The New York City Seismic Code: Local Law 17/1995

BY

New York City Seismic Code Committee
New York City, New York

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Multidisciplinary Center for Earthquake Engineering Research

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MULTIDISCIPLINARY CENTER FOR EARTHQUAKE ENGINEERING RESEARCH
A National Center of Excellence in Advanced Technology Applications

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Preface

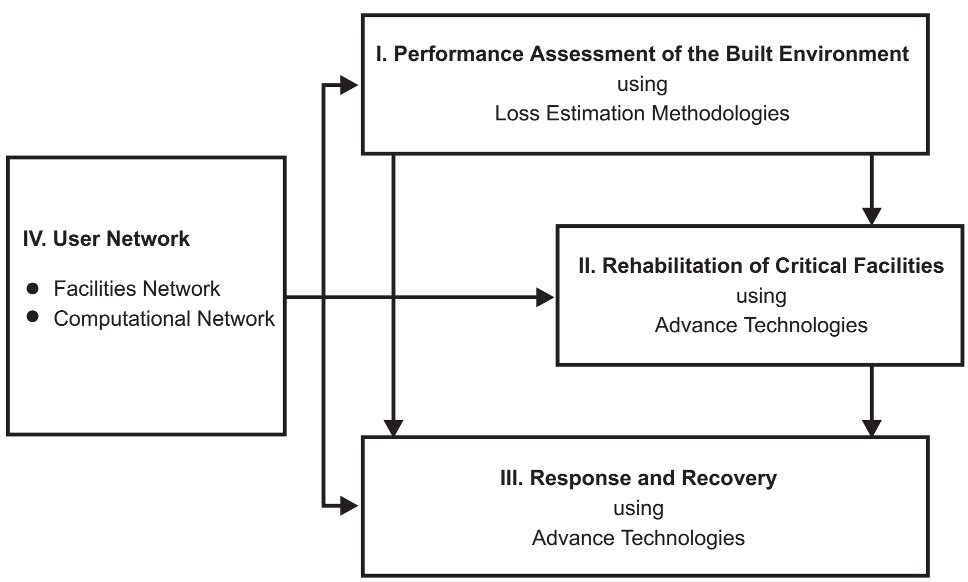
The Multidisciplinary Center for Earthquake Engineering Research (MCEER) is a national center of excellence in advanced technology applications that is dedicated to the reduction of earthquake losses nationwide. Headquartered at the University at Buffalo, State University of New York, the Center was originally established by the National Science Foundation in 1986, as the National Center for Earthquake Engineering Research (NCEER).

Comprising a consortium of researchers from numerous disciplines and institutions throughout the United States, the Center's mission is to reduce earthquake losses through research and the application of advanced technologies that improve engineering, pre-earthquake planning and post-earthquake recovery strategies. Toward this end, the Center coordinates a nationwide program of multidisciplinary team research, education and outreach activities.

MCEER's research is conducted under the sponsorship of two major federal agencies: the National Science Foundation (NSF) and the Federal Highway Administration (FHWA), and the State of New York. Significant support is derived from the Federal Emergency Management Agency (FEMA), other state governments, academic institutions, foreign governments and private industry.

The Center's NSF-sponsored research is focused around four major thrusts, as shown in the figure below:

- quantifying building and lifeline performance in future earthquake through the estimation of expected losses;
- developing cost-effective, performance based, rehabilitation technologies for critical facilities;
- improving response and recovery through strategic planning and crisis management;
- establishing two user networks, one in experimental facilities and computing environments and the other in computational and analytical resources.



Acknowledgment

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Klaus Jacob was Chairman of the Geotechnical Subcommittee, Joseph Kelly of the Loads Subcommittee, Mohamed Ettouney of the Detailing Subcommittee and Deborah Beck was Chairwoman of the Nonstructural Subcommittee. The Committee was established by Commissioner of Buildings Charles Smith in 1989, and encouraged and guided by Commissioners Rudolph Rinaldi and Joel Miehle. Commissioner Joel Miele oversaw the completion of the code and its signing by Mayor Rudolph Giuliani on 21 February 1995 as Local Law 17/95. The Law took effect on 21 February 1996.

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Section 1
Introduction

Concern over the possibility of damaging earthquakes in the eastern United States developed through the 1970's as a result of improved knowledge of both the history of eastern U.S. earthquake occurrences and effects and of the underlying tectonics. These subjects are treated elsewhere in this report. Of interest here is the fact that this improved knowledge led to changes in both national and local building codes through the 1970's. The 1973 Uniform Building Code (UBC, 1973) included northern New York State, Boston, Massachusetts and Charleston, South Carolina in the next to highest seismic zone. The subsequent ATC 3-06 (ATC, 1978) maps, developed for equal probabilities of occurrence and intensity nationwide, refined the zoning further. At the same time, the Nuclear Regulatory Commission was carefully evaluating the seismic hazards and risks at potential plant sites. In an advisory note (NRC, 1983), the Nuclear Regulatory Commission commented:

> "Because the geologic and tectonic features of the Charleston region are similar to those in other regions of the eastern seaboard, we conclude that although there is no recent or historical evidence that other regions have experienced strong earthquakes, the historical record is not of itself sufficient ground for ruling out the occurrence in these other regions of strong seismic ground motions similar to those experienced near Charleston in 1886 (M=7). Although the probability of strong ground motion due to an earthquake in any given year at a particular location on the eastern seaboard may be very low, deterministic and probabilistic evaluations of the seismic hazard should be made for individual sites on the eastern seaboard to establish the seismic engineering parameters for critical facilities."

The particular seismicity of New York City (NYC), where the seismicity is "moderate," can be characterized as follows:

- Earthquakes with intensity of about MM (Modified Mercalli) Intensity VII have occurred every 100 years in the New York City area.
- Regional seismicity indicates that earthquakes of MM Intensity VII are likely to occur on average every 100-200 years (i.e., 20 to 40 percent probability of occurrence in 50 years).
- Larger earthquakes, with MM Intensity VIII-IX, or magnitude 5.75 to 6.75 (probable upper bound range) may occur.
- Even larger magnitude and/or higher intensities, at very low levels of probability, cannot be excluded.
- NYC seismicity is very similar to that of the Boston area, where seismic design requirements are in effect.

These were the conclusions, in 1986, of a Committee (Nordenson, 1987) of the New York Association of Consulting Engineers formed to assess the need for seismic design in New York City.

1.1 Issues

Several issues, or questions, merit continued consideration as seismic design requirements are evaluated for use in places such as New York City.

- What are the maximum probable (say with a 10 to 20% probability of occurrence in 50 years) and maximum credible (say with a 2% probability of occurrence in 50 years) earthquakes for the region, and what would be the nature of the resulting ground motion at varying types of sites?

- How difficult and costly is it to provide seismic resistance to buildings?

- What would the human and economic effects of an occurrence in New York City of the maximum probable or maximum credible earthquakes be, both locally and nationally?

- To what extent will improved building codes mitigate these consequences? What other measures are needed? Should these take precedence over improved building codes? (e.g., post disaster planning, upgrading medical or other emergency facilities and lifelines, abatement of hazardous buildings, etc.)?

- What corollary benefits are there to adopting seismic design requirements in improved design and construction practices?

Clearly it is necessary to assess the need for seismic code provisions alongside other human needs. With limited resources, is it right to allocate them to seismic resistance, given the infrequency of earthquakes, as against housing, or education? Especially as it will affect such a small percentage of the building stock? Yet can one ignore an event, with even a low probability, of such severe consequences?

1.2 Overview of the New York City Seismic Code

1.2.1 Background

The 1982 edition of the ANSI A58.1 *Minimum Design Loads in Buildings and Other Structures* (ANSI, 1982) included a new seismic design section, modeled after the UBC and ATC 3-06, which placed New York City in Seismic Zone 2 (vs. 4 for California). This would in turn have triggered the application of the ductile design provisions of the ACI-318 code for concrete design. The New York City Building Commissioner asked the New York Association of Consulting Engineers (NYACE) to review the matter. The initial response, in the summer of 1984, was to recommend that such requirements be omitted. However, after some discussion, it was decided that a group of seismologists and engineers review the issues and advise NYACE. The Committee's conclusions have been reported above. Following this, the NYACE Board unanimously recommended to the Commissioner, in June 1987, that seismic design be mandated in New York City, and that these should follow the 1988 UBC (UBC, 1973).

At the same time (1986), the National Center for Earthquake Engineering Research (NCEER) was established in Buffalo, New York. Several conferences were organized as a result (Jacob et al., 1987; Jacob and Turkstra, 1989) which addressed the particular issues of seismic hazard and design in the eastern U.S. Following these, the Commissioner appointed, in April 1989, a Seismic Code Committee to draft seismic code provisions for New York City. This Committee included engineers, seismologists, and representatives of the building industries and real estate community. The Seismic Code Committee voted unanimously to submit its final report to the Commissioner in early 1991, and the report was submitted on 18 April 1991.

1.2.2 Principles and Approach

The development of the code has been guided by several agreed principles:

- to focus on provisions for the prevention of life threatening collapse of buildings and components and not the protection of property *per se*.

- to seek improvements, not radical changes in construction practices.

- to modify the characterization of the loading to reflect local seismicity.

The Committee divided its efforts into (1) Geotechnical (2) Loads and Systems (3) Detailing (4) Economic Implications and (5) Nonstructural Subcommittees. After several months deliberation, the Committee decided to adapt the 1988 UBC, following the above stated principles, for inclusion in the New York City Building Laws.

The Committee's work therefore consisted of preparing amendments to the provisions of the 1988 UBC, except that the steel provisions of the 1990 Supplement to the UBC were referenced. The following summarizes these changes as agreed by the Subcommittees. It should be noted that one and two-family dwellings not more than three stories in height are exempted from the provisions.

1.2.3 Geotechnical Subcommittee

1.1 Seismic Zone: NYC is deemed to be in Seismic Zone 2A with a factor, or effective zero period acceleration of 0.15 in S_1 type rock.

1.2 Site geology and characteristics: Soils are classified with reference to the New York City classification system. A new soil type, S_0, is introduced for hard rock, with a factor of 0.67. The Massachusetts code provisions for evaluating soil liquefaction potential are introduced.

1.3 Foundations: Pile caps and caissons are to be connected, unless the soil can provide equivalent restraint. Wood stud wall plates and sills are to be bolted to foundations.

1.4 Site specific response spectra: These may be used for design, and are recommended for S_4 soil type sites. However, the calculated base shear must be scaled to the equivalent static value (see 2.5 below).

1.2.4 Loads and Systems Subcommittee

2.1 Analysis method: The equivalent static lateral force procedure is accepted for all buildings except irregular buildings over 400 feet in height in Occupancy Categories I through III. These include emergency and hazardous facilities, and special occupancy structures, including those with more than 5,000 occupants. All regular buildings, of any height, can be designed by the equivalent static procedure.

2.2 Dual system: These are systems with shear walls or braced frames and moment frames working in parallel. The moment frames are required to carry at least 25% of the lateral load considered on their own. The system is intended to possess sufficient ductility by virtue of the moment frames. The New York City Seismic Code adds the provision that the walls or braced frame have sufficient shear capacity to carry 75% of the cumulative story shear (not overturning). This is in recognition of the interaction effects of tall frame/wall structures, and the desire to insure possible shear redistribution.

2.3 Period determination: The coefficients (C_t) for estimating periods have been revised for concrete moment frames, dual systems and eccentric braced frames.

2.4 Vertical component: The coefficient has been reduced in line with the local seismicity.

2.5 Scaling of dynamic analysis results: The UBC provisions have been adjusted to allow scaling to a reduced equivalent static coefficient for periods above 3 seconds. This reflects the lesser ordinates, at long periods, of response spectra developed for eastern U.S. sites. The coefficient C is calculated as 1.80 S/T for T > 3 seconds.

2.6 Building separation: Building separation is limited to 1 inch for every 50 feet of total building height. Thus, a building 400 feet tall would, at the typical 120 foot zoning setback elevation, be separated by 8 inches from the adjacent building. The provision notes that "smaller separation may be permitted when the effects of pounding can be accommodated without collapse of the building." This provision is intentionally empirical to reflect the uncertain knowledge of the effects of pounding on building collapse.

2.7 Structural systems: Several new systems are recognized as follows:

- Ordinary concrete moment-resisting frames limited to sites with soils S_0 to S_2 and under 160 feet in height.

- Dual systems combining concrete and reinforced masonry shear walls and braced frames with "Special," "Intermediate" and "Ordinary" moment-resisting frames.

1.2.5 Detailing Subcommittee

3.1 Masonry: The ACI 530-88 *Building Code Requirements for Masonry Structures* has been used as a reference standard with modifications:

- All masonry bearing and shear walls shall be reinforced, regardless of whether they are designed as reinforced or unreinforced walls. Maximum spacing of vertical bars is 10 feet.

- All nonbearing back-up or infill walls and nonbearing partitions shall have minimum one-way only reinforcement to supports.

3.2 Concrete: ACI 318-89 is referenced without modifications.

3.3 Steel: The 1988 UBC requirements of Section 2723 are referenced without major modifications. The use of LFRD is prohibited for the design of seismic resisting elements.

3.4 Timber: The AITC and APA provisions for seismic design of plywood or other diaphragms and shear walls are referenced.

1.2.6 Nonstructural Subcommittee

4.1 Part or portion of structures: UBC Table 23-P (see page 27) is revised for nonbearing walls so that only those around means of egress are to be designed for the specified lateral forces.

4.2 Exterior nonstructural components: Provisions for interior components (e.g., access floors, ceilings, etc.) have been eliminated. Exterior appendages, chimneys, stacks, trussed towers, tanks on legs and exterior tanks and vessels are to be designed for seismic effects.

1.3 Conclusion

Mayor Rudolph Guiliani signed the New York City Seismic Code, known as Local Law 17, on 21 February 1995 to take effect a year from that day.

Section 2
The New York City Seismic Code

2.1 Introduction

LOCAL LAW 17/95

To amend the administrative code of the city of New York, in relation to the design and construction of buildings, structures and portions thereof to resist the effects of earthquakes.

Be it enacted by the Council as follows:

Section 1. Article 5 of subchapter 9 of chapter 1 of title 27 of the administrative code of the city of New York is amended to read as follows:

ARTICLE 5

WIND LOADS *AND EARTHQUAKE LOADS*

> **27-569 Wind loads *and earthquake loads. (a) WIND LOADS.* -** The structural frame and exterior components of all buildings, signs, tanks, and other exposed constructions shall be designed to resist the pressures due to wind as prescribed in reference standard RS 9-5. Wind shall be assumed to act from any direction. For continuous framing, the effects of partial loading conditions shall be considered.
>
> ***(b) EARTHQUAKE LOADS. -*** *Every building, structure and portion thereof shall, at a minimum, be designed and constructed to resist the effects of seismic ground motions as prescribed in reference standard RS 9-6.*

Section 2. The listing for "Unreinforced Masonry" under the column entitled "Operations on Structural Elements That Shall Be Subject to Controlled Inspection" of table 10-2 of subdivision (c) of section 27-588 of the administrative code of the city of New York is amended to read as follows:

> Placement and bedding of units and sizes of members including thickness of walls and wythes; sizes of columns; *cleanouts*; and provisions for curing and protection against freezing for all masonry construction proportioned on the basis of structural analysis as described in section four of reference standard RS 10-1B, unless such operations are specifically not designated for controlled inspection.

Section 3. The listings for "Reinforced Masonry" and "Unreinforced Masonry" under the column entitled "Operations on Structural Elements That Are Not Subject to Controlled Inspection" of table 10-2 of subdivision (c) of section 27-588 of the administrative code of the city of New York are amended by repealing item (1) under both "Reinforced Masonry" and "Unreinforced Masonry" and, in each case, renumbering items (2), (3) and (4) to be (1), (2) and (3), respectively.

Section 4. The opening paragraph of section 27-594 of the administrative code of the city of New York is amended to read as follows:

> Dead loads, live loads (including impact) and reduced live loads, where applicable, shall be considered as basic loads. Wind, *earthquake*, thermal forces, shrinkage, and unreduced live loads (where live load reduction is permitted by subchapter nine of this chapter) shall be considered as loads of infrequent occurrence. Members shall have adequate capacity to resist all applicable

combinations of the loads listed in subchapter nine of this chapter, in accordance with the following:

Section 5. Subdivision (a) of section 27-670 of the administrative code of the city of New York is amended to read as follows:

(a) Earth and ground water pressure. Every foundation wall or other wall serving as a retaining structure shall be designed to resist, in addition to the vertical loads acting thereon, the incident lateral earth pressures and surcharges, plus hydrostatic pressures corresponding to the maximum probable ground water level. *Retaining walls shall be designed to resist at least the superimposed effects of the total static lateral soil pressure, excluding the pressure caused by any temporary surcharge, plus an earthquake force of 0.045* $w_s h^2$ *(horizontal backfill surface), where* w_s *equals unit weight of soil and h equals wall height. Surcharges which are applied over extended periods of time shall be included in the total static lateral soil pressure and their earthquake lateral force shall be computed and added to the force of 0.045* $w_s h^2$*. The earthquake force from backfill shall be distributed as an inverse triangle over the height of the wall. The point of application of the earthquake force from an extended duration surcharge shall be determined on an individual case basis. If the backfill consists of loose saturated granular soil, consideration shall be given to the potential liquefication of the backfill during the seismic loading using reference standard RS 9-6.*

Section 6. The list of referenced national standards of reference standard RS-9 of the appendix to chapter 1 of title 27 of the administrative code of the city of New York is amended by adding a new standard to read as follows:

2.2 Loads

UBC SECTION 2312
Earthquake Regulations with Accumulative Supplement 1990

Section 7. Reference standard RS-9 of the appendix to chapter 1 of title 27 of such code is amended by adding a new reference standard RS 9-6 to read as follows:

The following text includes the amendments to the 1988 UBC within the text of the UBC provisions. These amendments are shown in italics.

REFERENCE STANDARD RS 9-6
EARTHQUAKE LOADS

UBC SECTION 2312-1990 Earthquake Regulations with Accumulative Supplement

MODIFICATIONS - *The provisions of UBC Section 2312 shall be subject to the following modifications. The subdivisions, paragraphs, subparagraphs and items are from this section.*

Earthquake Regulations

Sec. 2312 (a) **General.** 1. **Minimum seismic design**. *The following types of construction shall, at a minimum, be designed and constructed to resist the effects of seismic ground motions as provided in this section:*

- new structures on new foundations;
- new structures on existing foundations; and
- enlargements in and of themselves on new foundations.

Buildings classified in New York City occupancy group J-3 and not more than three stories in height need not conform to the provisions of this section.

