purposes of Federal agencies as an adaptation of existing voluntary standards, model building codes, Federal regulations, and research reports. The standard is most closely related to the Uniform Building Code, 1979 edition (published by the International Conference of Building Officials and subsequently referred to as the UBC). The seismic provisions of the UBC are based on the Recommended Lateral Force Requirements and Commentary, 1975 edition (published by the Structural Engineers Association of California and subsequently referred to as the SEAOC

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[1] From the SEAOC Commentary

requirements). Substantial use is also made of the Tentative Provisions for the Development of Seismic Regulations for Buildings (prepared by the Applied Technology Council and published by the National Bureau of Standards and subsequently referred to as the ATC provisions). The seismic regulations of the Veterans Administration, Army, Navy, Air Force, General Services Administration, and Department of Housing and Urban Development were all considered in the development of this standard.

One of the objectives of this commentary is to explain the differences between the provisions of this standard and those of its sources. Because the seismic provisions of the UBC are the most widely used and because they served as the starting point for this standard, particular care is taken to relate these provisions to those of the UBC. Although there are many differences with the UBC, owing to different contexts, policies, and styles, the basic technical approach to the analysis of seismic loads and the design of buildings to resist such loads is the same.

This commentary also addresses specific issues to assist the user of the standard, but it is not a complete commentary on seismic-resistant design. Both the SEAOC and ATC provisions are accompanied by extensive commentaries that are recommended reading for users of this standard. In addition, the Army, Navy, and Air Force manual Seismic Design for Buildings (referred to as the Tri-Services Manual) contains guidelines for procedures and details that facilitate implementation of provisions for seismic resistance of buildings.

C1.1 SCOPE. This standard is primarily intended for new buildings. Structures that are not associated with buildings are outside the scope because their particular functional requirements and the nature of hazard they present have not been considered in developing the requirements of this standard and because the idealization of seismic loadings presented in this standard may not be an appropriate model for predicting their physical response to seismic ground shaking. Another exception is that buildings and structures associated with nuclear power plants are subjected to the more rigorous standards of the Nuclear Regulatory Commission.

The coverage of existing buildings in this standard is brief; a related document (also to be issued by the Interagency Committee on Seismic Safety in Construction) will provide more specific criteria for those occasions when the requirements of this standard indicate the need for their application. The application of this standard to Federally leased buildings and to Federal grant and regulatory programs is covered in another document, which will be issued by the Interagency Committee on Seismic Safety in Construction.

The reference to varying degrees of safety is in part a recognition that different levels of reliability are desired for buildings that present different risks. Just as in the UBC, this standard accomplishes this purpose by means of a numerical coefficient for occupancy hazard, which is specified in 1.4.2. The statement is also a recognition that many provisions are based on crude approximations and may result in different levels of safety for different situations.

This standard is limited to consideration of the effects of seismic ground shaking. Other seismic effects, such as tsunami, and other environmental loads, such as wind, are not within the scope. Existing codes and standards commonly contain clauses pertaining to wind forces in many provisions, such as stiffness and overturning resistance. No such clause is in this standard, because criteria for resistance to the two forces are too different. Because the response to seismic ground shaking will ordinarily require inelastic straining of the seismic force-resisting system, the many ductility and detailed requirements in this standard must be satisfied even for buildings in which wind force exceeds the nominal design seismic force. For buildings in which the nominal design seismic force exceeds the nominal design wind force, it is likely that wind requirements may yet control the design of various components because the local pressure and suction coefficients may create higher force resultants or because the drift criterion is different for wind than for seismic loadings, and so on. Thus, no comparisons of the two loadings are stated or implied in the provisions of the standard.

C1.2 DEFINITIONS. The bulk of the definitions are taken from, or based upon, the following sections of the UBC: Chapter 4 for general terms, section 2302 for structural terms, and 2312(b) for seismic terms. The definition for height is original, but is consistent with the use of the terms in the UBC. Designated authority is original, and in the context of the UBC it can be thought to replace the term building official. The definition for diaphragm is taken from the ATC. In all the definitions an emphasis has been placed on lateral seismic forces when compared to the definitions in the UBC. The definitions of the classes of building systems are contained in 3.1.2, and the discussion of those definitions is in that section of commentary. The definitions for space frame, moment-resisting space frame, and ductile moment-resisting space frame are reworded in an attempt to improve their clarity.

C1.3 NOTATION. Each of the symbols used in section 2312 of the UBC are used for the same meaning in this standard. In addition, several symbols for modal analysis and for load combination equations appear in this standard that are not found in the UBC. The verbal expression defining many of the symbols is changed from the UBC in an attempt to make the collected notation more useful.

C1.4 HAZARD CLASSIFICATIONS. Buildings are classified on two scales in this standard, much as they are in the UBC. One scale, the zone, relates to the likelihood of seismic activity, and the other scale, the occupancy importance factor, relates to the consequences of structural failure due to an earthquake. These classifications are used throughout the standard to determine the applicability of specific requirements. Table C1 summarizes, very generally, the use of these hazard classifications for this purpose. It should be noted that there are several differences with the UBC.

C1.4.1 Seismic Ground Shaking Hazard. The map in this draft has been derived from the "Effective Peak Velocity-Related Acceleration" map contained in the ATC Commentary. It has the following characteristics necessary for consistent use with the basic provisions of the standard:

(1) It is a zone map, not a contour map, with zones, 0, 1, 2, 3, and 4.

(2) The boundaries are smooth lines that do not necessarily accomplish microzonation.

Table C1.

Applicability of Requirements

Zone	Occupancy Importance Factor 1	1.25	1.5
0	(a)	(a)	(a)
1	(b)	(b)	(b)
2	(b)	(c)	(c)
3	(c)	(c)	(c)+(d)
4	(e)	(e)	(e)+(d)

KEY:

(a) no requirements apply

(b) basic structural resistance required

(c) provisions consistent with current practice in California required, including upgraded ductility, nonstructural component resistance, and improved quality control

(d) special site evaluations required for some buildings

(e) modal analysis required for some buildings

(3) The relative design acceleration for the five zones 4 through 0 is in the proportion 1 : 3/4 : 3/8 : 3/16 : 0.

There are other maps with these characteristics. The Subcommittee on Standards for Buildings preferred this particular map over the current UBC map because it was developed on a basis of consistent risk across all zones and over the ATC "Effective Peak Acceleration" map because it accounts for the effect of large, distant earthquakes on tall buildings. The Subcommittee on Evaluation of Site Hazards also tentatively agreed to recommend this map.

It is expected that improvements will be made in the map as knowledge is accumulated in the extensive research programs ongoing in seismic hazard assessment. It should be expected that individual site evaluations will vary with regard to this map, because it is not possible to microzone with such a map. These variations may be particularly significant in the zones of highest seismicity. Various catalogs are available that correlate the seismic hazards in cities of interest around the world with the seismic hazard zones used for design in the United States.

Some buildings will be located on sites for which a comprehensive site evaluation will have been performed, and thus a site-specific value for design ground acceleration will occasionally be available. It is not possible to microzone on the map included in this standard, and thus, in many instances, such site specific accelerations may not be equivalent to the ground acceleration implied by the map. The relation to obtain zone from a value of site-specific ground acceleration is provided for such buildings. Such a relation is necessary for the proper application of various requirements that depend on the zone even though the seismic forces on such buildings will normally be calculated directly from the site-specific ground acceleration. The relation for the conversion is based on the assumption that the design ground motion would have, roughly, a 0.002 chance of being exceeded in one year. For approved site evaluations based on a different probability of earthquake occurance, the values in the table should be adjusted accordingly. The assumption corresponds to the basis for the map in this standard.

C1.4.2 Occupancy Hazard. The categories of occupancies in this section are the same as in section 2312(k) of the UBC with these exceptions: power stations and other emergency utilities are added to the essential category, and buildings in which the occupant's mobility is restricted or impaired or in which the contents are hazardous are added to the high risk category.

C1.5 ALTERNATIVE PROVISIONS. This provision is very general. Similar provisions are found in sections 2312(a) and (i) of the UBC.

C2. GENERAL REQUIREMENTS

Because this standard is not a chapter of a larger document dealing with all aspects of buildings, as are the seismic provisions in the UBC, and because this standard incorporates several provisions not found in the UBC, the organization of this standard is quite different from that of the seismic provisions of the UBC. This chapter sets the framework for the use of all the remaining chapters of this standard.

C2.1 GENERAL PERFORMANCE REQUIREMENT. This very general requirement is essentially the same as the first sentence of the UBC seismic provisions. The exemption for zone 0 merely gives formal recognition to present practice, although designers would be well advised to satisfy the intent of 3.1.1 even in zone 0, particularly for important structures.

C2.2 SITE PLANNING FOR NEW BUILDINGS

C2.2.1 Site Evaluation Study. This section is quite similar to the present practice of the Army, Navy, Air Force, and the Veterans Administration. The requirement is not arbitrarily applied to all essential facilities in high risk zones for reason of economics. The cost of the site evaluation could exceed the cost of the building for very small fire stations, which might not be the most efficient way of providing the additional reliability desired. Nor should the requirement for all hospitals be arbitrarily extended to all medical buildings; clinics and other small medical facilities may be provided with additional safety without resorting to full site evaluations. Thus, the designated authority is called upon to judge each case on its own merits. Procedures for carrying out site evaluation studies and guidelines for the approval of site evaluation studies are included in the manual of Standard Practice for the Evaluation of Site Hazards published by the Interagency Committee on Seismic Safety in Construction.