The Commissioner may require that the following types of construction be designed and constructed to incorporate safety measures as necessary to provide safety against the effects of seismic ground motions at least equivalent to that provided in a structure to which the provisions of this section are applicable:

- new buildings classified in occupancy group J-3 and which are three stories or less in height; and
- enlargements in and of themselves where the costs of such enlargement exceeds sixty percent of the value of the building.

Pursuant to section 27-191 of the code the Commissioner shall have the authority to reject an application for a building permit which fails to comply with the requirements of this section.

2. **Seismic and wind.** When the code-prescribed wind design produces greater effects, the wind design shall govern, but detailing requirements and limitations prescribed in this and referenced sections shall be followed.

(b) **Definitions**. For the purposes of this section certain items are defined as follows:

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

BASE SHEAR, V, is the total design lateral force or shear at the base of a structure.

BEARING WALL SYSTEM is a structural system without a complete vertical load-carrying space frame. See Section 2312 (d) 6 B.

BOUNDARY ELEMENT is an element at edges of openings or at perimeters of shear walls or diaphragms.

BRACED FRAME is an essentially vertical truss system of the concentric or eccentric type which is provided to resist lateral forces.

BUILDING FRAME SYSTEM is an essentially complete space frame which provides support for gravity loads. See Section 2312 (d) 6 C.

COLLECTOR is a member or element provided to transfer lateral forces from a portion of a structure to vertical elements of the lateral force-resisting system.

CONCENTRIC BRACED FRAME is a braced frame in which the members are subjected primarily to axial forces.

DIAPHRAGM is a horizontal or nearly horizontal system acting to transmit lateral forces to the vertical resisting elements. The term "diaphragm" includes horizontal bracing systems.

DIAPHRAGM CHORD is the boundary element of a diaphragm or shear wall which is assumed to take axial stresses analogous to the flanges of a beam.

DIAPHRAGM STRUT (drag strut, tie, collector) is the element of a diaphragm parallel to the applied load which collects and transfers diaphragm shear to vertical resisting elements or distributes loads within the diaphragm. Such members may take axial tension or compression.

DRIFT see STORY DRIFT.

DUAL SYSTEM is a combination of a special or intermediate moment-resisting space frame and shear walls or braced frames designed in accordance with the criteria of Section 2312 (d) 6 E.

ECCENTRIC BRACED FRAME (EBF) *is a steel-braced frame designed in conformance with reference standard RS 10-5C.*

ESSENTIAL FACILITIES *are those structures which are necessary for emergency operations subsequent to a natural disaster.*

FLEXIBLE ELEMENT or system is one whose deformation under lateral load is significantly larger than adjoining parts of the system. Limiting ratios for defining specific flexible elements are set forth in Section 2312 (e) 3 B (ii), 2312 (e) 6 or 2312 (g) 2.

HORIZONTAL BRACING SYSTEM is a horizontal truss system that serves the same function as a diaphragm.

INTERMEDIATE MOMENT-RESISTING FRAME (IMRF) *is a concrete frame designed in accordance with the requirements of Chapters 1 through 20 and Sections 21.1, 21.2 and 21.9 of reference standard RS 10-3.*

LATERAL FORCE-RESISTING SYSTEM is that part of the structural system assigned to resist lateral forces.

MOMENT-RESISTING FRAME *is a frame in which members and joints are capable of resisting forces primarily by flexure.*

ORDINARY MOMENT-RESISTING FRAME (OMRF) *is a moment-resisting frame conforming to the requirements of Chapters 1 through 20 of reference standard RS 10-3 or reference standards RS 10-5A and RS 10-5C but not meeting the special detailing requirements for ductile behavior.*

ORTHOGONAL EFFECTS are the effects on structural elements common to the resisting systems along two orthogonal axes due to earthquake forces acting in a direction other than those axes.

P-DELTA EFFECT is the secondary effect on shears, axial forces and moments of frame members induced by the vertical loads acting on the laterally displaced building frame.

REINFORCED MASONRY SHEAR WALL *is that form of masonry wall construction in which reinforcement acting in conjunction with masonry is used to resist lateral forces parallel to the wall and which is designed using reinforcement in conformance with Chapter 7 of reference standard RS 10-2.*

SHEAR WALL is a wall designed to resist lateral forces parallel to the plane of the wall (sometimes referred to as a vertical diaphragm).

SOFT STORY is one in which the lateral stiffness is less than 70 percent of the stiffness of the story above. See Table No. 23-M.

SPECIAL MOMENT-RESISTING FRAME (SMRF) *is a moment-resisting frame conforming to reference standards RS 10-3 or RS 10-5A and RS 10-5C and specially detailed to provide ductile behavior by complying with the requirements of Chapters 1 through 20 and Sections 21.1 through 21.8 of reference standard RS 10-3 or reference standards RS 10-5A and RS 10-5C.*

STORY is the space between levels. Story *x* is the story below Level x.

STORY DRIFT *is the displacement of one level relative to the level above or below, including translational and torsional deflections.*

STORY DRIFT RATIO is the story drift divided by the story height.

STORY SHEAR, V_x, is the summation of design lateral forces above the story under consideration.

STRENGTH is the useable capacity of a structure or its members to resist load within the deformation limits prescribed in this section and referenced sections.

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

VERTICAL LOAD-CARRYING FRAME *is a frame designed to carry all vertical gravity loads.*

WEAK STORY is one in which the story strength is less than 80 percent of that of the story above. See Table No. 23-M.

(c) **Symbols and Notations**. The following symbols and notations apply to the provisions of this section:

A_c = The combined effective area, in square feet, of the shear walls in the first story of the structure.

A_e = The minimum cross-sectional shear area in any horizontal plane in the first story, in square feet, of a shear wall.

A_x = Delete this term.

C = Numerical coefficient specified in Section 2312 (e) 2 A.

C_p = Numerical coefficient specified in Section 2312 (g) and given in Table No. 23-P.

C_t = Numerical coefficient given in Section 2312 (e) 2 B.

D_e = The length, in feet, of a shear wall in the first story in the direction parallel to the applied forces.

di = Horizontal displacement at Level i relative to the base due to applied lateral forces, f, for use in Formula (12-5).

f_i = Lateral force at Level i for use in Formula (12-5).

F_i,F_n,F_x = Lateral force applied to Level i, n, or x, respectively.

F_p = Lateral forces on a part of the structure.

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

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 = Importance factor given in Table No. 23-L.

Level i = Level of the structure referred to by the subscript i. "i = 1" designates the first level above the base.

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

Level x = That level which is under design consideration. "x = 1" designates the first level above the base.

R_w = Numerical coefficient given in Tables Nos. 23-0 and 23-Q.

S = Site coefficient for soil characteristics given in Table No. 23-J.

T = Fundamental period of vibration, in seconds, of the structure in the direction under consideration.

V = The total design lateral force or shear at the base.

V_x = The design story shear in Story x.

W = The total seismic dead load defined in Section 2312 (e) 1.

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

w_{px} = The weight of the diaphragm and the elements tributary thereto at Level x, including applicable portions of other loads defined in Section 2312 (e) 1.

W_p = The weight of an element or component.

Z = Seismic zone factor given in Table No. 23-1.

(d) ***Criteria Selection.*** *1.* ***Basis for design****. The procedures and limitations for the design of structures shall be determined considering site characteristics, occupancy, configuration, structural system and height in accordance with this section. The minimum design seismic forces shall be those determined in accordance with the static lateral force procedure of Section 2312 (e) except as modified by 2312 (f) 5 C.*

2. ***Seismic zone****. The seismic zone factor, Z, for buildings, structures and portions thereof in New York City shall be 0.15. The seismic zone factor is the effective zero period acceleration for* S_1 *type rock.*

3. ***Site geology, soil characteristics and foundations****. A.* ***General.*** *Soil profile type and site coefficient, S, shall be established in accordance with Table No. 23-J.*

B. ***Liquefaction****.*

(i) Soils of classes 7-65, 8-65, 10-65 and non-cohesive class 11-65 below the ground water table and less than fifty feet below the ground surface shall be considered to have potential for liquefaction,

(ii) The potential for liquefaction of level ground shall be determined on the basis of Standard Penetration Resistance (N) in accordance with Figure No. 4;

Category A*. Soil shall be considered liquefiable.*

Category B*. Liquefaction is possible.*

Soil shall be considered liquefiable for structures of Occupancy Categories I, II and III of Table No. 23-K.

Category C*. Liquefaction is unlikely and need not be considered in design. At any site the highest category of liquefaction potential shall apply to the most critical strata or substrata.*

(iii) Liquefiable soils shall be considered to have no passive (lateral) resistance or bearing capacity value during an earthquake. An analysis shall be submitted by an engineer, which demonstrates, subject to the approval of the Commissioner, that the proposed construction is safe against liquefaction effects on the soil.

(iv) Where liquefiable soils are present in sloped ground or over sloped nonliquefiable substrata and where lateral displacement is possible, a stability analysis shall be submitted by an engineer, which demonstrates, subject to the approval of the Commissioner, that the proposed construction is safe against failure of the soil.

C. ***Foundation Plates and Sills****. Foundation plates or sills shall be bolted to the foundation or foundation wall with not less than one-half inch nominal diameter steel bolts embedded at least seven inches into the concrete or masonry and spaced not more than six feet apart. There shall be a minimum of two bolts per piece with one bolt located within twelve inches of each end of each piece. A properly sized nut and washer shall be tightened on each bolt to the plate.*

D. ***Foundation Interconnection of Pile Caps and Caissons****. Individual pile caps and caissons of every structure subjected to seismic forces shall be interconnected by ties. Such ties shall be capable of resisting, in tension or compression, a minimum horizontal force equal to the product of ZI/4 and the larger column vertical load at the end of each tie.*

EXCEPTION*: Other approved effective methods of foundation interconnection may be used where it can be demonstrated by an analysis that equivalent restraint and relative displacement can be provided.*

4. **Occupancy categories**. For purposes of earthquake-resistant design, each structure shall be placed in one of the occupancy categories listed in Table No. 23-K. Table No. 23-L lists importance factors, I, and review requirements for each category.

5. **Configuration requirements.** A. **General**. Each structure shall be designated as being structurally regular or irregular.

B. **Regular structures**. Regular structures have no significant physical discontinuities in plan or vertical configuration or in their lateral force-resisting systems such as the irregular features described below.

C. **Irregular structures**.

(i) Irregular structures have significant physical discontinuities in configuration or in their lateral force-resisting systems. Irregular features include, but are not limited to, those described in Tables Nos. 23-M and 23-N.

(ii) Structures having one or more of the features listed in Table No. 23-M shall be designated as having a vertical irregularity.

EXCEPTION: Where no story drift ratio under design lateral forces is greater than 1.3 times the story drift ratio of the story above the structure may be deemed to not have the structural irregularities of Type A or Type B in Table No. 23-M. The story drift ratio for the top two stories need not be considered. The story drifts for this determination may be calculated neglecting torsional effects.

(iii) Structures having one or more of the features listed in Table No. 23-N shall be designated as having a plan irregularity.

6. **Structural systems.** A. **General**. Structural systems shall be classified as one of the types listed in Table No. 23-0 and defined in this subsection.

B. **Bearing wall system**. A structural system without a complete vertical load-carrying space frame. Bearing walls or bracing systems provide support for all or most gravity loads. Resistance to lateral load is provided by shear walls or braced frames.

C. **Building frame system**. A structural system with an essentially complete space frame providing support for gravity loads. Resistance to lateral load is provided by shear walls or braced frames.

D. **Moment-resisting frame system**. A structural system with an essentially complete space frame provides support for gravity loads. Moment-resisting space frames provide resistance to lateral load primarily by flexural action of members.

E. **Dual system**. A structural system with the following features:

(i) An essentially complete space frame which provides support for gravity loads.

(ii) Resistance to lateral load is provided by shear walls or braced frames and a moment-resisting frame (SMRF, IMRF or OMRF). The moment-resisting frames shall be designed to independently resist at least 25 percent of the design base shear. The shear walls or braced frames shall be designed to resist at least 75 percent of the cumulative story shear at every level. Overturning effects may be distributed in accordance with item (iii) below.

(iii) The two systems shall be designed to resist the total design base shear in proportion to their relative rigidities considering the interaction of the dual system at all levels.

F. **Undefined structural system.** A structural system not listed in Table No. 23-O.

G. **Non-building structural system**. A structural system conforming to Section 2312 (i).

*7. **Height limits**. Delete paragraph.*

*8. **Selection of lateral force procedure.** All structures shall be designed using either the static lateral force procedure of Section 2312 (e) or using the dynamic lateral force procedure of Section 2312 (f). In addition, the dynamic lateral force procedure shall be considered, but is not required, for the design of the following:*

A. Structures over 400 feet in height.

B. Irregular structures.

C. Structures located on Soil Profile Type S_4 which have a period greater than 1 second. The analysis should include the effects of soils at the site and should conform to Section 2312 (f) 2.

9. **System limitations**. A. **Discontinuity**. Structures with a discontinuity in capacity, vertical irregularity Type E as defined in Table No. 23-M, shall not be over two stories or 30 feet in height where the weak story has a calculated strength of less than 65 percent of the story above.

EXCEPTION: Where the weak story is capable of resisting a total lateral seismic force of 3 (R_w/8) times the design force prescribed in Section 2312 (e).

B. **Undefined structural systems**. Undefined structural systems shall be shown by technical and test data which establish the dynamic characteristics and demonstrate the lateral force resistance and energy absorption capacity to be equivalent to systems listed in Table No. 23-O for equivalent R_w values.

*C. **Irregular features**. Only structures having either vertical irregularities Type D or E as defined in Table 23-M or horizontal irregularities Type D or E as defined in Table 23-N shall be designed to meet the additional requirements of those sections referenced in the tables.*

10. **Alternate procedures.** Alternative lateral force procedures using rational analyses based on well established principles of mechanics may be used in lieu of those prescribed *when such procedures are consistent with this standard and subject to the approval of the Commissioner.*

(e) **Minimum Design Lateral Forces and Related Effects**. 1. **General**. Structures shall be designed for seismic forces coming from any horizontal direction.

The design seismic forces may be assumed to act noncurrently in the direction of each principal axis of the structure, except as required by Section 2312 (h) 1.

Seismic dead load, *W*, is the total dead load and applicable portions of other loads listed below.

A. In *parking structures*, storage and warehouse occupancies, a minimum of 25 percent of the floor live load shall be applicable.

B. Where a partition load is used in the floor design, a load of not less than 10 psf shall be included.

C. Total weight of permanent equipment shall be included.

2. **Static force procedure.** A. **Design base shear**. The total design base shear in a given direction shall be determined from the following formula:

$$V = \frac{ZIC}{R_w} W \qquad (12\text{-}1)$$

$$C = \frac{1.25S}{T^{2/3}} \qquad (12\text{-}2)$$

The value of C need not exceed 2.75 and may be used for any structure without regard to soil type or structure period.

Except for those provisions where code prescribed forces are scaled up by 3 (R_w/8) the minimum value of the ratio C/R_w shall be *0.050*.

B. **Structure period**. The value of T shall be determined from one of the following methods:

(i) **METHOD A:** For all buildings, the value T may be approximated from the following formula:

$$T = C_t(h_n)^{3/4} \qquad (12\text{-}3)$$

WHERE:

C_t = *0.035 for concrete and steel moment-resisting frames.*

C_t = *0.030 for eccentric braced frames.*

C_t = *0.030 for dual systems where the building height exceeds 400 feet or 0.020 for heights less than 160 feet and varies linearly from 0.020 to 0.030 for building heights from 160 to 400 feet.*

C_t = *0.020 for all other structures.*

Alternatively, the value of T for structures with concrete or masonry shear walls may be taken as $0.1(h_n)^{3/4}/\sqrt{A_c}$.

The value of A_c shall be determined from the following formula:

$$A_c = \Sigma A_e[0.2 + (D_e/h_n)^2] \qquad (12\text{-}4)$$

The value of D_e/h_n used in formula (12-4) shall not exceed 0.9.

(ii) **METHOD B:** The fundamental period T may be calculated using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. This requirement may be satisfied by using the following formula:

$$T = 2\pi\sqrt{\left(\sum_{i=1}^{n} w_i\delta_i^2\right) \div \left(g\sum_{i=1}^{n} f_i\delta_i\right)} \qquad (12\text{-}5)$$

The values of f_i represent any lateral force distributed approximately in accordance with the principles of Formulas (12-6), (12-7) and (12-8) or any other rational distribution. The elastic deflections, δ_i, shall be calculated using the applied lateral forces, f_i. The value of C shall be not less than 80 percent of the value obtained by using T from Method A.

3. **Combinations of structural systems**. A. **General**. Where combinations of structural systems are incorporated into the same structure, the requirements of this subsection shall be satisfied.

B. **Vertical combinations**. The value of R_w used in the design of any story shall be less than or equal to the value of R_w used in the given direction for the story above.

EXCEPTION: This requirement need not be applied to a story where the dead weight above that story is less than 10 percent of the total dead weight of the structure.

Structures may be designed using the procedures of this section under the following conditions:

(i) The entire structure is designed using the lowest R_w of the lateral force-resisting systems used.

(ii) The following two-stage static analysis procedures may be used for structures conforming to Section 2312 (d) 8, Item B (iv).

(a) The flexible upper portion shall be designed as a separate structure, supported laterally by the rigid lower portion, using the appropriate value of R_w.

(b) The rigid lower portion shall be designed as a separate structure using the appropriate value of R_w. The reactions from the upper portion shall be those determined from the analysis of the upper portion amplified by the ratio of the R_w of the upper portion over the R_w of the lower portion.

*C. **Combinations along different axes**. Delete this paragraph.*

4. **Vertical distribution of force**. The total force shall be distributed over the height of the structure in conformance with Formulas (12-6), (12-7) and (12-8) in the absence of a more rigorous procedure.

$$V = F_t + \sum_{i=1}^{n} F_i \qquad (12\text{-}6)$$

The concentrated force F_t, at the top, which is in addition to F_n, shall be determined from the formula:

$$F_t = 0.07TV \qquad (12\text{-}7)$$

The value of T used for the purpose of calculating F_t may be the period that corresponds with the design base shear as computed using Formula (12-1). F_t need not exceed 0.25V and may be considered as zero (0) where T is 0.7 seconds or less. The remaining portion of the base shear shall be distributed over the height of the structure, including Level n, according to the following formula:

$$F_x = \frac{(V - F_t) w_x h_x}{\sum_{i=1}^{n} w_i h_i} \qquad (12\text{-}8)$$

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 at that level. Stresses in each structural element shall be calculated as the effect of forces F_x and F_t applied at the appropriate levels above the base.