C2.2.2 Site Limitations. Note that the requirement for siting essential facilities to avoid surface rupture due to fault displacement does not apply to those essential facilities for which no site evaluation study is performed. Various engineering solutions for liquefaction, landslide, and subsidence problems exist, but no criteria are provided in this standard. The Interagency Committee manual on site hazard evaluations previously referenced includes some information on this subject.

C2.3 DESIGN OF NEW BUILDINGS. The bulk of this standard deals with design. This section merely establishes the applicability of the design requirements and requires a standard amount of documentation to confirm compliance with the requirements.

C2.4 CONSTRUCTION. The importance of quality assurance cannot be overstated. Federal agencies have general quality assurance programs; this provision supplements those agency provisions in the higher seismic zones for issues of special concern in seismic safety.

C2.5 EXISTING BUILDINGS. This standard does not contain much in the way of detailed criteria for existing buildings. Another document published by the Interagency Committee on Seismic Safety in Construction includes much more detail on the subject [in preparation by Subcommittee 3, Existing Buidings]. In particular, that document contains the methodology and mitigation techniques for corrective actions and it defines the appropriate level of strengthening called for in 2.5.2.

The second paragraph of 2.5.2 is taken from 2312(j)2A of the UBC. Note that overstrengthening one portion of a building could cause higher loads in other portions of the building and thus should be avoided.

C3. STRUCTURAL DESIGN CRITERIA

Chapter 3 includes the governing criteria for the structural portions of a building. Reference is made to chapter 4 for the determination of the force effects of seismic ground shaking, to chapter 5 for the basic standards for proportioning structural components, and to chapter 6 for special details important to assure the assumed behavior of the seismic force-resisting system in an earthquake.

C3.1 SEISMIC FORCE-RESISTING SYSTEMS. The ground motions observed in past large earthquakes are greater than those corresponding to the forces prescribed in this standard (or in any other current standards for ordinary structures). In spite of discrepancy between real and implied ground motions, many buildings designed according to the UBC and similar standards have performed well in past earthquakes. There are several factors that contribute to this successful performance, but ductility, the ability of a structure to be strained beyond its elastic limit, is probably the most important factor. Large earthquakes will ordinarily subject some portions of a building to repeated cycles of inelastic strain. Thus, it is important for a designer to define a seismic force-resisting system. The seismic force-resisting system must be designed and constructed with special attention so that it will maintain its resistance while undergoing the repeated cycles of inelastic straining caused by strong ground shaking.

The seismic force-resisting system will frequently be a subset of the overall structural system. For example, in a building with many frame bents in each direction, a few in each direction might be designated as seismic force-resisting and designed as ductile moment-resisting space frames or as braced frames. Although it would not be a common occurrence, it is possible for a designer to use a two-level approach in this. In such an approach, components more rigid yet less ductile than the designated seismic force-resisting system are considered effective in moderate earthquakes.

Such components may not be used to satisfy the strength criteria of 3.1.3, but may be used to satisfy the stiffness criteria of 3.1.4.

C3.1.1 Integrity. This requirement, taken from the ATC provisions, is very elementary. Movement of the ground beneath a building generates inertial forces due to each mass within the building, because the building responds as an integral unit in the motion. This design standard simulates the effect of these inertial forces by specifying equivalent seismic forces to be applied at each story. This requirement reminds the designer that the simulation is complete only when the specified seismic forces are carried back into the ground, which is the "final point of resistance." Conspicuous failures of building components in past earthquakes can be attributed to the lack of a complete and continuous path of structural resistance for seismic loads.

C3.1.2 System Response Classification. Seismic force-resisting systems possess different degrees of ductility, damping, redundancy, etc. These important characteristics affect the level of inertial force generated by a given ground motion and level of safety for a given level of force. Therefore, different seismic force coefficients and detailing rules are specified for various types of seismic force-resisting systems. The four classes defined in this section are taken from the UBC. The exact description of the systems is slightly different than in the UBC, but the changes are only an attempt to clarify the classification. The SEAOC Commentary provided the basis for discerning the intent of the original classification.

Several features of the classification deserve comment. The first decision in classing a building concerns bearing walls. Unfortunately, this decision often requires experienced judgment. Buildings in which "a significant fraction of the gravity load is supported on bearing walls" must be classed as a "Box System" in each principal direction. The presence of minor load bearing walls (for example, around a stairwell within one bay of the frame) does not mean that a building must be categorized as a "Box System" if the overall response of the building is not significantly influenced by the walls. The seismic force-resisting system of "Box Systems," will normally include the bearing walls functioning as shear walls plus any other designated components. Buildings that are not classed as "Box Systems" may be classed separately about their two principal axes.

The second important decision regards the use of ductile moment-resisting space frames in the designated seismic force-resisting system. Because rigid elements may exist in buildings without being a part of the designated seismic force-resisting system, it is possible for a building with both ductile moment-resisting space frames and shear walls (or braced frames) to be in any of the three remaining classes, as follows: 1) where the designated seismic force-resisting system includes only the ductile moment-resisting space frame, $K = 0.67$ is correct; 2) where the designated seismic force-resisting system includes both types of components and where the relative strengths of the two types of components satisfy the requirements of 3.1.3, $K = 0.80$ is correct: 3) otherwise, $K = 1.00$ is correct. The classification of such buildings becomes quite complex when the two level approach mentioned in the commentary on seismic force-resisting systems is adopted.

A great variety of seismic force-resisting systems is included in the "Building Frame System" class, such as shear walls, braced frames, arches, moment-resisting frames that do not meet the special ductility requirements of chapter 6, etc.

3.1.6 places restrictions on the class of seismic-force resisting systems allowed for tall buildings in zones 2, 3, and 4.

C3.1.3 Strength. This provision is consistent with the UBC. The intent is to continue present practice unchanged, which generally means increasing the allowable stress by 1/3 when considering the effects of seismic forces. Increases in allowable stresses and factors for the combinations of loads are specified in the material design standards referenced in chapter 5.

C3.1.4 Stiffness. Although the number looks quite different, these requirements are essentially the same as section 2312(h) of the UBC. Drift limits are imposed on the basis of experience and judgment. The principal reasons are to ensure that the stiffness is large enough to prevent stability failures and to control life hazards resulting from the failure of brittle nonstructural elements such as windows. In both cases the protection provided by the specified drift limit is only approximate. An added, but not primary, reason for the drift limit is to reduce damage and repair costs following moderate earthquakes. The designer should recognize that the actual deflections and drifts will be larger than those calculated on an elastic basis, due to the inelastic straining of the seismic force-resisting system. The SEAOC Commentary states that the real deflections will exceed the calculated elastic deflections by a factor of 3/K. Thus, the UBC limit of 0.005 times the story height is simply a nominal value applied to the elastic deflections. The SEAOC Commentary further advises that the anticipated real deflections rather than the elastic deflections be used in determining the distance necessary for structural separation joints. In order to avoid confusion and reduce the number of computations, all deflections and drifts are multiplied by 3.0/K (in 4.4.7) and the drift limit is changed from 0.005 to 0.015.

C3.1.5 Overturning Stability. This requirement is the same as that in section 2312(f) of the UBC, except that wind is not mentioned, for the reasons stated in C1.1. Overturning stability has rarely been a problem in real buildings subject to earthquakes. The calculated overturning moment is likely to be conservative because these provisions do not account for the rocking of a building on its foundation, the lengthening of the period of vibration due to inelastic effects below the base and in the soil, the contributions of the higher modes of vibration (except in 4.4.4 for tall buildings), and other similar factors which tend to reduce the real overturning in the building. Therefore, no margin of safety is called for between the overturning and resisting moments.

C3.1.6 Height. The requirement is essentially the same as section 2312(j)1B of the UBC. Its impact is that buildings over 160 feet high in the specified zones will have structural systems that qualify for a K value of 0.67 or 0.80.

C3.2 OTHER STRUCTURAL ELEMENTS. Seismic ground shaking affects the entire structural system, not just the designated seismic force-resisting system. These effects are primarily of two types, the inertial forces due to the mass of the structural element and the distortions the structural element experiences as the seismic force-resisting systems responds to the ground shaking.

C3.2.1 Strength and Anchorage. This requirement is based on section 2312(g) of the UBC. It applies in zone 1, just as the basic requirements for the seismic force-resisting system in 3.1. The increase in allowable stresses by 1/3 normally permitted in reference standards for seismic forces is applicable when satisfying this requirement.

C3.2.2 Compatibility. This requirement is taken from section 2312(j)1D of the UBC. The distortion specified in 4.4.7 (which includes the amplification by the factor 3/K) is intended to include both the ductility of the seismic force-resisting system and the P-delta effect, as indicated in the SEOAC Commentary.

C3.3 FOUNDATIONS.

C3.3.1 Soil and Foundation Capacity. This requirement is very performance oriented, by necessity. It is not possible to develop specific provisions at this time because soil and foundation conditions for buildings exhibit such a wide variety. The wording used is quite similar to the ATC provisions. The intent is to cause the geotechnical engineer and the foundation engineer to be cognizant of the differing effects of ground shaking on the properties of various soil types when establishing the allowable bearing pressures and other design criteria for load combinations involving earthquake. The performance objectives are to avoid bearing capacity failures and to avoid settlements so severe as to cause failure of the structural system of the building. The requirement should not be interpreted to imply that soil-structure interaction analyses are necessary; such analyses may be useful for special types of structures with specific soil and foundation conditions, but that is an issue to be decided on a case by case basis.