5. **Horizontal distribution of shear.** The design story shear, V_x, in any story is the sum of the forces F_t and F_x above that story. V_x shall be distributed to the various elements of the vertical lateral force-resisting system in proportion to their rigidities, considering the rigidity of the diaphragm. See Section 2312 (h) 2D for rigid elements that are not intended to be part of the lateral force-resisting systems.

To account for the uncertainties in locations of loads, the mass at each level shall be assumed to be displaced from the calculated center of mass in each direction a distance equal to five percent of the building dimension at that level perpendicular to the direction of the force under consideration. The effect of this displacement on the story shear distribution shall be considered.

6. **Horizontal torsional moments**. Provision shall be made for the increased shears resulting from horizontal torsion where diaphragms are not flexible. Diaphragms shall be considered flexible for purposes of this paragraph when the maximum lateral deformation of the diaphragm is more than two times the average story drift of the associated story. This may be determined by comparing the computed midpoint in-plane deflection of the diaphragm under lateral load with the story drift of adjoining vertical resisting elements under equivalent tributary lateral load.

The torsional design moment at a given story shall be the moment resulting from eccentricities between applied design lateral forces at levels above that story and the vertical resisting elements in that story plus an accidental torsion.

The accidental torsional moment shall be determined by assuming the mass is displaced as required by Section 2312 (e) 5.

7. **Overturning**. A. Every structure shall be designed to resist the overturning effects caused by earthquake forces specified in Section 2312 (e) 4. At any level, the overturning moments to be resisted shall be determined using those seismic forces (F_t and F_x) which act on levels above the level under consideration. At any level. the incremental changes of the design overturning moment shall be distributed to the various resisting elements in the manner prescribed in Section 2312 (e) 5. Overturning effects on every element shall be carried down to the foundation. See Section 2312 (h) for combining gravity and seismic forces.

B. Where a lateral load-resisting element is discontinuous, such as for vertical irregularity Type D in Table No. 23-M or plan irregularity Type D in Table No. 23-N, columns supporting such elements shall have the strength to resist the axial force resulting from the following load combinations, in addition to all other applicable load combinations:

$$1.0\ DL + 0.8\ LL + 3\ (R_w/8)E$$

$$0.85\ DL + 3\ (R_w/8)E$$

(i) The axial forces in such columns need not exceed the capacity of other elements of the structure to transfer such loads to the column.

(ii) Such columns shall be capable of carrying the above-described axial forces without exceeding the axial load strength of the column. For designs using working stress methods this capacity may be determined using an allowable stress increase of 1.7

(iii) Such columns shall meet the detailing or member limitations of reference standard RS 10-3 for concrete and reference standard RS 10-5C for steel structures.

C. For regular buildings, the force F_t may be omitted when determining the overturning moment to be resisted at the foundation-soil interface.

8. **Story drift limitation**. Story drift is the displacement of one level relative to the level above or below due to the design lateral forces. Calculated drift shall include translational and torsional deflections.

Calculated story drift shall not exceed $0.04/R_w$ nor 0.005 times the story height for structures having a fundamental period of less than 0.7 seconds. For structures having a fundamental period of 0.7 seconds or greater, the calculated story drift shall not exceed $0.03/R_w$ nor 0.004 times the story height.

These drift limits may be exceeded when it is demonstrated that greater drift can be tolerated by both structural elements and nonstructural elements that could affect life safety.

The design lateral forces used to determine the calculated drift may be derived from a value of C based on the period determined from Formula (12-5) neglecting the lower bound ratio for C/R_w of *0.050* of Section 2312 (e)2A and the 80 percent limitation of Section 2312 (e)2B (ii).

*9. **P-delta effects**. The resulting member forces and moments and the story drifts induced by P-delta effects shall be considered in the evaluation of overall structural frame stability. P-delta need not be considered when the ratio of secondary moment to primary moment does not exceed 0.10; the ratio may be evaluated for any story as the product of the total dead and live load above the story times the seismic drift in that story divided by the product of the seismic shear in that story times the height of that story.*

*10. **Vertical component of seismic forces**. Horizontal cantilever components shall be designed for a net upward force of $0.05\ W_p$.*

In addition to all other applicable load combinations horizontal prestressed components shall be designed using not more than 50 percent of the dead load for the gravity load, alone or in combination with the lateral force effects.

(f) **Dynamic lateral force procedure**. 1. **General**. Dynamic analyses procedures, when used, shall conform to the criteria established in this section. The analysis shall be based on an appropriate ground motion representation and shall be performed using accepted principles of dynamics. Structures which are designed in accordance with this section shall comply with all other applicable requirements of these provisions.

2. **Ground motion**. The ground motion representation shall, as a minimum, be one having a 10 percent probability of exceedance in 50 years and may be one of the following:

A. For soil profile types S_1, S_2 and S_3, the normalized response spectrum is given in Figure No. 3. *For soil profile type S_4, see B. below*.

B. A site-specific response spectrum based on the geologic, tectonic, seismologic and soil characteristics associated with the specific site. The spectrum shall be developed for a damping ratio of 0.05 unless a different value is shown to be consistent with the anticipated structural behavior at the intensity of shaking established for the site. *The design of all structures located on a soil type S_4 profile shall be based on properly substantiated site-specific spectra*.

C. Ground motion time histories developed for the specific site shall be representative of actual earthquake motions. Response spectra from time histories, either individually or in combination, shall approximate the site design spectrum conforming to Section 2312 (f) 2 B.

D. For structures on Soil Profile Type S_4 the following requirements shall apply when required by Section 2312 (d) 8 C (iv):

(i) The ground motion representation shall be developed in accordance with paragraphs B and C above.
(ii) Possible amplification of building response due to the effects of soil-structure interaction and lengthening of building period caused by inelastic behavior shall be considered.
(iii) The base shear determined by these procedures may be reduced to a design base shear, V, by dividing by a factor not greater than the appropriate R_w factor for the structure.

E. The vertical component of ground motion may be defined by scaling corresponding horizontal accelerations by a factor of two thirds. Alternative factors may be used when substantiated by site-specific data.

3. **Mathematical model**. A mathematical model of the physical structure shall represent the spatial distribution of the mass stiffness of the structure to an extent which is adequate for the calculation of the significant features of its dynamic response. A three-dimensional model shall be used for the dynamic analysis of structures with highly irregular plan configurations such as those having a plan irregularity defined in Table No. 23-N and having a rigid or semirigid diaphragm.

4. **Description of analysis procedures.** A. **Response spectrum analysis.** An elastic dynamic analysis of a structure utilizing the peak dynamic response of all modes having a significant contribution to total structural response. Peak modal responses are calculated using the ordinates of the appropriate response spectrum curve which correspond to the modal periods. Maximum modal contributions are combined in a statistical manner to obtain an approximate total structural response.

B. **Time history analysis**. An analysis of the dynamic response of a structure at each increment of time when the base is subjected to a specific ground motion time history.

5. **Response spectrum analysis.** A. **Number of modes**. The requirement of Section 2312(f)4A that all significant modes be included may be satisfied by demonstrating that for the modes considered at least 90 percent of the participating mass of the structure is included in the calculation of response for each principal horizontal direction.

B. **Combining modes.** The peak member forces, displacements, story forces, story shears and base reactions for each mode shall be combined by recognized methods. When three-dimensional models are used for analysis, modal interaction effects shall be considered when combining modal maxima.

C. **Scaling of results**. The base shear for a given direction determined using these procedures, *including the appropriate Importance Factor, I,* when less than the values below, shall be scaled up to these values.

(i) The base shear shall be increased to the following percentage of the value determined from the procedures of Section 2312(e), including consideration of the minimum value of C/R_w, except that the coefficient C, for a period, T, greater than 3 seconds, may be calculated as 1.80 S/T:

(a) 100 percent for irregular buildings; or

(b) 90 percent for regular buildings, except that the base shear shall not be less than 80 percent of that determined from Section 2312(e) using the period, T, calculated from Method A.

All corresponding response parameters, including deflections, member forces and moments, shall be increased proportionately.

(ii) The base shear for a given direction determined using these procedures need not exceed that required by paragraph (i) above. All corresponding response parameters may be adjusted proportionately.

*D. **Directional effects**. Directional effects for horizontal ground motion shall conform to the requirements of Section 2312 (e) 1. The effects of vertical ground motions on horizontal cantilevers shall be considered in accordance with Section 2312 (e) 10. Vertical seismic response may be determined by dynamic response methods; in no case shall the response used for design be less than that obtained by the static method.*

E. **Torsion.** The analysis shall account for torsional effects, including accidental torsional effects as prescribed in Section 2312 (e) 6. Where three-dimensional models are used for analysis, effects of accidental torsion shall be accounted for by appropriate adjustment in the model such as adjustment of mass locations, or by equivalent static procedures such as provided in Section 2312 (e) 6.

*F. **Dual systems.** Where the lateral forces are resisted by a dual system, as defined in Section 2312(d)6E above, the combined system shall be capable of resisting the base shear determined in accordance with this section. The moment-resisting frame, shear walls and braced frames shall conform to Section 2312(d)6E. The moment-resisting frame may be analyzed using either the procedures of Section 2312 (e)4 or those of Section 2312 (f) 5.*

6. **Time history analysis.** Time history analyses shall meet the requirements of Section 2312(d)10 *and the results shall be scaled in accordance with Section 2312 (f) 5C.*

(g) **Lateral Force on Elements of Structures and Nonstructural Components Supported by Structures**.

1. **General**. Parts and portions of structures and their attachments, permanent nonstructural components and their attachments, and the attachments for permanent equipment supported by a structure shall be designed to resist the total design seismic forces prescribed in Section 2312 (g) 2.

Attachments shall include anchorages and required bracing. Friction resulting from gravity loads shall not be considered to provide resistance to seismic forces.

When the structural failure of the lateral force-resisting systems of nonrigid equipment would cause a life hazard, such systems shall be designed to resist the seismic forces prescribed in Section 2312 (g) 2.

EXCEPTION: Equipment weighing less than 400 pounds, furniture, or temporary or movable equipment.

When allowable design stresses and other acceptance criteria are not contained in or referenced by this code in the U.B.C. Standards, such criteria shall be obtained from approved national standards.

2. Design for total lateral force. The total design lateral seismic force, F_p, shall be determined from the following formula:

$$F_p = ZIC_pW_p \qquad (12\text{-}10)$$

The values of Z and I shall be the values used for the building.

EXCEPTIONS: 1. For anchorage of machinery and equipment required for life-safety systems, the value of I shall be taken as 1.5.

2. For the design of tanks and vessels containing sufficient quantities of highly toxic or explosive substances to be hazardous to the safety of the general public if released, the value of I shall be taken as 1.5.

3. The value of I for panel connectors for panels in Section 2312 (h) 2 D (iii) shall be 1.0 for the entire connector.

The coefficient C_p is for elements and components and for rigid and rigidly supported equipment. Rigid or rigidly supported equipment is defined as having a fundamental period less than or equal to 0.06 second. Nonrigid or flexibly supported equipment is defined as a system having a fundamental period, including the equipment, greater than 0.06 second.

The lateral forces calculated for nonrigid or flexibly supported equipment supported by a structure and located above grade shall be determined considering the dynamic properties of both the equipment and the structure which supports it, but the value shall not be less than that listed in Table No. 23-P. In the absence of an analysis or empirical data, the value of C_p for nonrigid or flexibly supported equipment located above grade on a structure shall be taken as twice the value listed in Table No. 23-P, but need not exceed 2.0.

EXCEPTION: Piping, ducting and conduit systems which are constructed of ductile materials and connections may use the values of C_p from Table No. 23-P.

The value of C_p for elements, components and equipment laterally self-supported at or below ground level may be two thirds of the value set forth in Table No. 23-P. However, the design lateral forces for an element or component or piece of equipment shall not be less than would be obtained by treating the item as an independent structure and using the provisions of Section 2312 (i).

The design lateral forces determined using Formula (12-10) shall be distributed in proportion to the mass distribution of the element or component.

Forces determined using Formula (12-10) shall be used to design members and connections which transfer these forces to the seismic-resisting systems.

For applicable forces in connectors for exterior panels and diaphragms, refer to Section 2312 (h) 2 D and I.

Forces shall be applied in the horizontal directions, which result in the most critical loadings for design.

3. **Specifying lateral forces.** Design specifications for equipment shall either specify the design lateral forces prescribed herein or reference these provisions.

4. **Essential or hazardous facilities and life-safety systems**. For equipment in facilities assigned to Occupancy Categories I and II and for life-safety systems, the design and detailing of equipment which needs to be functional following a major earthquake shall consider the effect of drift.

5. **Alternative designs**. Where an approved national standard or approved physical test data provide a basis for the earthquake-resistant design of a particular type of equipment or other nonstructural component, such a standard or data may be accepted as a basis for design of the items with the following limitations:

(i) These provisions shall provide minimum values for the design of the anchorage and the members and connections which transfer the forces to the seismic-resisting system.

(ii) The force, F_p, and the overturning moment used in the design of the nonstructural component shall not be less than 80 percent of the values that would be obtained using these provisions.

(h) **Detailed Systems Design Requirements**. 1. **General**. All structural framing systems shall comply with the requirements of Section 2312 (d). Only the elements of the designated seismic-force-resisting system shall be used to resist design forces. The individual components shall be designed to resist the prescribed design seismic forces acting on them. The components shall also comply with the specific requirements for the material contained in r*eference standard RS 10*. In addition, such framing systems and components shall comply with the detailed system design requirements contained in Section 2312 (h).

All building components shall be designed to resist the effects of the seismic forces prescribed herein and the effects of gravity loadings from dead, floor, live and snow loads.

Consideration shall be given to design for uplift effects caused by seismic loads. For materials which use working stress procedures, dead loads shall be multiplied by 0.85 when used to reduce uplift.

Provision shall be made for the effects of earthquake forces acting in a direction other than the principal axes in each of the following circumstances:

The structure has plan irregularity Type E as given in Table No. 23-N.

The structure has plan irregularity Type A as given in Table No. 23-N for both major axes.

A column of a structure forms part of two or more intersecting lateral force-resisting systems.

EXCEPTION: If the axial load in the column due to seismic forces acting in either direction is less than 20 percent of the column allowable axial load.

The requirement that orthogonal effects be considered may be satisfied by designing such elements for 100 percent of the prescribed seismic forces in one direction plus 30 percent of the prescribed forces in the perpendicular direction. The combination requiring the greater component strength shall be used for design. Alternatively, the effects of the two orthogonal directions may be combined on a square root of the sum of the squares (SRSS) basis. When the SRSS method of combining directional effects is used, each term computed shall be assigned the sign that will result in the most conservative result.

2. **Structural framing systems.** A. **General.** Four types of general building framing systems defined in Section 2312 (d) 6 are recognized in these provisions and shown in Table No. 23-O. Each type is subdivided by the types of vertical elements used to resist lateral seismic forces. Special framing requirements are given in this section and in *reference standard RS 10*.

B. **Detailing requirements for combinations of systems**. For components common to different structural systems, the more restrictive detailing requirements shall be used.

*C. **Connections**. Delete this paragraph.*

D. **Deformation compatibility.**

(i) All framing elements not required by design to be part of the lateral force-resisting system shall be investigated and shown to be adequate for vertical load-carrying capacity when displaced 3 (R_w/8) times the displacements resulting from the required lateral forces. P-delta effects on such elements shall be accounted for. For designs using working stress methods, this capacity may be determined using an allowable stress increase of 1.7. The rigidity of other elements shall also be considered.

(ii) Moment-resistant space frames may be enclosed by or adjoined by more rigid elements which would tend to prevent the space frame from resisting lateral forces where it can be shown that the action or failure of the more rigid elements will not impair the vertical and lateral load resisting ability of the space frame.

(iii) Exterior nonbearing, nonshear wall panels or elements which are attached to or enclose the exterior shall be designed to resist the forces per Formula (12-10) and shall accommodate movements of the structure resulting from lateral forces or temperature changes. Such elements shall be supported by means of cast-in-place concrete or by 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 two times story drift caused by wind, $3(R_w/8)$ times the calculated elastic story drift caused by design seismic forces, or 1/2 inch, whichever is greater.

Connections to permit movement in the plane of the panel for story drift shall be sliding connections using slotted or oversize holes, connections which permit movement by bending of steel, or other connections providing equivalent sliding and ductility capacity.

Bodies of connections 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 connection shall be designed for one and one-third times the force determined by Formula (12-10).

All elements of the connecting system such as bolts, inserts, welds, dowels, etc., shall be designed for four times the forces determined by Formula (12-10). *Fasteners embedded in concrete shall be attached to, or hooked around, reinforcing steel or otherwise terminated so as to effectively transfer the forces.*

E. **Ties and continuity**. All parts of a structure shall be interconnected and the connections shall be capable of transmitting the seismic force, induced by the parts being connected. As a minimum, any smaller portion of the building shall be tied to the remainder of the building with elements having at least a strength to resist Z/3 times the weight of the smaller portion.

A positive connection for resisting a horizontal force acting parallel to the member shall be provided for each beam, girder or truss. This force shall be not less than Z/5 times the dead plus live load.

F. **Collector elements**. Collector elements shall be provided which are capable of transferring the seismic forces originating in other portions of the building to the element providing the resistance to those forces.

*G. **Concrete frames**. Concrete frames required by design to be part of the lateral force resisting system shall, as a minimum, be intermediate moment-resisting frames, except as noted in Table 23-O.*

H. **Anchorage of concrete or masonry walls**. Concrete or masonry walls shall be anchored to all floors and roofs which provide lateral support for the wall. The anchorage shall provide a positive direct connection between the wall and floor or roof construction capable of resisting the horizontal forces specified in Section 2312 (g) or r*eference standards RS 9-6, 10-1 and 10-2*. Requirements for developing anchorage forces in diaphragms are given in Section 2312 (h) 2 I below. Diaphragm deformation shall be considered in the design of the supported walls.

I. **Diaphragms**.

(i) The deflection in the plane of the diaphragm shall not exceed the permissible deflection of the attached elements. Permissible deflection shall be that deflection which will permit the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads.

(ii) Floor and roof diaphragms shall be designed to resist the forces determined in accordance with the following formula:

$$F_{px} = \frac{\sum_{l=x}^{n} F_l}{\sum_{l=x}^{n} w_t} w_{px} \qquad (12\text{-}11)$$

The force F_{px} determined from Formula (12-11) need not exceed 0.75 Z I w_{px}, but shall not be less than 0.35 Z I w_{px}.

When the diaphragm is required to transfer lateral forces from the vertical resisting elements above the diaphragm to other vertical elements below the diaphragm due to offset in the placement of the elements or to changes in stiffness in the vertical elements, these forces shall be added to those determined from Formula (12-11).