C3.3.2 Structural Ductility. As this provision indirectly indicates, the base and the foundation of a structure need not be the same entity. For a building with one or more levels below grade in which the levels below grade are considerably more stiff than the levels above grade and the soil around the basement levels is not exceptionally soft, the base can be considered to be at grade, because the most accurate simple model is to consider that the rigid substructure moves with the surrounding ground in imparting the motions to the superstructure. On the other hand, buildings in which the soil around the basement levels is exceptionally soft are more accurately modeled by considering the base to be at the foundation level. For those buildings in which the base is above the foundation, it is only reasonable that the ductility of the seismic force-resisting system be continued to the foundation.

C4. STRUCTURAL ANALYSIS PROCEDURES

This chapter provides for the calculation of seismic loads and their effects. No analysis is provided for the vertical motions of ground shaking, except for the approximation introduced by varying the load factor for dead load as specified in 4.4.6. Although the direction of force is critical for some types of structures, such as four legged towers, independent analysis on the two principal axes is sufficient for buildings, given the special provision for combining load effects in 4.4.5.

C4.1 REQUIRED METHOD FOR SEISMIC-RESISTING SYSTEMS. The minimum level of analysis, contained in 4.2, is quite similar to the UBC and is thought appropriate for most buildings. The modal analysis, contained in 4.3, is somewhat similar to the ATC provisions and is appropriate for multistory buildings that are not uniform throughout their height. The use of more sophisticated analyses is discussed in C4.5. The provision of analytical procedures for irregular buildings should not be inferred to encourage such buildings. Indeed, because such buildings have been shown to be more vulnerable in past earthquakes and because their response is considerably more difficult to predict, irregular buildings are discouraged.

It should be noted that soil-structure interaction analyses will generally have the effect of reducing the forces and increasing the deflections in the structure. In some instances, secondary geometrical effects resulting from the increased deflections will have the effect of a net increase on the forces. The Interagency Committee's Manual on site evaluation methods referred to previously, discusses both the technique and appropriateness of soil-structure interaction analyses.

C4.2 ELASTIC STATIC LOAD ANALYSIS. Except where noted, 4.2 is based on sections 2312(d) and (e) of the UBC.

C4.2.1 Base Shear. The SEAOC Commentary is a valuable resource for understanding these provisions. The provision for calculating Z for those buildings on sites with an approved site evaluation is simply a means of calibrating the spectrum for base shear to the site acceleration. If the site acceleration is greater than 40 percent of gravity, the resulting Z will be greater than 1.0. Permanent loads that are fixed to the structure, such as some types of computer installations, should be included in W. When dealing with liquids, W should be the weight of the effective mass.

C4.2.2 Period of Vibration. Although the equations for calculating the period are the same as given in the UBC, the philosophy of their use is more like that given in the ATC provisions. Equations 3 and 4 are very approximate and are based on the periods found in actual buildings. The use of a proper analysis for the period is encouraged, but a limit is placed on the result in order to prevent the use of periods significantly larger than observed in real buildings. Use of unrealistically large values for the period results in low and unsafe values for the base shear.

C4.2.3 Site Coefficient. Two alternative methods are presented. Method I is taken from the ATC, and method II is taken from the UBC. Method II has been used for several years in the highly seismic areas of the country. It requires the determination of a value for the site period, for which SEAOC Standard No. 1, "Determination of the Characteristic Site Period, T_S" is recommended (also published as UBC Standard No. 23-1).

In method I, Soil Profile Type 2 is specified for sites at which the soil properties are not known in detail. It is anticipated that the designer would always be aware of the soil properties at sites actually fitting Soil Profile Type 3 because of the likely need for deep or special foundations.

C4.2.4 Vertical Distribution. The specified distribution is based on buildings in which the stiffness, mass, and strength are relatively consistent from one level to the next. The proceduress of 4.3 will account for the lack of such consistency in stiffness and mass, but more sophisticated analyses are required for substantial differences in strength.

C4.3 ELASTIC DYNAMIC LOAD ANALYSIS. This analysis is only applicable to multistory buildings. The provisions are based on the ATC procedures for modal analysis, although they are not as detailed. The intent is to establish the basic limits for such analyses rather than to completely specify a method. The design spectrum is not from the ATC, but is simply consistent with the spectrum used in 4.2.1. Although the combination of modes by means of the common square root of the sum of the squares method is specified, more precise combinations may be necessary for buildings with closely coupled modes.

C4.3.3 Design Values. The limits placed on the base shear are intended to prevent misuse of advanced analyses. The reason to use the advanced analysis is to determine the distribution of seismic forces more accurately, not to reduce the overall force.

C4.4 ELASTIC LOAD EFFECT ANALYSIS. Except where noted, the provisions of 4.4 are takken from sections 2312(e) and (f) of the UBC.

C4.4.1 Shear. It is frequently useful to idealize the relative rigidity of horizontal to vertical bracing systems as "flexible" or "rigid." For "flexible" diaphragms (or bracing systems), the shear is distributed from the diaphragm to the vertical elements by modeling it as a beam on unyielding supports. The amount of shear in a particular wall or frame would depend on the shear in the diaphragm spans that are tributary to it. There would be no effect of continuity in the diaphragm, since the shear strain normally dominates the flexural strain in the diaphragm. For the "rigid" diaphragm, the shear is distributed by modeling the diaphragm as a rigid beam on yielding supports. The amount of shear in a particular wall would depend on its rigidity in relation to all other walls. Buildings with plywood deck diaphragms and masonry or concrete shear walls are normally considered to have "flexible" diaphragms, while buildings with concrete slabs of normal proportions without large openings are normally considered to have "rigid" diaphragms. Where the horizontal and vertical bracing systems have equivalent rigidities, a more complex analysis is required. For such cases, it is normally acceptable to conduct two simple analyses, one for each of the previously described extremes, and use the more conservative values for design.

C4.4.2 Horizontal Torsion. In consideration of the commentary on 4.4.1, the phrase, "Where the vertical resisting elements depend on diaphragm action for shear distribution at any level," can be interpreted as "where the horizontal bracing systems cannot be characterized as 'flexible'."

C4.4.4 Overturning. The reduction allowed for tall buildings is taken from the ATC. It is allowed because the story forces defined in 4.2.4 overestimate the overturning moment in such structures. Those story forces are designed to produce story shear forces that are consistent with the envelope of maximum story shear forces found in modal analysis for such structures over a wide range of periods. Similar story forces could be specified to produce overturning moments found in such modal analyses, but the forces would not be identical. It is simpler to adjust the overturning moments to remove the discrepancy. The reduction is unrelated to live load reduction for area, to load combination factors, or to rocking of a building foundation. When applying this reduction, designers should be particularly alert to situations where uplift from overturning is combined with minimum likely dead load.

The additional reduction allowed at the soil-foundation interface accounts for the rocking on a foundation in a very approximate way. It is also taken from the ATC. Both reductions are allowed in this standard in an attempt to partially account for the discrepancy between the lack of overturning failures of buildings and the fact that design force levels are below the real force levels in a strong earthquake.

C4.4.5 Orthogonal Effect. This provision is similar to the ATC provisions. Vertical elements at or near the corners of buildings are a typical example of a component that is utilized in both directions and would be affected by this provision. For conventional rectangular buildings with horizontal and vertical framing, the beams, girders, diagonals of individual braced frames, and shear walls not continuous with orthogonal walls are examples of components that are utilized in only one direction for seismic resistance. It is not the intent of this section to require a great amount of extra calculation.

C4.4.6 Vertical Motion Effect. This provision is also taken from the ATC. It accounts for vertical accelerations in an approximate way. The $\pm$ 20 percent does not reflect the maximum vertical response likely, however, since it was considered unlikely for the maximum horizontal and vertical responses to occur simultaneously. Changing the coefficient on dead load for horizontal cantilevers accounts for a dynamic response of an upward acceleration of 120 percent of gravity.

C4.4.7 Displacements. The amplification of the displacements by 3.0/K is different than in the UBC, which specifies 1.0/K. The SEAOC Commentary states that the real displacements are likely to be about 3.0/K, and the UBC uses this factor in some provisions. As explained in C3.1.4, the amplification by 3.0/K is used throughout in this standard.

C4.5 INELASTIC ANALYSIS. The specified methods of analysis are the minimum levels deemed appropriate. More sophisticated analyses are frequently necessary for adequate prediction of the structural response to ground shaking. It is important to realize that dynamic analysis should be used primarily to determine a more accurate distribution of forces and deformations among the various parts of a structure, and not to bring about a significant reduction in the overall base shear. The philosophy behind the lower limit on base shear is similar to that in the ATC.

C4.6 ANALYSIS OF OTHER STRUCTURAL ELEMENTS. This section is taken from 2312(g) of the UBC except for the minimum interconnection force, which is taken from the ATC.

C5 DESIGN AND CONSTRUCTION STANDARDS FOR STRUCTURAL MATERIALS

This standard makes reference to widely accepted national standards for design and construction provisions for specific structural materials. This is in keeping with both the current practice of Federal agencies and Federal policy as set forth in Circular A-119 of the Office of Management and Budget.

In many instances the materials' provisions in the UBC are derived from these same national standards. Thus, the application of this chapter should produce results similar to present practices and to the UBC.