(iii) Diaphragms supporting concrete or masonry walls shall have continuous ties or struts between diaphragm chords to distribute the anchorage forces specified in Section 2312 (h) 2 H. Added chords may be used to form subdiaphragms to transmit the anchorage forces to the main crossties.

(iv), (v) and (vi): Delete these items.

J. **Framing below the base.** The strength and stiffness of the framing between the base and the foundation shall not be less than that of the superstructure. The special detailing requirements of *reference standards RS 10-3 and RS 10-5C*, as appropriate, shall apply to columns supporting discontinuous lateral force-resisting elements and to SMRSF, IMRSF and EBF system elements below the base which are required to transmit the forces resulting from lateral loads to the foundation.

K. Building Separations. *All structures shall be separated from adjoining structures. Separation due to seismic forces shall allow for 1 inch displacement for each 50 feet of total building height. Smaller separation may be permitted when the effects of pounding can be accommodated without collapse of the building.*

(i) **Non-building Structures.** 1. **General**. A. Non-building structures include all self-supporting structures other than buildings which carry gravity loads and resist the effects of earthquake. Non-building structures shall be designed to resist the minimum lateral forces specified in this section. Design shall conform to the applicable provisions of other sections as modified by the provisions contained in Section 2312 (i).

B. The minimum design lateral forces prescribed in this section are at a service level (rather than yield or ultimate level). The design of Non-building structures shall provide sufficient strength and ductility, consistent with the provisions specified herein for buildings, to resist the effects of seismic ground motions as represented by these design forces.

When applicable, allowable stresses and other design criteria shall be obtained from approved national standards.

When applicable design stresses and other design criteria are not contained in or referenced by this code or the U.B.C. Standards, such criteria shall be obtained from approved national standards.

C. The weight W for Non-building structures shall include all dead load as defined for buildings in Section 2312 (e) 1. For purposes of calculating design seismic forces in Non-building structures, W shall also include all normal operating contents for items such as tanks, vessels, bins and piping.

D. The fundamental period of the structure shall be determined by rational methods such as by using Method B in Section 2312 (e) 2.

E. The drift limitations of Section 2312 (e) 8 need not apply to Non-building structures. Drift limitations shall be established for structural or nonstructural elements whose failure would cause life hazards. P-delta effects shall be considered for structures whose calculated drifts exceed the values in Section 2312 (e) 8.

F. In Seismic Zones 3 and 4, structures which support flexible nonstructural elements whose combined weight exceeds 25 percent of the weight of the structure shall be designed considering interaction effects between the structure and the supported elements.

2. **Lateral force.** Lateral force procedures for Non-building structures with structural systems similar to buildings [those with structural systems which are listed in Table No. 23(O).] shall be selected in accordance with the provisions of Section 2312 (d).

EXCEPTION: Intermediate moment-resisting space frames (IMRSF) may be used in Zones Nos. 3 and 4 for Non-building structures in Occupancy Categories III and IV if (1) the structure is less than 50 feet in height and (2) an $R_w = 4.0$ is used for design.

Rigid structures (those with period T less than 0.06 second), including their anchorages, shall be designed for the lateral force obtained from Formula (12-12).

$$V = 0.5\ Z\ I\ W \qquad (12\text{-}12)$$

The force V shall be distributed according to the distribution of mass and shall be assumed to act in any horizontal direction.

3. **Tanks with supported bottoms.** Flat bottom tanks or other tanks with supported bottoms, founded at or below grade, shall be designed to resist the seismic forces calculated using the procedures in Section 2312 (i) for rigid structures considering the entire weight of the tank and its contents. Alternatively, such tanks may be designed using one of the two procedures described below.

A response spectrum analysis, which includes consideration of the actual ground motion anticipated at the site and the inertial effects of the contained fluid.

A design basis prescribed for the particular type of tank by an approved national standard, provided that the seismic zones and occupancy categories shall be in conformance with the provisions of Sections 2312 (d) 2 and 2312 (d) 4, respectively.

4. **Other Non-building structures.** Non-building structures which are not covered by Section 2312 (i) 2 and 3 shall be designed to resist minimum seismic lateral forces not less than those determined in accordance with the provisions in Section 2312 (e) with the following additions and exceptions:

(i) The factor R_w shall be as given in Table No. 23-Q. The ratio C/R_w used for design shall be not less than 0.5.

(ii) The vertical distribution of the lateral seismic forces in structures covered by this section may be determined by using the provisions of Section 2312 (e) 4 or by using the procedures of Section 2312 (f).

EXCEPTION: For irregular structures assigned to Occupancy Categories I and II which cannot be modeled as a single mass the procedures of Section 2312 (f) shall be used.

(iii) Where an approved *reference* standard provides a basis for the earthquake-resistant design of a particular type of Non-building structure covered by this Section 2312(i)4, such a standard may be used, subject to the limitations in this subsection:

The occupancy categories shall be in conformance with the provisions of Sections 2312(d)5 and 2312(d)4, respectively.

The values for total lateral force and total base overturning moment used in design shall not be less than 80 percent of the values that would be obtained using these provisions.

*(j) **Earthquake-recording Instrumentations**. Delete this paragraph.*

Table No. 23-I
Seismic Zone Factor Z

ZONE	*NEW YORK CITY*
Z	*0.15*

Table No. 23-J
Site Coefficients

TYPE	*DESCRIPTIONS*	*FACTOR*
S_o	*A profile of Rock materials of class I-65 to 3-65.*	*0.67*
S_1	*A soil profile with either: (a) Soft Rock (4- 65) or Hardpan (5-65) or similar material characterized by shear-wave velocity greater than 2500 feet per second, or (b) Medium Compact to Compact Sands (7-65) and Gravels (6-65) or Hard Clays (9-65) where the soil depth is less than 100 feet.*	*1.0*
S_2	*A soil profile with Medium Compact to Compact Sands (7-65) and Gravels (6-65) or Hard Clays (9-65), where the soil depth exceeds 100 feet.*	*1.2*
S_3	*A total depth of overburden of 75 feet or more and containing more than 20 feet of Soft to Medium Clays (9-65) or Loose Sands (7-65, 8-65) and Silts (10-65), but not more than 40 feet of Soft Clay or Loose Sands and Silts.*	*1.5*
S_4	*A soil profile containing more than 40 feet of Soft Clays (9-65) or Loose Sands (7-65, 8-65), Silts (10-65) or Uncontrolled Fills (11-65), where the sheer-wave velocity is less than 500 feet per second.*	*2.5*

Notes:

1. The site S Type and corresponding S Factor shall be established from properly substantiated geotechnical data with the classes of materials being defined in accordance with Section 27-675 (C26-1103.1) of the administrative code of the city of New York.

2. The soil profile considered in determining the S Type shall be the soil on which the structure foundations bear or in which pile caps are embedded and all underlying soil materials.

3. Soil density/consistency referred to in the table should be based on standard penetration test blow counts (N-values) and taken as: (a) for sands, loose - where N is less than 10 blows per foot, medium compact - where N is between 10 and 30, and compact - where N is greater than 30 blows per foot; and (b) for clays, soft - where N is less than 4 blows per foot, medium - where N is between 4 and 8, stiff to very stiff - where N is between 8 and 30, and hard - where N is greater than 30 blows per foot.

4. When determining the type of soil profile for profile descriptions that fall somewhere in between those provided in the above table, the S Type with the larger S factor shall be used.

5. For Loose Sands, Silts or Uncontrolled Fills below the ground water table, the potential for liquefaction shall be evaluated by the provisions of Section 2312(d) 3.

Table No. 23-K
Occupancy Categories

OCCUPANCY CATEGORIES	OCCUPANCY TYPE OR FUNCTION OF STRUCTURE
I. Essential Facilities	Hospitals and other medical facilities having surgery and emergency treatment areas. Fire and Police stations. *Buildings for schools through secondary or day-care centers - capacity > 250 students.* Tanks or other structures containing, housing or supporting water or other fire-suppression materials or equipment required for the protection of essential or hazardous facilities, or special occupancy structures. Emergency vehicle shelters and garages. Structures and equipment in emergency-preparedness centers. Stand-by power generating equipment for essential facilities. Structures and equipment in government communication centers and other facilities required for emergency response.
II. Hazardous Facilities	Structures housing, supporting or containing sufficient quantities of toxic or explosive substances to be dangerous to the safety of the general public if released.
III. Special Occupancy Structure	Covered structures whose primary occupancy is public assembly - capacity > 300 persons. Buildings for colleges or adult education schools - capacity > 500 students. Medical facilities with 50 or more resident incapacitated patients, but not included above. Jails and detention facilities. All structures with occupancy > 5000 persons, *excluding Occupancy Group E buildings*. Structures and equipment in power generating stations and other public utility facilities not included above, and required for continued operation.
IV. Standard Occupancy Structure	All structures having occupancies or functions not listed above.

Table No. 23-L
Occupancy Requirements

OCCUPANCY CATEGORY[1]	IMPORTANCE FACTOR[2] *I*
I. Essential facilities	1.25
II. Hazardous facilities	1.25
III. Special occupancy structures	1.0
IV. Standard occupancy structures	1.0

[1]Occupancy types or functions of structures within each category are listed in Table No. 23-K. Review and inspection requirements are given in Sections 305 and 306.

[2]For life-safety-related equipment, see Sections 2312 (g) I.

Table No. 23-M
Vertical Structural Irregularities

IRREGULARITY TYPE AND DEFINITION	REFERENCE SECTION
A. Stiffness Irregularity - Soft Story A soft story is one in which the lateral stiffness is less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above.	2312 (d) 8 C (ii)
B. Weight (mass) Irregularity Mass irregularity shall be considered to exist where the effective mass of any story is more than 150 percent of the effective mass of an adjacent story. A roof which is lighter than the floor below need not be considered.	2312 (d) 8 C (ii)
C. Vertical Geometric Irregularity Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral force-resisting system in any story is more than 130 percent of that in an adjacent story. One-story penthouses need not be considered.	2312 (d) 8 C (ii)
D. In-Plane Discontinuity in Vertical Lateral Force-resisting Element An in-plane offset of the lateral load-resisting elements greater than the length of those elements.	2312 (e) 7
E. Discontinuity in Capacity - Weak Story A weak story is one in which the story strength is less than 80 percent of that in the story above. The story strength is the total strength of all seismic resisting elements sharing the story shear for the direction under consideration.	2312 (d) 9 A

Table No. 23-N
Plan Structural Irregularities

IRREGULARITY TYPE AND DEFINITION	REFERENCE SECTION
A. Torsional Irregularity - to be considered when diaphragms are not flexible.	
Torsional irregularity shall be considered to exist when the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts of the two ends of the structure.	2312 (h) 2 I (v)
B. Reentrant Corners	
Plan configurations of a structure and its lateral force-resisting system contain reentrant corners, where both projections of the structure beyond a reentrant corner are greater than 15 percent of the plan dimension of the structure in the given direction.	2312 (h) 2 I (v) 2312 (h) 2 I (vi)
C. Diaphragm Discontinuity	
Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50 percent of the gross enclosed area of the diaphragm, or changes in effective diaphragm stiffness of more than 50 percent from one story to the next.	2312 (h) 2 I (v)
D. Out-of-plane Offsets	
Discontinuities in a lateral force path, such as out-of-plane offsets of the vertical elements.	2312 (e) 7, 2312(h) 2 I (v)
E. Nonparallel Systems	
The vertical lateral load-resisting elements are not parallel to nor symmetric about the major orthogonal axes of the lateral force-resisting system.	2312 (h) I

Table No. 23-O
Structural Systems

BASIC STRUCTURAL SYSTEM	LATERAL LOAD-RESISTING SYSTEM DESCRIPTION	R_W
A. Bearing Wall System	*1. Light-framed walls with shear panels*	
	a. Plywood walls for structures three stories or less	*8*
	b. All other light-framed walls	*6*
	2. Shear Walls	
	a. Concrete	*6*
	b. Reinforced masonry	*5*
	3. Light steel-framed bearing walls with tension-only bracing	*4*
	4. Braced frames where bracing carries gravity load	
	a. Steel	*6*
	b. Concrete	*4*
	c. Heavy timber	*4*
B. Building Frame System	*1. Steel eccentric braced frame (EBF)*	*10*
	2. Light-framed walls with shear panels	
	a. Plywood walls for structures three stories or less	*9*
	b. All other light-framed walls	*7*
	3. Shear walls	
	a. Concrete	*8*
	b. Reinforced masonry	*6*
	4. Concentric braced frames	
	a. Steel	*8*
	b. Concrete	*8*
	c. Heavy timber	*8*
C. Moment-resisting Frame System	*1. Special moment-resisting frames (SMRF)*	
	a. Steel	*12*
	b. Concrete	*12*
	2. Concrete intermediate moment-resisting frames (IMRF)	*8*
	3. Ordinary moment-resisting frames (OMRF)	
	a. Steel	*6*
	b. Concrete[4]	*4*
D. Dual Systems	*1. Shear walls*	
	a. Concrete with SMRF	*12*
	b. Concrete with steel OMRF	*6*
	c. Concrete with concrete IMRF	*9*
	d. Concrete with concrete OMRF	*5*
	e. Reinforced masonry with SMRF	*8*
	f. Reinforced masonry with steel OMRF	*6*
	g. Reinforced masonry with concrete IMRF	*7*
	2. Steel eccentric braced frame	
	a. With steel SMRF	*12*
	b. With steel OMRF	*6*
	3. Concentric braced frames	
	a. Steel with steel SMRF	*10*
	b. Steel with steel OMRF	*6*
	c. Concrete with concrete SMRF	*9*
	d. Concrete with concrete IMRF	*6*

Notes:
1. Basic structural systems are defined in Section 2312 (d) 6.
2. See Section 2312 (e) 3 for combinations of structural systems.
3. See Section 2312 (d) 8C and 2312 (d) 9B for undefined systems.
4. Prohibited with S_3 or S_4 soil profiles or where the height exceeds 160 feet.

Table No. 23-P
Horizontal Force Factor $\boldsymbol{C_p}$ [1]

ELEMENTS OF STRUCTURES, NONSTRUCTURAL COMPONENTS AND EQUIPMENT	*VALUE OF* C_p
I. Part or Portion of Structure	
1. Walls, including the following:	
a. Unbraced (cantilevered) parapets.	*2.00*
b. Other exterior walls above street grade.[2]	*0.75*
c. All interior bearings walls.	*0.75*
d. All interior nonbearing walls and partitions around vertical exits, including offsets and exit passageways.	*0.75*
e. Nonbearing partitions and masonry walls in areas of public assembly > 300 people.	*0.75*
f. All interior nonbearing walls and partitions made of masonry in Occupancy Categories I, II and III.	*0.75*
g. Masonry or concrete fences at grade over 10 feet high.	*0.50*
2. Penthouses (defined in article 2 of subchapter 2 of chapter 1 of title 27 of the building code), except where framed by an extension of the building frame.	*0.75*
3. Connections for prefabricated structural floor and roof elements other than walls (see above) with force applied at center of gravity.	*0.75*
4. Diaphragms.[3]	--
II. Nonstructural Components	
1. a. Exterior ornamentation and appendages including cornices, ornamental statuaries or similar pieces of ornamentation.	*2.00*
b. Interior ornamentation and appendages in areas of public assembly including cornices, ornamental statuaries or similar pieces of ornamentation.	*2.00*
2. Chimneys, stacks, trussed towers, and tanks on legs.	
a. Supported on or projecting as an unbraced cantilever above the roof more than one-half its total height.	*2.00*
b. All others, including those supported below the roof with unbraced projection above the roof less than one-half its height, or braced or guyed to the structural frame at or above its center of mass.	*0.75*
3. Exterior signs or billboards.	*2.00*
III. Equipment and Machinery[4]	
1. Tanks and vessels (including contents), including support systems and anchorage.	*0.75*

Notes:

[1]*See Section 2312 (g) 2 for additional requirements for determining* C_p *for nonrigid equipment or for items supported at or below grade.*

[2]*See Section 2312 (h) 2 D (iii) and Section 2313 (g) 2.*

[3]*See Section 2312 (h) 2 I.*

[4]*Equipment and machinery include such items as pumps for fire sprinklers, motors and switch gears for sprinkler pumps, transformers and other equipment related to life-safety including control panels, major conduit ducting and piping serving such equipment and machinery.*

Table No. 23-Q
R_w Factors for Non-Building Structures

STRUCTURE TYPE	R_w
1. Tanks, vessels or pressurized spheres on braced or unbraced legs.	3
2. Cast-in-place concrete silos and chimneys having walls continuous to the foundation.	5
3. Distributed mass cantilever structures such as: stacks, chimneys, silos and skirt-supported vertical vessels.	4 4
4. Trussed towers (freestanding or guyed), guyed stacks and chimneys.	3
5. Inverted pendulum-type structures.	5
6. Cooling towers.	4
7. Bins and hoppers on braced or unbraced legs.	5
8. Storage racks.	5
9. Signs and billboards.	3
10. Amusement structures and monuments.	4
11. All other self-supporting structures not otherwise covered.	

Table No. 23-R
Spectral Acceleration
In Fraction of G
5% Damping

T (sec)	S_0	S_1	S_2	S_3
.01	*0.150*	*0.150*	*0.150*	*0.150*
.02	*0.150*	*0.150*	*0.150*	*0.150*
.05	*0.375*	*0.283*	*0.262*	*0.244*
.075	*0.375*	*0.375*	*0.336*	*0.303*
.090	*0.375*	*0.375*	*0.375*	*0.334*
.112	*0.375*	*0.375*	*0.375*	*0.375*
.267	*0.375*	*0.375*	*0.375*	*0.375*
.40	*0.250*	*0.375*	*0.375*	*0.375*
.48	*0.208*	*0.313*	*0.375*	*0.375*
.60	*0.167*	*0.250*	*0.300*	*0.375*
1.00	*0.100*	*0.150*	*0.180*	*0.225*
2.00	*0.050*	*0.075*	*0.090*	*0.113*
3.00	*0.033*	*0.050*	*0.060*	*0.075*

Note: This table presents acceleration (g) versus natural period (seconds) to facilitate the presentation of spectra in log-log form.

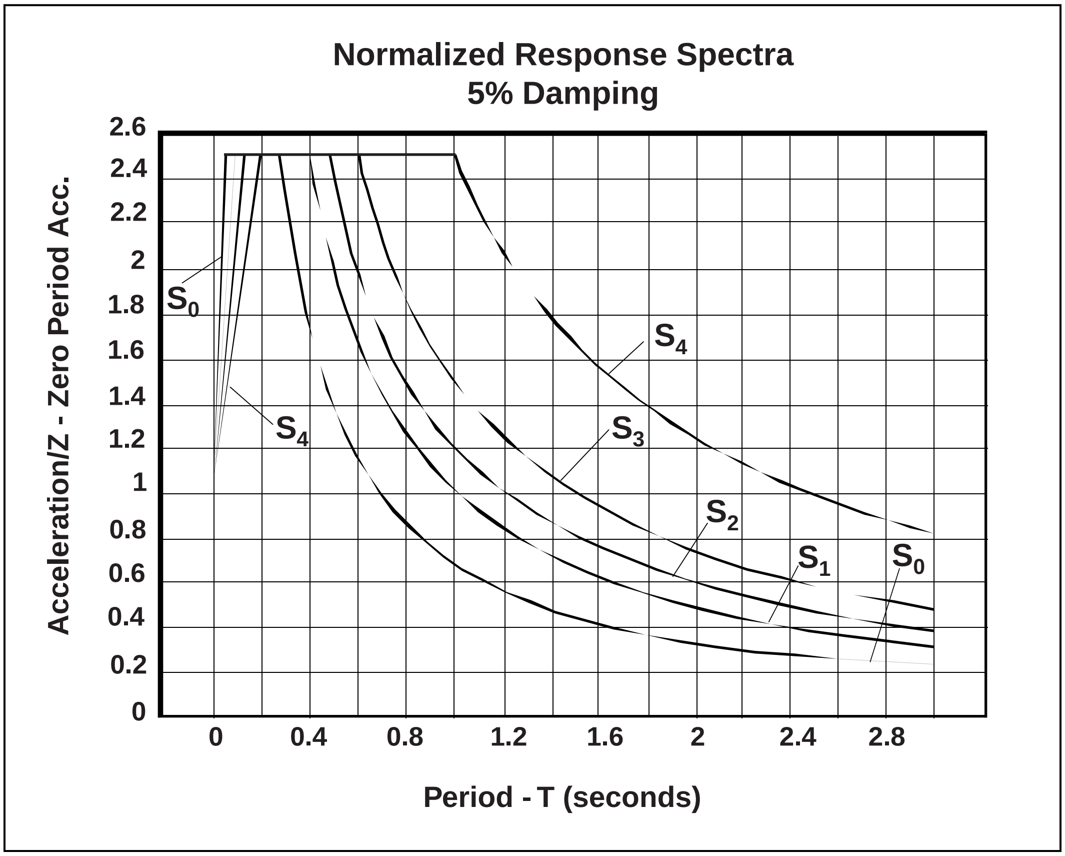

Figure 3: Response Spectra

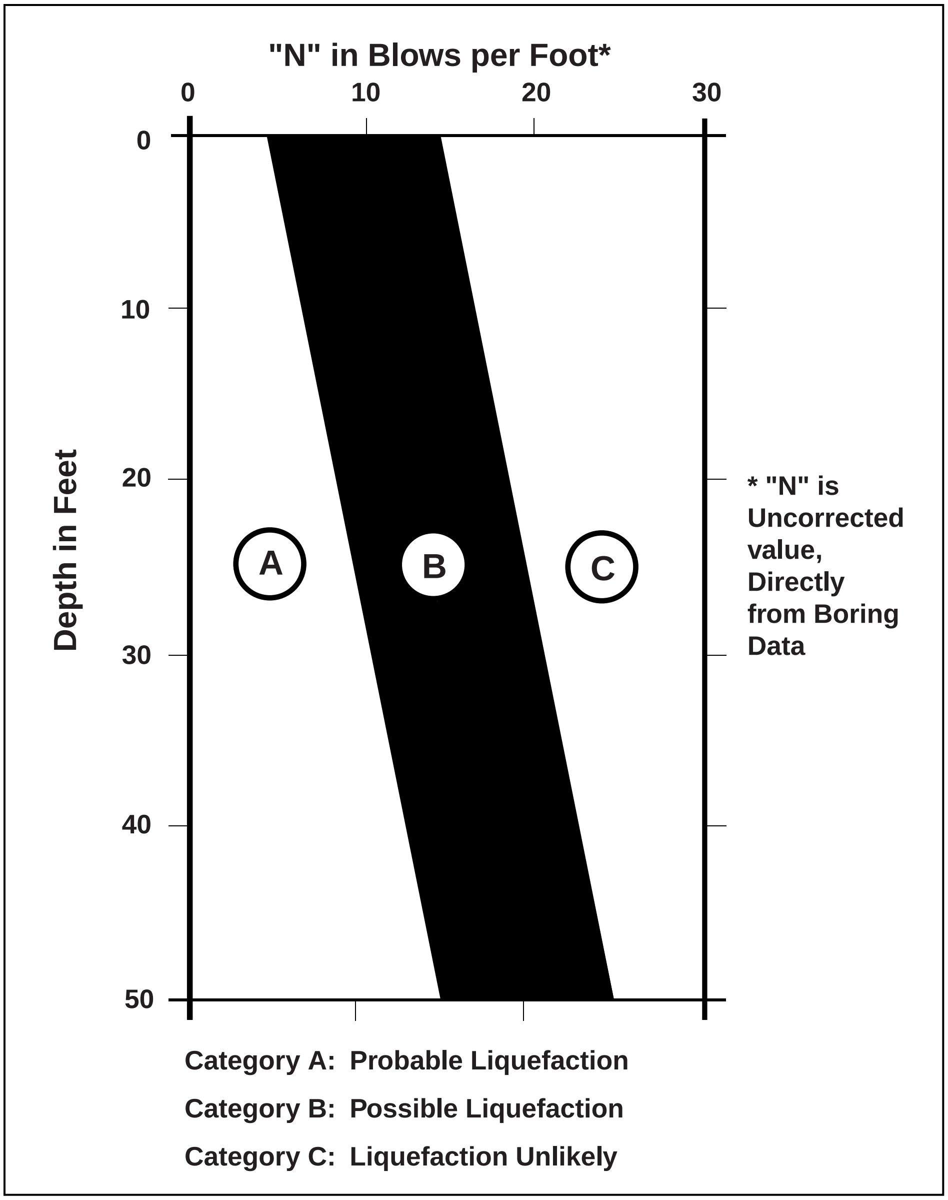

Figure 4: **Potential for Liquefaction of Level Ground**

2.3 Materials

The following sections refer to standards. Amendments have not been inserted into the original standards' text.

Section 8. The list of referenced national standards of reference standard RS-10 of the appendix to such chapter 1 of title 27 of the administrative code of the city of New York is amended by adding eight new standards to read as follows:

ACI 530/ASCE 5	*Building Code Requirements for Masonry Structures, as modified*	*1992*
ACI 530.1/ASCE 6	*Specifications for Masonry Structures, as modified*	*1992*
ANSI/ACI 318	*Building Code Requirements for Reinforced Concrete, as modified*	*1989*
MNL-120	*Prestressed Concrete Institute Design Handbook, Third Edition.*	*1985*
UBC Section 2723	*Steel Structures Resisting Forces Induced by Earthquake Motions in Seismic Zones Nos. 1 and 2 with Accumulative Supplement, as modified*	*1990*
AITC 117	*Specification for Structural Glued Laminated Timber of Softwood Species - Design Standard*	*1987*
	and Manufacturing Standard	*1988*
APA Form No. L350C	*Diaphragms - Design / Construction Guide*	*1989*
APA Form No. E30K	*Residential & Commercial Design/Construction Guide*	*1989*

Section 9. Reference standard RS 10-1 of the appendix to chapter 1 of title 27 of such code is amended by designating said standard RS10-1A and amending section 1.1 therein to read as follows:

1.1 **Scope**. This standard provides minimum requirements for the design and construction *of non enlargement alterations to* unit masonry *in buildings constructed on or before the effective date of this local law as an alternate to RS 10-1B*, not including plain or reinforced unit concrete, reinforced gypsum, or reinforced unit masonry. *All new construction and enlargement alterations in and of themselves of unit masonry on new or existing foundations, not including plain reinforced concrete, reinforced gypsum, or reinforced unit masonry shall comply with reference standard RS 10-1B*.

Section 10. Reference standard RS-10 of the appendix to chapter 1 of title 27 of such code is amended by adding new reference standard RS-10-1B to read as follows:

REFERENCE STANDARD RS 10-1B
MASONRY

ACI 530-92/ASCE 5-92	*Building Code Requirements for Masonry Structures, as modified.*
ACI 530.1-92/ASCE 6-92	*Specifications for Masonry Structures, as modified.*

MODIFICATIONS - The provisions of ACI 530-92/ASCE 5-92 shall be subject to the following modifications. The chapter and section numbers are from that standard.

Chapter 1 - General Requirements

Delete Section 1.3 - Approval of special systems of design or construction

Section 1.4 - Standards cited in this code

Section 1.4.1 Delete "ANSI A58.1-82 - Minimum Design Loads for Buildings and other Structures."

Chapter 5 - General Analysis and Design Requirements

Section 5.2.2 - Delete this section and substitute the following:

5.2.2 Service loads shall be in accordance with the building code of the city of New York of which this standard forms a part, with such live load reductions as are permitted in the building code of the city of New York. The load provisions of the referenced standards RS 9 shall be used.

Chapter 6 - Design Allowing Tensile Stresses in Masonry

Section 6.1.1 - Delete this section and substitute the following:

6.1.1 The provisions of this chapter are to be applied in conjunction with the provisions of Chapter 5 - General Analysis and Design Requirements and Appendix A.

Section 6.4 - Axial tension

Add the following at the end of section 6.4:

Axial tension stress shall be resisted entirely by steel reinforcement in accordance with Chapter 7.

Chapter 7 - Design Neglecting Tensile Strength of Masonry

Section 7.1.2 - Delete this section and substitute the following:

7.1.2 The provisions of this chapter are to be applied in conjunction with the provisions of Chapter 5 - General Analysis and Design Requirements and Appendix A.

Chapter 9 - Empirical Design of Masonry

Section 9.1.1.1 Seismic - Delete this section and substitute the following:

9.1.1.1 Seismic - Empirical requirements may apply to the design or construction of masonry for buildings, parts of buildings or other structures located in New York City.

Section 9.1.1.2 Wind - Delete this section and substitute the following:

9.1.1.2 Wind - Empirical requirements shall not apply to the design or construction of masonry for buildings, parts of buildings, or other structures to be located in areas where the basic wind speed will result in a wind pressure that exceeds 20 psf.

Section 9.2 - Height

Add the following sentence at the end of section 9.2:

However, members which are not part of the lateral force resisting system of the building are permitted to be designed in accordance with the provisions of Chapter 9 of RS 10-1B in buildings greater than 35 feet in height.

Section 9.9 - Miscellaneous requirements

Delete this section and add the following new Chapter 10:

Chapter 10 - Miscellaneous Requirements

10.1 CHASES AND RECESSES - *Masonry directly above chases or recesses wider than 12 inches shall be supported on lintels.*

10.1.1 Where permitted: *Chases and recesses shall be prohibited in any wall less than 12 inches thick and in the required area of piers and buttresses; except that where permitted in 8-inch walls, in residential buildings and in the apron under window openings, the maximum depth of chases shall be 4 inches.*

10.1.2 Maximum size: *The maximum permitted depth of a chase in any wall shall not be more than one-third of the wall thickness, and the maximum length of a horizontal chase or the maximum horizontal projection of a diagonal chase shall not exceed 4 feet except as provided for in Section 10.1.6; and except further that the maximum length of the apron below window sills in all walls shall not exceed the width of the window opening. Waterproofed chases in such aprons in 8-inch walls shall not exceed 4 inches in depth. The aggregate area of recesses and chases shall be not more than one-fourth of the area of the face of the wall in any one story.*

10.1.3 Waterproofing chases: *The backs and sides of all chases in exterior walls with less than 8 inches of masonry to the exterior surface shall be insulated and waterproofed.*

10.1.4 Fire resistive limitations: *Chases or recesses shall not reduce the thickness of masonry material below the minimum equivalent thickness required for firewalls, fire separation assemblies or required fire resistive coverings of structural members.*

10.1.5 Hollow walls: *Where chases and recesses are permitted in hollow walls and walls constructed of hollow blocks or tile, the chases and recesses shall be built in with the wall. Chases shall not be cut in hollow walls after erection.*

10.1.6 Continuous chases: *Where horizontal chases for the bearing of reinforced concrete floors and roof slabs are continuous, anchors shall be installed above and below the floor construction to resist bending and uplift in the wall due to flexure of the slab.*

10.2 LINTELS - *The design for lintels shall be in accordance with the provisions of Sections 5.6 and 7.3.3. Minimum end bearing shall be 4 inches.*

10.3 SUPPORT ON WOOD - *No masonry shall be supported on wood girders or other forms of wood construction.*

10.4 CORBELLING

10.4.1 *Solid masonry units shall be used for corbelling. The maximum corbelled projection beyond the face of the wall shall be not more than one half of the wall thickness or one half the wythe thickness for hollow walls; the maximum projection of one unit shall neither exceed one half the height of the unit nor one third its thickness at right angles to the face which is offset. Corbelling of hollow walls or walls built of hollow units shall be supported on at least one full course of solid masonry.*

10.4.2 Molded cornices: *Unless structural support and anchorage are provided to resist the overturning moment, the center of gravity of all projecting masonry or molded cornices shall lie within the middle one-third of* *the supporting wall. Terra cotta and metal cornices shall be provided with a structural frame of non-combustible anchored material.*

10.5 ARCHES AND LINTELS - *The masonry above openings shall be supported by properly buttressed arches or by lintels that bear on the wall at each end for at least 4 inches.*

10.6 PARAPET WALLS - *All cells in the hollow masonry units and all joints in solid, cavity, or masonry bonded hollow wall construction shall be filled solid with mortar. All corners of masonry parapet walls shall be reinforced with joint reinforcement or its equivalent at vertical intervals not greater than 12 inches. Such reinforcement shall extend around the corner for at least 4 feet in both directions and splices shall be lapped at least 6 inches. Parapet walls shall be properly coped and flashed with noncombustible, weatherproof material of a width not less than the width of the parapet wall plus sufficient overage for overlaps. Masonry parapet walls shall be not less than 8 inches in thickness and their height shall not exceed three times their thickness. Parapet walls shall be designed in accordance with the provisions of Appendix A.*

10.7 ISOLATED PIERS - *Isolated masonry piers shall be bonded as required for solid walls of the same thickness and shall be provided with adequate means for distributing the load at the top of the pier.*

10.8 BEARING DETAILS - *Concentrated loads shall be supported upon construction of solid masonry, concrete, or masonry of hollow units with cells filled with mortar, grout, or concrete and of sufficient height to distribute safely the loads to the wall or column, or other adequate provisions shall be made to distribute the loads.*

10.8.1 Joists: *Solid construction for support under joists shall be at least 2 1/4 inches in height and joists supported on such construction shall extend into the masonry at least 3 inches.*

10.8.2 Beams: *Solid construction for support under beams, girders, or other concentrated loads shall be at least 4 inches in height and the bearing of beams shall extend into the masonry at least 4 inches.*

10.9 USE OF EXISTING WALLS - *An existing masonry wall may be used in the alteration or extension of a building provided that it meets the requirements of this standard.*

10.9.1 Walls of insufficient thickness: *Existing walls of masonry units that are structurally sound, but that are of insufficient thickness when increased in height, may be strengthened by an addition of similar masonry units laid in type M or S mortar. The foundations and lateral support shall be equivalent to those required for newly constructed walls under similar conditions. All such linings shall be thoroughly bonded into existing masonry by toothings to assure combined action of wall and lining. Toothings shall be distributed uniformly throughout the wall, and shall aggregate in vertical cross-sectional area at least 15 percent of the total surface area of the lining. Stresses in the masonry under the new conditions shall not exceed the allowable stresses.*

10.10 PRECAUTIONS DURING ERECTION - *Temporary bracing shall be used wherever necessary to take care of any loads to which the walls may be subjected during erection. Such bracing shall remain in place as long as may be required for safety.*

10.11 HORIZONTAL COMPRESSION JOINTS - *All concrete framed buildings to be constructed over 35 ft. in height (as measured from adjoining grade to the main roof level), whose exterior wythe are of cavity wall construction with steel lintels, shall have horizontal compression joints in the exterior wythe to prevent masonry distress induced by vertical shortening of the structural frame.*

(a) Unless substantiated as indicated by (b) below, horizontal compression joints shall be 1/4 inch minimum thickness, with neoprene, polyethylene, or urethane gasket or equivalent joint filler filling the entire joint, except for a recess from the toe of the lintel angle to the exterior of the facing brick, to provide space for caulking. These joints shall be spaced at each floor.

(b) The applicant of record shall submit an engineering analysis establishing that proposed building compression joints as shown on the plans further apart than in (a) above are sufficient to provide for the effects of vertical shortening of the structural frame.