The provisions of this standard frequently modify or take exception to the referenced standards, in which cases this standard shall control (for example, most of the provisions of chapter 6). With the exception of the standards for masonry, the references are listed such that the designer or contractor will make use of each standard as it is applicable. As discussed in C5.4, this standard allows some choice between different standards for masonry.

The set of standards referenced does not form a complete set of standards for construction or for structural design. Only those standards which would fulfill some need in carrying out the provisions of this standard are referenced. In some instances the standards referenced here may conflict with other standards referenced by Federal agencies. For issues concerned with seismic safety, priority should be given to this standard and its references. The editions listed for each standard are the latest available at the time of development of this standard. Revisions to these editions should be examined by designers and designated authorities on a timely basis.

C5.1 STEEL. The "Design Manual for Floor Decks and Roof Decks" published by the Steel Deck Institute does not provide allowable diaphragm shear values for steel deck diaphragms. It is expected that the Steel Deck Institute will soon adopt a standard design manual for diaphragm construction that would include such values; however, until such a document becomes an accepted standard, the Tri-Services Manual is an available source of design values for steel deck diaphragms.

C5.2 MASONRY. There is more regional diversity in the design and construction practices for masonry than for other construction materials. Therefore, the standards referenced for masonry tend to overlap more than the standards referenced for other materials. The UBC is widely used in the West but rarely used in the East, where the standards of ANSI, ACI, NCMA, and BIA are more widely used. Although uniformity has some positive aspects, uniformity in masonry practices will not be accomplished by this standard alone. It is expected that current regional practices will continue under this standard, although a general preference for the UBC is expressed for use in the higher zones, regardless of the geographic region.

C6 STRUCTURAL DESIGN DETAILS

The details of proportioning, reinforcing, and connecting structural members are of extreme importance in achieving successful performance in earthquakes. This is because the details have an enormous effect on the capability of the seismic force-resisting system to dissipate energy through ductility and damping. Five widely accepted standards for structural detailing for seismic performance are referenced at several points in this chapter. The referencing is consistent with the philosophy expressed in C5 and it has the added benefit of brevity in this standard. The designer need not have all five references on hand. Because they are required by Chapter 5, he would already have the standards of ACI and AISC on hand. The UBC, SEAOC, and Tri-Services Manual are always used as alternates, so no more than one is absolutely necessary. UBC is referenced here because of its familiarity to those experienced in seismic design and because of the convenience of its self-contained nature. SEAOC is referenced because it is the original source of much seismic provisions of the UBC and because it is accompanied by a valuable commentary. The Tri-Services Manual is referenced because of its unique treatment of several levels of performance for ductile moment-resisting frames of reinforced concrete.

C6.1 MOMENT-RESISTING FRAMES. This section is taken from 2312(j)C and F of the UBC.

C6.1.2 Concrete Frames. Unless the actual drift is substantially less than the maximum allowable drift, the repeated distortions imposed on a concrete frame, whether it is a part of the seismic force-resisting system or not, are likely to cause substantial cracking and spalling of the concrete. Thus, any concrete frame used in the seismic force-resisting system needs the reinforcement required for ductile moment-resisting frames in order to maintain its integrity. Likewise, any concrete frame on the exterior of a building needs the same reinforcement to prevent spalling hazardous large chunks of concrete unless that building has a very stiff seismic force-resisting system.

C6.2 STEEL DUCTILE MOMENT-RESISTING SPACE FRAMES. This section is based on 2312(j)F of the UBC. The following table summarizes the appropriate K factor, and thus the category of the seismic force-resisting system, for the two common types of steel frames used as seismic force-resisting systems:

	Zone 1	2(I=1)	2(I>1)	3&4
Steel frame in compliance with section 2722 of the UBC	0.67	0.67	0.67	0.67
Ordinary steel frame with joints meeting AISC Type I	0.67	0.67	1.0	1.0

C6.3 REINFORCED CONCRETE DUCTILE MOMENT-RESISTING SPACE FRAMES. This section is based on 2312(j)1F and 2625(b) and (c) of the UBC. The following table summarizes the appropriate K factor for three types of reinforced concrete frames:

	Zone 1	2(I=1)	2(I>1)	3&4
Highly ductile frame meeting section 2626 of the UBC	0.67	0.67	0.67	0.67
Moderately ductile frame meeting 6.3.1 and 6.3.2 **	0.67	0.67	*	*
Ordinary frame meeting ACI 318 without Appendix A	1.0	*	*	*

* system not permitted

** the UBC specifies a different K factor for this system, K = 1.0.

C6.4 BRACED FRAMES. These requirements are taken from 2312(j)1G and 2627(b) of the UBC; the only difference is in the application of the requirements in zones 1 and 2. Higher member and connection capacity is called for braced frames to assure the necessary ductility and to reduce the possibility of non-ductile connection failures.