10.12 DRY-STACKED, SURFACE-BONDED MASONRY WALLS

10.12.1 General: *Dry-stacked, surface-bonded masonry walls may be used for only one and two family dwellings and shall comply with the requirements of this code for masonry wall construction.*

10.12.2 Materials: *Surface-bonding mortar shall comply with ASTM C476. Concrete masonry units shall comply with ASTM C55, C90 or C145.*

10.12.3 Design: *Dry-stacked, surface-bonded masonry walls shall be of adequate strength and proportions to support all superimposed loads without exceeding the allowable stresses listed in Table 10.12.3. Allowable stresses not specified in Table 10.12.3 shall comply with the requirements in this standard.*

Table 10.12.3
ALLOWABLE STRESS
GROSS CROSS-SECTIONAL AREA

Description	*Maximum allowable stress (psi)*
Compression	
Standard block	*45*
Shear	*10*
Flexural tension	
Vertical span	*18*
Horizontal span	*30*

10.12.4 Construction: *Construction of dry-stacked, surface-bonded masonry walls, including stacking and leveling of units, mixing and application of mortar, curing and protection, shall comply with ASTM C946.*

10.13 GLASS-BLOCK WALLS

10.13.1 Exterior wall panels: *The maximum dimensions of glass-block wall panels in exterior walls, where used singly or in multiples to form continuous bands of glass blocks between structural supports, shall be 25 feet in length and 20 feet in height between structural supports and expansion joints; and the area of each individual panel shall not be more than 250 square feet. Intermediate structural supports shall be provided to support the dead load of the wall and all other superimposed loads. Where individual panels are more than 144 square feet in area, a supplementary stiffener shall be provided to anchor the panels to the structural supports.*

10.13.2 Joint materials: *Glass blocks shall be laid up in Type S or N mortar with approved galvanized metal panel anchors in the horizontal mortar joints of exterior panels. The sills of glass-block panels shall be coated with approved water-based asphaltic emulsion, or other elastic waterproofing material, prior to laying the first mortar course, and the perimeter of the panels shall be caulked to a depth of not less than 1/2 inch with nonhardening caulking compound on both faces, or other expansion joints shall be provided. Where laid up in joint materials other than mortars herein defined, a single panel shall not be more than 100 square feet in area, nor more than 10 feet in either length or height.*

10.13.3 Wind and seismic loads: *Exterior wall panels held in place in the wall openings shall be designed to resist both the internal and external loads due to wind and seismic loads.*

10.13.4 Interior wall panels: *Solid or hollow glass blocks shall not be used in fire walls, party walls, fire separation assemblies or fire partitions, or for loadbearing construction. Such blocks shall be erected with mortar and reinforcement in metal frames, structural channels or embedded panel anchors as provided for exterior walls or other joint materials. All mortar-bearing surfaces of the glass block shall be precoated or prepared to insure* *adhesion between mortar and glass. Wood strip framing shall not be used in fire separation assemblies that are required to be fire resistance rated.*

> ***EXCEPTIONS:*** *Glass-block assemblies with a material and equipment acceptance number or Board of Standards and Appeals number having a fire resistance rating of not less than 3/4 hour shall be permitted in fire separations which have a required fire resistance rating of one hour or less and do not enclose exit stairways or exit passageways.*

10.14 VENEER

10.14.1 General - *Veneer, as used in this section, refers to an exposed facing wythe of brick, tile, ceramic veneer, terra cotta, concrete masonry units, cast stone, natural stone, or similar weather-resistant noncombustible masonry units laid in mortar and securely attached to a surface for the purpose of providing ornamentation, protection or insulation, but not intentionally so bonded as to exert common action under load. In lieu of the provisions of Section 10.14, veneers may be designed according to Chapters 5, 6 and 9 of reference standard RS 10-1B.*

10.14.1.1 Limitations: *Veneer shall not be assumed to add to the strength of any wall, nor shall it be assumed to support any load other than its own weight. No veneer shall be less than the thickness specified in Table 10.14.1.1. The height and length of veneer areas shall be unlimited, except as required to control expansion and contraction, and except as provided in subsection 10.14.2.*

TABLE 10.14.1.1 MINIMUM THICKNESS
OF MASONRY VENEER

Type of Veneer	*Minimum Thickness Actual (in)*
Anchored Type:	
Solid masonry units	*1 5/8*
Hollow masonry units	*1 5/8*
Ceramic veneer	*1*
Adhesion Type:	
Solid masonry units	*3/8*
Ceramic veneer	*3/8*

10.14.1.2 Design: *All anchor attachments shall be designed to resist a minimum positive or negative horizontal force as required for wind or seismic effects, and adhesion type veneer shall be designed to have a bond sufficient to withstand a shearing stress of 50 psi. At a minimum, the veneer shall also meet the attachment requirements of Section 10.14.2.1 and 10.14.3.1.*

10.14.1.3 Support of veneer: *The weight of all anchored type veneer shall be supported upon footings, noncombustible foundation walls, or other approved supports. Veneer above openings shall be supported upon noncombustible, non-corrosive lintels.*

10.14.2 Veneer on wood: *Anchored masonry veneer attached to wood frame structures shall be supported on noncombustible footings or foundation walls. The height of the veneer shall not exceed 35 feet measured from the top of the supporting footings or foundation walls. Where anchored veneer exceeding 20 feet in height is applied, it shall be supported in a manner that will provide for movement between the veneer and it backing.*

10.14.2.1 Attachment: *At a minimum, veneer of unit masonry shall be attached directly to wood studs, by one of the following means:*

a) With at least 22 gauge corrosion-resistance corrugated steel ties at least one inch wide, at vertical intervals of not more than 24 inches and horizontal intervals of not more than 32 inches, but in no case less than one tie for 3 1/2 square feet of wall area;
b) Directly to a 1 inch reinforced cement mortar base.

10.14.3 Veneer on masonry: *Veneer attached to masonry or concrete backing shall not be limited in height other than by compressive stresses.*

10.14.3.1 Attachment: *At a minimum, veneer shall be securely attached to the masonry or concrete backing by one of the following means or by a means that is equivalent in strength:*

a) Metal ties conforming to Section 5.8 except that ties shall be spaced not more than 24 inches apart either horizontally or vertically;
b) Corrosion-resistant dovetail slot anchors where the backing and the veneer has been designed for this type of attachment. Such anchors shall be formed from at least 16 gauge steel at least 1 inch wide;
c) Adhesion type masonry veneer shall be installed in accordance with the manufacturer's recommendations and setting plans;
d) Where anchored veneer is not grouted to the back, it shall be supported in a manner that will provide for movement between the veneer and its backing.

10.15 MISCELLANEOUS STRUCTURES AND SYSTEMS

10.15.1 Flat or Segmental Masonry Floor or Roof Arches: *The provisions of this section do not apply when masonry floor or roof arches are proportioned on the basis of structural analysis.*

10.15.1.1 Span: *The maximum clear span between supporting beams shall be 8 feet.*

10.15.1.2 Tie Rods: *All masonry flat arches or segmental arches shall be provided with tie rods in both the exterior and interior spans. The minimum size and spacing of tie rods shall be:*

For exterior spans - 1 1/4 inches round rods spaced 4 feet 6 inches apart.
For interior spans - 7/8 inches round rods spaced 4 feet 6 inches apart.
Washers shall be used with all tie rods. All tie rods shall have a minimum specified yield point of 36,000 psi.

10.15.1.3 Flat arches: *The depth of flat arches of burnt clay or shale hollow blocks shall be at least 1 1/2 inches for each foot of span, inclusive of the portion of the block extending below the under side of the beam, and such arches shall be at least 6 inches thick. Brick shall not be used for flat arches.*

10.15.1.4 Segmental arches: *Segmental arches shall have a rise of at least 1 inch per foot of span, and the minimum thickness shall be 6 inches for hollow tile arches and 4 in. for brick arches with a span of 5 feet or less and 8 inches for brick arches with a span exceeding 5 feet.*

10.15.1.5 Structural clay tile arches: *The blocks shall be at least two cells deep, shall be laid in type M or S mortar, and shall be properly keyed.*

10.15.1.6 Brick arches: *Brick arches shall be laid in a full bed of type M or S mortar and shall be solidly bonded.*

10.15.1.7 Openings in floors and roofs: *Suitable metal framing or reinforcement shall be provided in masonry arch and roof construction around any opening more than 1 foot 6 inches on a side.*

Delete sections 10.15.2 through 10.15.2.4.

Appendix A - Special Provisions for Seismic Design

Section A.1.1.1 - Delete this section and substitute the following:

A.1.1.1 Appendix A sets special requirements for masonry and construction of masonry building elements for seismic design as defined in reference standard RS 9-6.

Section A.2 - Delete this section.

Section A.3 - Delete the words "for Seismic Zone 2."

Section A.3.3 - Delete this section and substitute the following:

A.3.3 Distribution of seismic loads or forces shall be in accordance with the provisions of reference standard RS 9-6.

Section A.3.6 - Delete the first two sentences and substitute the following:

Masonry walls which require lateral forces shown in Table 23-P of reference standard RS 9-6 shall be anchored to all floors and roofs which provide lateral support for the walls. The anchorage of such walls or partitions shall provide direct connection capable of resisting the forces derived from Table No. 23-P or a minimum of 200 pounds per lineal foot of wall, whichever is greater.

Section A.3.8 - Delete this section and substitute the following:

A.3.8 Vertical reinforcement of at least 0.20 square inches in cross sectional area shall be provided continuously from support to support at each corner, at each side of each opening, at the ends of walls and at a maximum spacing of 10 feet apart throughout the wall. Horizontal reinforcement not less than 0.20 square inches in cross sectional area shall be provided: (1) at the bottom and top of wall openings and shall extend not less than 24 inches nor less than 40 bar diameters past the opening; (2) continuously at structurally connected roof and floor levels and at the top of walls; (3) at the bottom of the wall or in the top of the foundations when doweled to the wall; (4) at maximum spacing of 10 feet unless uniformly distributed joint reinforcement is provided. Reinforcement at the top and bottom of openings when used in determining the maximum spacing specified in item (4) above shall be continuous in the wall.

Add the following sections:

A.3.10 Non-bearing back-up or infill walls and non-bearing partitions need not comply with the vertical and horizontal (2 way) reinforcing requirements of section A.3.8 if the requirements set forth in A.3.10.1 through A.3.10.4 are met.

A.3.10.1 The cross sectional area of uniformly spaced steel reinforcing in either the horizontal or the vertical direction shall equal or exceed 0.0005 times the gross cross sectional area of the masonry.

A.3.10.2 Reinforcement shall be continuous between supports.

A.3.10.3 Spacing of prescribed horizontal reinforcement shall not exceed 16 inches for joint reinforcement, and 4 feet for reinforcement bars in grouted bond beams. When vertical reinforcement is used, bars shall not exceed placement at 10 feet on center and at the ends of walls.

A.3.10.4 Lateral support anchorage shall be provided between the non-loadbearing back-up, infill or partition wall and its structural support. Spacing of anchors shall conform to the provisions of Sections 4.2 and 5.11 and shall not exceed the spacing of prescribed reinforcement. Anchorage shall be designed to transfer lateral (out-of-plane) forces to the adjacent structural support.

Section A.4 - Delete this section.

MODIFICATIONS - *The provisions of ACI 530.1-92/ASCE 6-92 shall be subject to the following modifications. The chapter and section numbers are from that standard.*

Section 2.3.1 - Inspection and testing - Delete the opening sentence and substitute the following:

Inspection shall conform to the requirements of Articles 1.5 and 1.6, the inspection and testing provisions of the building code of the city of New York, and the following:

Section 11. Reference standard RS 10-2 of the appendix to chapter 1 of title 27 and of the administrative code of the city of New York is REPEALED and reenacted to read as follows:

REFERENCE STANDARD RS 10-2
REINFORCED MASONRY

ACI 530-92/ASCE 5-92 *Building Code Requirements for Masonry Structures, as modified.*

ACI 530.1-92/ASCE 6-92 *Specifications for Masonry Structures, as modified.*

MODIFICATIONS - *The provisions of ACI 530-92 and ACI 530.1-92 shall be subject to the same modifications as set forth in reference standard RS 10-1B and shall apply to reinforced masonry.*

EXCEPTION *- For buildings designed utilizing reinforced masonry construction in existence on the effective date of this local law, repairs or alterations to the facade or interior of the structure shall be done in accord with ACI 530-92/ASCE 5-92 Building Code Requirements for Masonry Structures, as modified and ACI 530.1-92/ASCE 6-92 Specifications for Masonry Structures, as modified except for those provisions found in Appendix A Special Provisions for Seismic Design.*

Section 12. Reference standard RS 10-3 of the appendix to chapter 1 of title 27 of such code is REPEALED and reenacted to read as follows:

REFERENCE STANDARD RS 10-3
ACI 318-1989 BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE

Comments - The Commentary on Building Code Requirements for Reinforced Concrete (ACI 318-89) may be used as a guide for interpreting this standard.

MODIFICATIONS - *The provisions of ACI 318-89 shall be subject to the following modifications. The section and subdivision numbers are from that standard.*

Chapter 1 General Requirements

Delete section 1.2.1 and substitute the following:

1.2.1 - The applicable provisions of the building code of the city of New York shall apply.

Delete section 1.2.2.

Delete section 1.2.3.

Delete section 1.3.1 and substitute the following:

1.3.1 - The applicable provisions of the building code shall apply.

Delete section 1.3.2.

Delete section 1.3.4.

Delete this section 1.4 and substitute the following:

1.4 - The provisions of the building code for equivalent systems of design shall apply.

Chapter 3 Specifications for Tests and Materials

Delete section 3.1.3.

Delete section 3.6.6 and substitute the following:

3.6.6 - Fly Ash may be used in lieu of Chemical Admixtures (ASTM C494) RS 10-44.

Chapter 5 Concrete Quality, Mixing and Placing

Section 5.1.1

Delete "5.3.2" on the second line and insert the words "Section 5.6.1.6 as listed", and delete the last sentence.

Section 5.1.2

Delete "5.6.2" on the second line and insert the words "the New York City Building Code".

Section 5.2.1(c)

Delete "5.6" on the second line and insert the words "the New York City Building Code".

Delete section 5.2.3.

Section 5.3

Delete the words "and/or trial mixtures".

Section 5.3.1

Delete the words "Standard deviation" and insert the words "Method II Proportioning on the basis of field experience".

Section 5.3.1.1(c)

Delete the words "except as provided in 5.3.1.2".

Delete section 5.3.1.2 in its entirety, including Table 5.3.1.2.

Section 5.3.2.1

Delete "or 5.3.1.2" at the end of the sentence.

Delete section 5.3.2.2. in its entirety, including Table 5.3.2.2.

Section 5.3.3

Delete the words "several strength test records, or trial mixtures" at the end of the sentence.

Section 5.3.3.1

Delete the third sentence starting with the words "For the purpose of" and ending with the words "less than 45 days".

Section 5.3.3.2

Add a period on the third line after the word "mixtures", delete the remainder of the section, and add the following sentence: "The Trial mixtures shall conform to the provisions of Section 27-605(a)(3) of the New York City Building Code."

Delete section 5.4.1 and substitute the following:

Proportioning shall conform to New York City Building Code Section 27-605 (a) Method I.

Delete Table 5.4 in its entirety.

Delete section 5.4.2.

Delete section 5.4.3.

Delete section 5.5 (b).

Delete section 5.6.1.1 and substitute the following:

5.6.1.1 - Whenever strength tests of concrete specimens are required by the provisions of the building code of the city of New York, compression test samples shall be taken directly from the mixer in accordance with reference standard RS 10-51, cured in accordance with reference standard RS 10-52, and tested at the age of 28 days or as otherwise specified in accordance with reference standard RS 10-17.

Delete section 5.6.1.2 and substitute the following:

5.6.1.2 - Three test specimens shall be molded for each 50 cubic yards or fraction thereof for each class of concrete placed in any one day's concreting. In addition, concrete test specimens shall be made from concrete taken out of the bucket, hopper or forms as directed by the engineer designated for controlled inspection. These test specimens shall be separate and distinct from those made from the mixer and shall be made from the same batch and cured and tested in the same manner as described above for the samples taken from the mixer.

Delete section 5.6.1.3 and substitute the following:
5.6.1.3 - The number of test specimens made from the concrete taken out of the bucket, hopper or forms may be reduced to a minimum of one set of three (3) specimens for every 150 cubic yards, or fraction thereof, for each class of concrete placed in any one day's concreting. When the concrete is being placed directly from the mixer into the forms without any intermediate conveyance, the above additional specimens will not be required.

Delete section 5.6.1.4 and substitute the following:

5.6.1.4 - Additional specimens may be molded and tested where there is a question as to the required interval between placing of concrete and stripping forms or placing the structure into use.

Add section 5.6.1.5:

5.6.1.5 - The test specimens shall be tested by a licensed concrete testing laboratory. The testing of each batch of three specimens shall be considered one strength test. The strength of such test shall be the average of the breaking strengths of the three specimens comprising the test, except that if one of the specimens shall manifest evidence of improper sampling, molding, handling or testing, it shall be discarded *and the remaining two averaged. If more than one specimen must be discarded, the entire strength test shall be voided.*

Add section 5.6.1.6:

5.6.1.6 - The average of all sets of three consecutive strength tests representing each class of concrete shall be equal or greater than the specified strength (f'_c) and not more than 10% of the strength tests shall have values less than the specified strength, but no test shall show an average strength less than 85% of the specified strength.

Delete section 5.6.2.1.

Delete section 5.6.2.2.

Section 5.6.2.3

After the word "satisfactory" in the second line, delete the remainder of the section and add the words "if the provisions of Section 5.6.1.6 are met".

Delete section 5.6.2.4.

Section 5.6.3.1

Delete the words "Building Official" on the first line and insert the word "Commissioner".

Section 5.6.4.1

Delete the first three lines through the words "[Section 5.6.2.3(b)]", and substitute the following: "If tests of laboratory cured specimens fail to conform to the requirements of Section 5.6.1.6 refer to Section 5.6.3.4."

Section 5.6.4.2

Delete the words "strength test more than 500 psi below specified value of f'_c" and insert the words "set of three specimen tests which fail to conform to the requirements of Section 5.6.1.6."

Add section 5.9.3:

5.9.3 - Conveying by pumping methods shall be in accordance with the applicable provisions of the Building Code.

Section 5.10.4

Add a period on the second line after the word "used", delete the words "unless approved by the Engineer" and add the following sentence: "For additional requirements see applicable provisions of the Building Code Section 27-607(a)(2)."

Add the following section:

5.14 SPECIAL REQUIREMENTS FOR HIGH STRENGTH CONCRETE

5.14.1 All high strength concrete (6000 PSI and higher) shall be proportioned and manufactured only in accordance with the provisions of Building Code Section 27-605(b) Method II Proportioning on the basis of field experience.

5.14.2 All high strength concrete specimens shall be made utilizing metal or plastic molds that comply with reference standard RS 10-52. Each test shall consist of eight specimens taken directly from the mixer. Two *specimens shall be tested at seven days, three at 28 days and three at 56 days. These requirements are in addition to the hopper specimens as required by the Building Code.*

5.14.3 At the time of placement of high strength concrete, two concrete production facilities shall be available. Said facilities shall have been previously approved by the architect or engineer designated for controlled inspection.

5.14.4 All high strength concrete for columns shall be of normal weight concrete.

5.14.5 The requirements of Section 10.13.4 shall be adhered to in all respects.

5.14.6 Where lightweight concrete is to be used for the floor system, the columns and the beam or slab "sandwich" immediately above the columns shall be stone concrete, placed in accordance with the requirements of Section 10.13.

5.14.7 The engineer will insure that there are no cold joints at the interface between the lightweight concrete and stone portions of the slabs or beams.

5.14.8 All data shall be submitted periodically to the Department of Buildings for review.

Chapter 6 Formwork, Embedded Pipes and Construction Joints

Add the following section:

6.1.7 For additional form work requirements, see applicable provisions of Building Code Section 27-1035.

Add the following sections:

6.3.13 Concrete cover over electrical cables and snow melting pipes in sidewalks shall meet the requirements of the Bureau of Highways of the Department of Transportation.

6.3.14 No conduits, pipes or other similar embedded items will be permitted in prestressed or post-tension concrete members other than as shown on the plans as filed with the Department of Buildings or on shop drawings reviewed by the engineer of record. Computations demonstrating the effects of such embedded items on the structural adequacy of prestressed or post-tensioned concrete shall be submitted.