It is possible to construct braced frames with diagonal members that are ineffective in resisting compression due to their extreme slenderness. Repeated inelastic straining of such frames leads to very large deflections because each cycle causes a net increase in the length of the diagonals; the phenomenon is often called "slap-back." Depending on the type of structure, this behavior may be very undesirable. The ATC provisions effectively prohibit such bracing systems for buildings over two stories in the highest seismic zones by requiring that the compressive strength of members in braced frames be at least 50 percent of the required tensile strength.

C6.4.1 Required Capacity. The term "full capacity" means the true failure load for the member. Thus strain hardening should be taken into account.

C6.5 REINFORCED CONCRETE SHEAR WALLS. This section is taken from 2312(j)1H of the UBC, except that it is not applied to the lower seismic zones.

C6.6 REINFORCED MASONRY WALLS. The table of minimum reinforcement in masonry walls is taken from the Tri-Service Manual and the seismic regulations of the Veterans Administration. It is not identical to the UBC.

C6.7 DIAPHRAGMS. This section is based on 2312(j)2D and 3A of the UBC, with significant simplification that is an accord with current practice. The need for these requirements was demonstrated by several failures in the 1971 San Fernando earthquake.

C6.8 OPENINGS IN SHEAR WALLS AND DIAPHRAGMS. This requirement is roughly based on a similar requirement in the ATC provisions. The importance of providing continuity in the chords of plate-like elements is often overlooked.

C6.9 CONCRETE AND MASONRY ELEMENTS. This section is taken from 2312(j)2B and 2310 of the UBC.

C6.10 FOUNDATIONS

C6.10.1 Ties Between Foundation Units. The provision for pile caps is taken from 2312(j)3B of the UBC; the provision for spread footings is original. In both cases the concerns are to assure that the foundation transmits the ground motion uniformly to the structure and to allow adjacent foundation units to participate in sharing lateral force overloads.

C6.10.2 Pile Cap Connections. The ATC provisions require the connection between the pile cap and the pile to be reinforced. One reason for a minimum tensile capacity, separate from consideration of overturning resistance, is vertical ground motions. The requirement given in this section is roughly equivalent to the ATC requirements, but it is stated in a performance-oriented fashion rather than in a series of prescriptive requirements for various types of piles.

C6.10.3 Concrete Piles. In addition to the need for tensile capacity implied by the preceding section, piles also need a minimum level of ductility, particularly near the top. Once again, these provisions are similar to the ATC provisions, although they are less detailed. Note that some types of metal casing might substitute for one type of reinforcement, but not the other. Some corrugated casings might fulfill the function of the transverse reinforcement without fulfilling the function of the longitudinal reinforcement.

C7 NONSTRUCTION DESIGN REQUIREMENTS

C7.1 ANCHORAGE FOR INERTIAL FORCES. This section is based on 2312(g) of the UBC. One difference is that consideration of nonstructural components has been separated from structural components in this document. The ATC provisions give some guidance on ducts and piping that are small enough so that special seismic restraints need not be designed. The Tri-Services Manual and the General Services Administration's Design Guidelines for Earthquake Resistance of Buildings both gives specific recommendations for the anchorage and protection of nonstructural components.

C7.2 DISTORTION COMPATIBILITY FOR EXTERIOR PANELS. This section is the same as 2312(j)3C of the UBC. In discussing a similar provision, the SEAOC Commentary notes that the force specified for fasteners attaching the connector to the panel or the structure shall be taken in any direction, not just horizontal, and that it need not be combined with other forces.

C7.3 PROTECTION AGAINST SECONDARY HAZARDS. This provision is based on a similar provision in the ATC.

C7.4 FUNCTIONALITY OF ESSENTIAL ELEMENTS. This provision is also based on a similar provision in the ATC. Full consideration of design for functional capability immediately following a major earthquake is beyond the scope of this standard.

C7.5 REINFORCEMENT OF CONCRETE AND MASONRY. This section is the same as 2312(j)3C of the UBC. It requires the same minimum reinforcement in zones 3 and 4 for nonstructural components as 6.9.1 does in zones 2, 3 and 4 for structural components.

C8 CONSTRUCTION QUALITY CONTROL

This chapter is loosely based on section 306 of the UBC. The intent is to cover those items whose successful performance in an earthquake is strongly dependent on sound quality control. It is not the basis for a complete quality control program, but should supplement existing agency programs.

C8.2 REQUIRED SPECIAL INSPECTION. In identifying the needed special inspection, the standard follows the UBC particularly closely. Although there is not universal agreement on the subject of special inspection, there is no intent to create new problems with this standard. It should be noted that continuous special inspection is not synonymous with full-time inspection.