Chapter 10 Flexure and Axial Loads

Add the following section:

10.13.4 When the specified compressive strength of concrete in a column is greater than 1.4 times that specified for a floor system, the following additional requirements shall be adhered to:

(a) All of the design provisions of Section 10.13 (unmodified) are adhered to.
(b) Application is made to the Borough Superintendent in each individual case.
(c) The concrete construction is supervised and inspected continuously by a full-time Professional Engineer responsible for controlled inspection of concrete and not in the regular employ of the owner or contractor. Such engineer shall perform no other work during the construction of the particular building nor shall he delegate his responsibility to any subordinates. (d) The Professional Engineer referred to in subdivision (c) above, without incurring any personal liability, shall be authorized to stop construction, reject any concrete, direct that the concrete testing laboratory being used be dismissed and a new laboratory be retained.

(e) Affidavits by the parties involved shall be filed with and acceptable to the Borough Superintendent prior to approval of any plans.

Chapter 12 Development and Splices of Reinforcement

Add the following section:

12.1.3. Development (Section 12.2) and splice lengths (Section 12.5) computed based on the minimum requirements of the ACI 318-83 code are deemed equally applicable for usage.

Chapter 16 Precast Concrete

Delete section 16.4.2 and substitute the following:

16.4.2 Lifting devices shall have a capacity sufficient to support four times the appropriate portion of the members dead weight. The inclination of the lifting force shall be considered.

Delete section 16.4.2.1.

Delete section 16.4.2.2.

Delete section 16.4.2.3.

Chapter 21 Reinforced Concrete Structure Resisting Forces Induced by Earthquake Motions

Add the following sentence to the beginning of section 21.2.1.2:

New York City is to be considered in a region of moderate risk.

Section 13. Reference standard RS 10-4 of the appendix to chapter 1 of title 27 of such code is REPEALED and reenacted to read as follows:

REFERENCE STANDARD RS 10-4
PRECAST CONCRETE AND PRESTRESSED CONCRETE

ACI 318-1989 *Building Code Requirement for Reinforced Concrete*
MNL-120-1985 *Prestressed Concrete Institute Design Handbook, Third Edition.*
MODIFICATIONS *- The applicable section of ACI 318-89 as modified by the applicable provisions of reference standard RS 10-3 shall apply for precast concrete and prestressed concrete.*

Section 14. Reference standard RS-10 of the appendix to such chapter, title and code is amended by adding a new reference standard RS10-5C to read as follows:

REFERENCE STANDARD RS 10-5C
STEEL STRUCTURES RESISTING EARTHQUAKE FORCES

The following text includes the amendments to the 1988 UBC within the text of the UBC provisions. These amendments are shown in italics.

UBC SECTION 2723-1990 Steel Structures Resisting Forces Induced by Earthquake Motions in Seismic Zones Nos. 1 and 2 with Accumulative Supplement

MODIFICATIONS - *The provisions of UBC Section 2723 shall be subject to the following modifications. The subdivisions, paragraphs, subparagraphs and items are from that standard.*

Sec. 2723. *(a)* ***General.*** *Design and construction of steel framing in lateral force resisting systems shall conform to the requirements of this reference standard. The use of reference standard RS 10-5B is prohibited for the design of seismic resisting elements.*

(b) ***Definitions.***

ALLOWABLE STRESSES *are prescribed in reference standard RS 10-5A.*

CHEVRON BRACING *is that form of bracing where a pair of braces located either above or below a beam terminates at a single point within the clear beam span.*

CONNECTION *is the group of elements that connect the member to the joint.*

DIAGONAL BRACING *is that form of bracing that diagonally connects joints at different levels.*

ECCENTRIC BRACED FRAME (EBF) *is that form of braced frame where at least one end of each brace intersects a beam at a point away from the column girder joint.*

GIRDER *is the horizontal member in a seismic frame. The words beam and girder may be used interchangeably.*

JOINT *is the entire assemblage at the intersections of the members.*

K BRACING *is that form of bracing where a pair of braces located on one side of a column terminates at a single point within the clear column height.*

LINK BEAM *is that part of a beam in an eccentric braced frame which is designed to yield in shear and/or bending so that buckling of the bracing members is prevented.*

STRENGTH *is the strength as prescribed in reference standard RS 10-5A.*

V BRACING *is that form of chevron bracing that intersects a beam from above and inverted V bracing is that form of chevron bracing that intersects a beam from below.*

X BRACING *is that form of bracing where a pair of diagonal braces cross near midlength of the bracing members.*

(c)1. ***Materials.*** *Materials shall be as prescribed in reference standard RS 10-5A. Structural steel designed to be part of the lateral force resisting system of multistory buildings shall not have a specified yield strength greater than 50,000 psi.*

2. ***Member Strength****. When these provisions require that the strength of the member be developed, the following shall be used:*

Members	***Strength***
Flexure	$M_s = ZF_y$
Shear	$V_s = 0.55F_y dt$
Axial compression	$P_{sc} = 1.7F_a A$
Axial tension	$P_{st} = F_y A$
Connectors:	
Full penetration welds	$F_y A$
Partial penetration and fillet welds	*1.7* Allowable*
Bolts	*1.7* Allowable*

Members need not be compact unless otherwise required by this chapter.

(d) **Ordinary Moment Frame Requirements.** Girder-to-column connections of ordinary moment frames shall meet the requirements of *paragraph 1 of Special Moment-resisting Frame (SMRF) Requirements* unless it can be shown that they are capable of resisting the combination of gravity loads and 3 (R_w/8) times the design seismic forces.

(e) **Special Moment-resisting Space Frame (SMRSF) Requirements.** 1. **Girder to column connection.** A. **Required Strength.** The girder to column connection shall be adequate to develop the lesser of the following:

(i) The strength of the girder in flexure.

(ii) The moment corresponding to development of *the panel zone strength, defined as:*

$V = 0.55F_y d_c t\ (1 + 3b_c t_{cf} 2/d_b d_c t)$

where:
t = the total thickness of the joint panel zone including doubler plates
d = the depth of the beam
d_c = the column depth
b = the width of the column flange
t_{cf} = the thickness of the column flange.

EXCEPTION: Where a connection is not designed to contribute flexural resistance at the joint, it need not develop the required strength if it can be shown to meet the deformation compatibility requirements of Section 2312 (h) 2 D.

B. Connection strength. The girder-to-column connection may be considered to be adequate to develop the flexural strength of the girder if it conforms to the following:

(i) The flanges have full penetration butt welds to the columns.

(ii) The girder web-to-column connection shall be capable of resisting the girder shear determined for the combination of gravity loads and the seismic shear forces which result from compliance with Section 2723 (e) A. This connection strength need not exceed that required to develop gravity loads plus 3 (R_w/8) times the girder shear resulting from the prescribed seismic forces.

Where the flexural strength of the girder flanges is greater than 70 percent of the flexural strength of the entire section (i.e., bt $(d\text{-}t_f)F_y > 0.7Z_xF_y$) the web connection may be made by means of welding or high-strength bolting.

For girders not meeting the criteria in the paragraph above, the girder web-to-column connection shall be made by means of welding the web directly or through shear tabs to the column. That welding shall have a strength capable of developing at least 20 percent of the flexural strength of the girder web. The girder shear shall be resisted by means of additional welds or friction-type high-strength bolts or both.

C. Alternate connection. Connection configurations utilizing welds or high-strength bolts not conforming with paragraph B above may be used if they are shown by test or calculation to meet the criteria in paragraph A above. Where conformance is shown by calculation, 125 percent of the strengths of the connecting elements may be used.

D. Flange detail limitations. For steel whose specified strength is less than 1.5 times the specified yield strength, plastic hinges shall not form at locations in which the beam flange area has been reduced, such as for bolt holes. Bolted connections of flange plates of beam-column joints shall have the net-to-gross area ratio A_e/A_g equal to or greater than $1.25_y/F_u$.

2. **Trusses in SMRSF.** Trusses may be used as horizontal members in SMRSF if the sum of the truss seismic force flexural strength exceeds the sum of the column seismic force flexural strength immediately above and below the truss by a factor of at least 1.25. For this determination, the strengths of the members shall be reduced by the gravity load effects. In buildings of more than one story, the column axial stress shall not exceed $0.4F_y$ and the ratio of the unbraced column height to the least radius of gyration shall not exceed 60. The connection of the truss chords to the column shall develop the lesser of the following:

A. The strength of the truss chord.

B. The chord force necessary to develop 125 percent of the flexural strength of the column.

3. **Girder-column joint restraint.** A. **Restrained joint.** Where it can be shown that the column of SMRSF remain elastic, the flanges of the columns need be laterally supported only at the level of the girder top flange. Columns may be assumed to remain elastic if one of the following conditions is satisfied:

(i) $S/Z_c\left(F_{yc} - f_a\right)/SZ_b F_{yb} > 1.0$

where $(f_a \geq 0)$.

(ii) The flexural strength of the column is at least 1.25 times the moment that corresponds to the panel zone shear strength.

(iii) Girder flexural strength of panel zone strength will limit column stress ($f_a + f_{bx} + f_{by}$) to F_y of the column.

(iv) The column will remain elastic under gravity loads plus $3(R_w/8)$ times the prescribed seismic forces.

Where the column cannot be shown to remain elastic, the column flanges shall be laterally supported at the levels of the girder top and bottom flanges. The column flange lateral support shall be capable of resisting a force equal to one percent of the girder flange capacity at allowable stresses (and at a limiting displacement perpendicular to the frame of 0.2 inch). Required bracing members may brace the column flanges directly or indirectly through the column web or the girder flanges.

B. **Unrestrained joint.** Columns without lateral support transverse to a joint shall conform to the requirements of *Section 1.6.2 of reference standard RS 10-5A* with the column considered as pin ended and the length taken as the distance between lateral supports conforming with A above. The column stress, f_a, shall be determined from gravity loads plus the lesser of the following:

(i) $3(R_w/8)$ times the prescribed seismic forces.

(ii) The forces corresponding to either 125 percent of the girder flexural strength or the panel zone shear strength.

The stress f_{by}, shall include the effects of the bracing force specified in *Section 2723(f)3A as well as P-delta effects*.

L/r for such columns shall not exceed 60.

At truss frames, the column shall be braced at each truss chord for a lateral force equal to one percent of the compression yield strength of the chord.

4. **Changes in beam flange area.** Abrupt changes in beam flange area are not permitted within possible plastic hinge regions of special moment-resistant frames.

*5. **Drift Calculations.** Drift calculations shall include bending and shear contributions from the clear girder and column spans, column axial deformation, and the rotation and distortion of the panel zone.*

> ***EXCEPTIONS:*** *1. Drift calculations may be based on column and girder centerlines where either of the following conditions is met:*
>
> *a. It can be demonstrated that the drift so computed for frames of similar configuration is typically within 15 percent of that determined above.*
>
> *b. The column panel zone strength can develop 0.8 Σ/M_s of the girders framing to the column flanges at the joint.*
>
> *2. Column axial deformations may be neglected if they contribute less than 10 percent to the total drift.*

(f) **Requirements for Braced Frames.** 1. **General.** The provisions of this section apply to all braced frames, except eccentrically braced frames (EBF) designed in accordance with Section 2722 (h). Those members which resist seismic forces totally or partially by shear or flexure shall be designed in accordance with Section 2723 (e).

2. **Bracing members.** A. **Stress Reduction**. The allowable stress, F_{as}, for bracing members resisting seismic forces in compression shall be determined from the following formula:

$$F_{as} = \beta F_a \qquad (22\text{-}4)$$

WHERE:

F_a = the allowable axial compressive stress allowed in *1.5.1.3 and Section 1.5.6 of reference standard RS 10-5A*.

β = the stress reduction factor determined from the following formula:

$$\beta = 1/ [1 + ([KL/r] /2C_c)] > 0.8$$

B. Built-up members. The L/r of individual parts of built-up bracing members between stitches, when computed about a line perpendicular to the axis through the parts, shall not be greater than 75 percent of the L/r of the member as a whole.

C. Compression elements in braces. The width-thickness ratio of stiffened and unstiffened compression elements used in braces shall be shown in *1.9 of reference standard RS 10-5A*.

3. **Bracing connections.** A. **Forces.** Bracing connections shall be designed for the lesser of the following:

(i) The tensile strength of the bracing.

(ii) $3(R_w/8)$ times the force in the brace due to the prescribed seismic forces.

(iii) The maximum force that can be transferred to the brace by the system. Beam-to-column connections for beams that are part of the bracing system shall have the capacity to transfer the force determined above.

B. **Net Area.** In bolted brace connections, the ratio of effective net section area to gross section area shall satisfy the formula:

$$\frac{A_e}{A_g} = \frac{1.2aF*}{F_u} \tag{22-6}$$

WHERE:

A_c = effective net area as defined in Section 2711 (b) 2.
F* = stress in brace as determined in Section *2723 (f) 3 A*.
F_u = Minimum tensile strength.
α = fraction of the member force from Section *2723 (f) 2 A* that is transferred across a particular net section.

4. **Bracing configuration for chevron and K bracing.** A. Bracing members shall be designed for 1.5 times the otherwise prescribed forces.

B. The beam intersected by chevron braces shall be continuous between columns.

C. Where chevron braces intersect a beam from below, i.e., inverted V brace, the beam shall be capable of supporting all tributary gravity loads presuming the bracing not to exist.

EXCEPTION: This limitation need not apply to penthouses, one-story buildings or the top story of buildings.

5. **One- and two-story buildings.** Braced frames not meeting the requirements of Sections 2723 (g) 2 and 3 may be used if they are designed for forces of *3 (R_w /8)* times the code equivalent static forces at code-allowable seismic stresses.

6. **Non-building Structures.** Non-building structures with R_w values defined by Table No. 23-Q, need only comply with the provisions of Section 2723 (f) 3.

(g) **Eccentrically Braced Frames (EBF).** EBFs shall comply with the requirements of Section 2722 (h).

(h) **Nondestructive Testing.** Shall comply with the provisions of *reference standard RS 10-5A*.

TIMBER

Section 15. The first paragraph of section 10 of reference standard RS 10-9 of the appendix to chapter 1 of title 27 of the administrative code of the city of New York is amended to read as follows:

> Plywood diaphragms may be used to resist horizontal forces in horizontal and vertical distributing or resisting elements, provided the deflection in the plane of the diaphragm, as determined by calculations, tests, or analogies drawn therefrom, does not exceed the permissible deflection of attached distributing or resisting elements. *Diaphragms to resist earthquake loads may be designed and constructed in accordance with reference standard RS 10-58.*

Section 16. Reference standard RS 10-18 of the appendix to chapter 1 of title 27 of such code is amended by adding three new standards to read as follows:

ANSI/AITC A190.1-1983	*Structural Glued Laminated Timber and AITC 200-1983 Inspection Manual.*
AITC 117-1987	*Specification for Structural Glued Laminated Timber of Softwood Species - Design Standard.*
AITC 117-1988	*Specification for Structural Glued Laminated Timber of Softwood Species - Construction Standard.*

Section 17. Reference standard RS-10 of the appendix to chapter 1 of title 27 of such code is amended by adding a new reference standard RS 10-58 to read as follows:

REFERENCE STANDARD RS 10-58

APA Form No. L350C-1989	*Diaphragms - Design/Construction Guide.*
APA Form No. E30K-1989	*Residential & Commercial, Design/Construction Guide.*

Section 18. This local law shall take effect one year after it shall have been enacted into law and shall apply to all buildings for which an application for the approval of plans for the construction of such building has been filed with the department of buildings on or after the effective date of this local law.

Section 3
Commentaries

The commentaries in the following subsections were provided by committee members with expertise in particular technical disciplines. The titles and authors of each subsection are as follows:

- 3.1 **Geotechnical and Seismological Aspects** by Klaus Jacob
- 3.2 **Loads and Load Procedures** by Joseph Kelly
- 3.3 **Seismic Design of Nonstructural Components, Elements of Structures and Non-Building Structures** by Leo E. Argiris
- 3.4 **Application to Building Additions and Alterations** by Ramon Gilsanz
- 3.5 **Economic Impact Study** by Irwin Cantor

3.1 Geotechnical and Seismological Aspects

New York City is not free from earthquakes. It is located in a zone of moderate seismicity. For instance, since 1884 the entire state of New York has experienced four damaging earthquakes with magnitudes between about M~5 and M~5.5 and many smaller earthquakes are recorded every year. The greater New York City area alone can expect on average one magnitude M~5 earthquake about once every 100 to 200 years (the last such damaging event with magnitude M~5.2 occurred in 1884). Based on geological arguments and comparison to geologically similar regions elsewhere, the possibility cannot be excluded that magnitudes as high as M~7 may occur near New York City, including adjacent offshore areas on the Atlantic coastal shelf. This possibility for $M > 6$ earthquakes exists despite the fact that in the short historic record (about 300 years), no larger earthquakes than M=5.2 have been observed near New York City. But during historic time much larger ($M = 7 \pm .5$) and quite damaging events have occurred elsewhere along the Atlantic coast of eastern North America.

The ground motions associated with earthquakes in the eastern United States differ distinctly from ground motions in the western U.S. in several important ways. Eastern earthquakes are reported to have released higher rock stresses compared to their western counterparts, thereby causing the ground motions to contain more high-frequency energy. The ground motion shaking is more intensely felt to larger distances because the Earth's crust and its rocks in the eastern U.S. transmit seismic waves more efficiently, especially at the higher frequencies of engineering interest. This stronger shaking, especially at shorter building periods and to larger distances, is caused by the fact that the crustal rocks in the eastern U.S. tend to be older, more competent, and less riddled with seismically active faults, compared to generally younger California rocks along the tectonically very active San Andreas fault system.

Other differences relate to the geological near-surface conditions: very competent, hard rocks are exposed at the surface in many portions of New York City (e.g. Bronx and Manhattan) ever since the glaciers retreated from the area about 10,000 years ago. Along the Hudson, Harlem and East River, along the water front in the New York Harbor estuary, and in other low-lying lands and former marshes (e.g. Flushing Meadow), very soft sediments have been deposited since glacial times, sometimes almost directly onto the exposed, hard bedrock formations. Land has been reclaimed by filling in marshland or along the waterfront. The contrasts in stiffness between the soft soils and underlying hard rocks create often extreme conditions for frequency-selective amplification of ground motions. On sites with deep soft soils, ground motions with periods shorter than 0.3 seconds tend to be attenuated (diminished in amplitude), and those with longer periods (> 0.3 sec) are often amplified, sometimes by factors up to 5 or 8, compared to those observed on adjacent hard-rock sites. Moreover, liquefaction of cohesionless soils, such as sands and silts, can be expected for smaller earthquakes and to larger distances in the eastern U.S. than in the western U.S. Soil liquefaction is caused by pore pressure built-up in water-saturated unconsolidated, cohesionless soils. Soil liquefaction often results in considerable loss of bearing strength when the soils liquefy during prolonged shaking. Liquefaction also can cause lateral spreads and settlement of soils.

Although the New York City Seismic Building Code became effective as late as 1996, most of the technical seismological content of the code provisions was drafted in the late 1980's. It was based on the combined seismic, geotechnical and strong ground motion information then available. This information was taken into account when reviewing whether the seismic provisions of the Uniform Building Code (UBC), used as the primary reference code, needed any modifications when applied to New York City. The original UBC seismic provisions had been largely based on California or other west-coast seismic data and experiences, which rarely apply to New York City's geologic conditions without sometimes considerable modifications. The seismic information available in the 1980's for the eastern U.S. and New York City affected the seismic load factors, seismic design-spectral shapes, spectral site coefficients for amplification of ground motions on soil sites, and the inferred liquefaction potential of soil sites. Accordingly, some of the seismic loads and soil parameters had to be modified from those quoted in the referenced Uniform Building Code.

The seismic-geotechnical features of the New York City seismic code provisions as drafted on the basis of the seismological information available in the late 1980's can be summarized as follows:

- The design ground motions were intended to represent ground motions expected to have a 90% probability of not being exceeded in 50 years, corresponding to an average recurrence period of about 500 years. Of course this implies a 10% chance that the motions (and corresponding seismic loads) may be exceeded in 50 years. These values are only estimated targets that unintentionally can vary somewhat between soil and rock sites, and also with the natural building period (i.e. building height). With the occurrence of future seismic events and with the development of new seismic hazard mapping procedures, new information will come to light that may require that these estimates be updated periodically. These changes may increase or decrease the specified seismic loads depending on site conditions and building type and period. Since the code only represents minimum requirements, some building owners may opt to seek greater protection by choosing design options capable of resisting ground motions and design forces that are higher than those outlined in the current code. This aspect is further commented on below.

- The seismic code provisions allow design ground motions either from code-prescribed design spectra, or those based on site-specific investigations.

- New York City belongs to a single seismic zone with a seismic zone factor Z=0.15. The seismic zone factor relates numerically (but is not identical) to peak ground acceleration when measured in units of g, the gravitational acceleration at the Earth's surface.

- Five rock and soil classes, S_0 to S_4, apply that can be determined using standard New York City material classifications, in conjunction with standard penetration test (SPT) blow counts and shear wave velocities for rock. The soil/rock profiles need to be defined from geotechnical information generally to depths of 100 feet or less, below grade.

- The five classes of rock and soil profiles used in the New York City code are associated with five site coefficients that vary from S_0=0.67 for hard rock, to S_4=2.5 for the softest soils; thus the code allows for a maximum site amplification of S_4/S_0=3.75 between soft soils and hard rock. This is a higher ratio than used in all U.S. seismic codes of pre-1994 vintage. These national codes generally failed to realistically quantify the observed differences in ground motions between soft-soil and hard-rock sites. The site factors adopted for the New York City site classes thus reduce the code-prescribed seismic loads for structures founded on the hardest rocks, and introduce load penalties for structures founded on the softest soil sites, in accordance with site-dependent damage patterns observed globally during many past earthquakes.

- A liquefaction screening test has been included for sites containing water-saturated non-cohesive soils in the upper 50 feet. These soil profiles are identified based on geotechnical borings that measure standard penetration test (SPT) blow counts. Two separate criteria for liquefaction screening are used; one applies to buildings that belong to ordinary occupancy categories, and a more stringent one for more important buildings in special occupancy categories.

The geotechnical/seismic data and other types of information that were used to modify the UBC code provisions and seismic loads were believed to reflect the then current level of knowledge of seismic and geotechnical processes in the eastern U.S. in general, and for New York City in detail. Nevertheless, it must be kept in mind that the information used was based on a limited number of observations and inferences. Also, there is no guarantee that the past limited seismic experience can be used as an unequivocal guide to the future behavior of earthquakes in this region. The resulting design motions and seismic loads were believed to be adequately representative of the seismic and geotechnical conditions characteristic for New York City, and for the targeted ground motion recurrence period of about 500 years. Therefore, the design motions were intended to provide a nearly uniform and balanced level of protection between the different soil and rock conditions, and for the different building periods that exist in New York City.

Available seismic information does change with time. Also one must keep in mind that the code provisions are intended to represent minimum requirements aimed primarily at preventing catastrophic structural collapse and related loss of lives, rather than losses from non-life-threatening damage and/or business interruptions. Owners and developers interested in added seismic protection for their structures are not prevented from adopting more severe ground motions than those specified as minimum conditions in the code. As a guide, one can estimate that the doubling of the code-specified ground motions raises their average recurrence period by a factor of five, from about 500 years to about 2,500 years, and lowers the corresponding exceedance probability to one fifth, from about 10% in 50 years to about 2% in 50 years.

3.2 Loads and Load Procedures

The New York City Building Code prescribes minimum seismic design loading provisions that include both dynamic and static lateral load procedures. The static load procedure is applicable to regular buildings and the dynamic load procedure applicable generally; where both can easily be combined with the gravity loads to be used to design the structural framing. The static lateral load procedure has been developed based on actual building seismic response experience of many years to approximate the dynamic response which a building frame undergoes during an earthquake. The lateral load coefficients represent the base acceleration, site soil dynamic displacement effects, as well as to approximate the nonlinear response characteristics of the structure. The "R_W" factor used in the building code has been developed to adjust the allowable stress design for the nonlinear inelastic response. The resulting total foundation seismic forces on the building are distributed at each floor to approximate the effects of the various modes of structural vibration. Special detailing of structural components is used to assure that ductility is provided for the expected inelastic strains that take place with the range of lateral seismic displacements.

3.2.1 Ground Motion

An earthquake causes the ground to shake. The amount of ground shaking at a particular building site will depend upon the energy release or magnitude of the earthquake at the source, distance to the location of the epicenter (point on the ground surface below which rupture begins) of the earthquake, and the characteristics of the soil at the site. The earth shaking decreases with distance to the building site. This decrease in earth vibrations is empirically established by attenuation relationships, which are a function of magnitude and distance. Several such attenuation equations have been used in establishing seismic loading in the northeastern United States.

For a building design in New York City which may be considered within 13.7 miles of a possible earthquake site with a maximum earthquake magnitude of 5 and also within 33.6 miles of another possible earthquake site with a maximum magnitude 6, a number of earthquakes can be expected during the lifetime of the building from each of the sites. Design loads are based on a return period of 475 years which has a 10% probability of being exceeded in 50 years. This criteria has been traditionally used in structural engineering practice for seismic design. It is of interest to note that the magnitude 6 earthquake centered 33.6 miles away will likely be less severe than a magnitude 5 earthquake 13.7 miles away. Combinations of both the magnitude and distance of the various design earthquakes possible have been considered to determine the basic seismic ground motion prescribed within the building code.

3.2.2 Response of a Structure

As the ground shakes, mass (weight/gravity) of all of the building components of a structure will be excited through their inertia characteristics, and the structure and components will vibrate, resulting in dynamic forces, stresses and displacements. A simple, single-degree-of-freedom, pendulum structure with a single mass, M, supported by a column with a lateral stiffness, k, will respond to a ground acceleration time history in its natural period, T. The period will be $2\pi\sqrt{M/k}$. The amplitude, velocity and acceleration of its response will be dependent on the amplitude and period characteristics of the ground motion and on the structural damping, β, of the structure. The response of the mass of the structure can be described either in terms of displacement, acceleration or velocity time histories. The single mass structure represents a very simple structure, however, it can be used to represent each mode of vibration of a multi-mass system such as a multi-story building.

A series of structures with varying periods will each respond differently to an earthquake ground motion. The maximum response acceleration of each structure is determined to form a plot called a response spectrum. This curve can then be used to calculate the maximum acceleration of any multi-degree-of-freedom system responding to the expected site ground motion. A multi-story building can be analyzed elastically to determine the various modes of vibration. Through the use of an elastic modal analysis and the participation factors, the maximum response for each component of the structural framing can be calculated and the design forces can be determined by adding the effects of each mode of the structure. Maximum story displacements, accelerations, forces, shears, and overturning moments can be approximated for each mode. The modal values can be combined by various random variable techniques such as, the Square Root of the Sum of the Squares (SRSS) and the Complete Quadratic Combination (CQC), to establish design forces and displacements.

The effects of various soil conditions at the site change the shape of the response spectrum at the site of the building. The S-factor in the code is used to adjust for these conditions. At a soft site (e.g., loose soil profile) the response will be greater for the building design, certainly with the longer periods. On a harder site (e.g. a dense granular soil or a soft rock site) the response will be lower for building design.

The seismic forces used in the design of buildings by the New York City Building Code are substantially less than the forces associated with maximum possible earthquake capable of happening in this area. If a structure were designed to remain totally elastic during such a maximum earthquake, the forces could be greater than the code prescribed lateral forces. However, because of the objective of preventing building collapse and accepting some building damage, building structures are not expected nor are they designed to remain elastic for the largest such earthquake at the building site. The excessive demands of such earthquakes on average can be resisted by framing ductility, redundancy and other nonlinear effects of the structural response.

3.2.3 Capacities of Structures to Resist Major Earthquakes

The explanation of the differences between the large demands of a major earthquake and the relatively lower design loads in the code is of further interest. Some of the differences may be explained by means of load factors and reduction factors (ϕ), or differences between average strengths of framing being greater than specified minimum strengths. Further differences can be accounted for by oversized members and reserves from unused gravity load capacities. After these adjustments, the ratio of the major earthquake demands and the design yield capacities can still be a substantially high factor. Redundancy and ductility are required. If the structure can respond in an inelastic ductile manner without collapsing, the effective demands of the earthquake will be reduced. The effective period lengthens, damping increases, and the amplification due to resonance decreases.

3.3 Seismic Design of Nonstructural Components, Elements of Structures and Non-Building Structures

The New York City Seismic Code provides for the design for seismic forces for all elements of structures and for selected nonstructural components supported on the structure. The seismic force shall be distributed in proportion to the mass distribution of the element or component and shall be considered to act in any horizontal direction.

3.3.1 Nonstructural Components

All nonstructural components and their attachments whose failure in an earthquake would constitute a life-safety hazard must be designed to resist seismic forces as defined by Equation 12-10 (see page 17). Components are permanent assemblies not having a structural function. Components deemed to present a life safety hazard include:

1. Exterior ornamentation and appendages including cornices, ornamental statuaries or similar pieces of ornamentation.
2. Exterior chimneys, stacks, trussed towers, tanks on legs and exterior signs and billboards.
3. Cantilevered parapets.

4. Interior non-loadbearing walls and partitions around vertical exits including offsets and exit passageways.
5. Non-loadbearing partitions and masonry walls in areas of public assembly with an occupancy of greater than 300 people.
6. Interior ornamentation and appendages in areas of public assembly. These items include cornices, ornamental statuaries and other ornamentation.
7. Masonry and concrete fences at grade over 10 feet in height.

The code also requires that all equipment and machinery necessary for life-safety operations be anchored to the structure for a seismic force as defined by Equation 12-10. Included in this category is also equipment which contains a sufficient quantity of potentially explosive or toxic substances which, if released, threatens the safety of the general public. These items include, but are not limited to:

1. Pumps for fire sprinklers
2. Motors and switchgears for sprinkler pumps
3. Transformers
4. Control panels
5. Major conduit ducting and piping serving such equipment and machinery

The code primarily focuses on equipment anchorage and attachments because past earthquakes have demonstrated that most mechanical and electrical equipment is inherently rugged and performs well provided it is properly attached to the structure. However, mechanical equipment components required to maintain containment of flammable or hazardous material should themselves be designed for seismic forces.

The reliability of equipment components after an earthquake can be increased if the following items are also considered:

1. All internal assemblies are attached in a manner that eliminates the potential of impact with other internal assemblies and the equipment wall.
2. Operates, motors, generators, and other such components functionally attached mechanical equipment by means of an operating shaft or mechanism are structurally connected or commonly supported with sufficient rigidity such that binding of the operating shaft will be avoided.

Some large equipment such as boilers or turbines may be constructed within a structure but be isolated from that structure. In such cases, the equipment, necessary for life-safety operations, should be considered as a non-building structure.

The code excludes seismic design for all equipment weighing less than 400 pounds, furniture, temporary equipment and movable equipment. No seismic design is required for access floors, raised floors or hung ceilings.

The interrelationship of components and their effect on each other shall be considered so that the failure of a life safety related component or non-essential nonstructural component shall not cause the failure of another life safety related component.

3.3.1.1 Calculation of Seismic Force (F_p)

Values F_p, the seismic force required to attach a nonstructural component to the structure is calculated based on Equation 12-10. In this equation, the zone factor (Z) and the importance factor (I) are the same factor used in the design of the building structure. Values of C_p are given in Table 23-P. These values have been set considering the results of analyses based on the linear elastic response of multi-story buildings with generally regular configurations. These analyses indicate that for most buildings:

1. The peak acceleration of the upper stories can be three or more times the peak ground acceleration.
2. The peak measured accelerations of the upper stories (as a percentage of gravity) are generally between 1.6 to 2.3 times the peak base shear coefficient.
3. The peak acceleration for the upper half to two-thirds of a building are nearly constant.

These peak accelerations do not occur simultaneously on each floor, but occur at different times since the modes of vibration combine differently for each story. C_p valves are assumed not to vary in accordance with building height leading to a constant seismic force for a given component regardless of its location in the building.

The code only requires that the seismic force (F_p) be considered acting only in horizontal directions. It is held that vertical ground motion effects are adequately covered by gravity load design.

Equipment and components are defined to be either rigid and rigidly supported or non-rigid or flexibly supported. Rigid and rigidly supported components are those having a fundamental period, including their attachment of less than or equal to 0.6 seconds. Non-rigid or flexibly supported equipment has a fundamental period of greater than 0.6 seconds.

Rigid components are assumed to experience the same acceleration as the building structures at the point of attachment. Non-rigid components experience an amplified acceleration. The code provides for a C_p value of two times the value listed in Table 23-P for non-rigid or flexibly supported items where an analysis considering the flexibility of the component and attachment is not performed. If such an analysis is performed, the value of C_p shall not be less than the value listed in Table 23-P.

3.3.1.2 Anchorage of Components

Where design for seismic forces is mandated, positive anchorage to the structure must be provided. For anchored equipment, friction resistance resulting from gravity loads cannot be used to resist seismic forces. However, frictional resistance resulting from seismic overturning forces or from clamping action may be used.

3.3.1.3 References and Standards

The New York City Seismic Code is based on the 1988 UBC code with all supplements through 1990. As such, the code provides a force level for the design at anchorage of nonstructural components.

Considerable development has occurred in seismic codes since the introduction of the 1988 UBC. One trend in this development has been toward incorporation of prescriptive recommendations for detailing of nonstructural components within building structures. The *NEHRP Recommended Provisions for the Seismic Design of New Buildings, 1994* compiles provisions for detailing of nonstructural components.

Following is a partial list of references containing similar guidelines. While compliance with any one of these guidelines does not imply conformance with the New York City Seismic Code force design requirement, these guidelines and standards can be a useful reference:

American Petroleum Institute (API), *API STD 650, Welded Steel Tanks For Oil Storage*, 1993.

American Society of Mechanical Engineers (ASME), *ASME A-17.1, Safety Code For Elevators and Escalators*, 1993.

American Society of Mechanical Engineers (ASME), *ASME B31, Power Piping-ASME Code For Pressure Piping, B31, An American National Standard*, 1995.

American Society of Mechanical Engineers (ASME), *Boiler and Pressure Vessel Code*, including addendums through 1993.

American Society For Testing and Materials (ASTM), *ASTM C635, Standard Specification For The Manufacturer, Performance, and Testing of Metal Suspension Systems For Acoustical Tile And Lay-in Panel Ceilings*, 1995.

American Society For Testing And Materials (ASTM), *ASTM C636, Standard Practice For Installation Of Metal Ceiling Suspension Systems For Acoustical Tile And Lay-in Panels*, 1992.

American Water Works Association (AWWA), *D100, Welded Steel Tanks For Water Storage*, 1989.

Ceilings and Interior Systems Construction Association (CISCA), *Recommendations for Direct-Hung Acoustical Tile and Lay-In Panel Ceilings, Seismic Zones 0-2*, 1991.

Ceilings and Interior Systems Construction Association (CISCA), *Recommendations for Direct-Hung Acoustical Tile and Lay-In Panel Ceilings, Seismic Zones 3-4*, 1990.

Institute of Electrical and Electronic Engineers (IEEE), *Standard 344, Recommended Practice for Seismic Qualification of Class 1E Equipment for Nuclear Power Generating Stations*, 1987.

Manufacturers, Standardization Society of the Valve and Fitting Industry (MSS), *SP-8, Pipe Hangers and Supports-Materials, Design, and Manufacture*, 1993.

National Fire Protection Association (NFPA*), NFPA-13, Standard for the Installation of Sprinkler System*. 1996.

Rack Manufacturers Institute (RMI), *Specification for the Design, Testing and Utilization of Industrial Steel Storage Racks*, 1990.

Sheet Metal and Air Conditioning Contractors National Association (SMACNA), *HVAC Duct Construction Standards, Metal and Flexible*, 1985.

Sheet Metal and Air Conditioning Contractors National Association (SMACNA*)*, *Rectangular Industrial Duct Construction Standards*, 1980.

Sheet Metal and Air Conditioning Contractors National Association (SMACNA), Sheet Metal Industry Fund of Los Angeles, and Plumbing and Piping Industry Council, *Guidelines for Seismic Restraint of Mechanical Systems and Plumbing Piping Systems*, 1992.

3.3.2 Elements of Structures

All parts and portions of structures and their attachments are required to be designed to resist a seismic force as defined by Equation 12-10. These provisions apply to all load-bearing interior and exterior walls, floor slabs, diaphragms, framing members, and other structural elements.

3.3.3 Non-Building Structures

Non-building structures include all self-supporting structures other than buildings which carry gravity loads and resist the effects of earthquakes. Non-building structures include, but are not limited to:

1. Tanks, vessels and pressurized spheres
2. Silos
3. Chimneys
4. Stacks
5. Trussed towers
6. Cooling towers
7. Inverted pendulum type structures
8. Signs and billboards
9. Bins and hoppers
10. Storage racks supported at grade
11. Amusement structures
12. Monuments

Non-building structures differ from nonstructural components of buildings by having independent foundations. Non-building structures experience ground accelerations unaffected by building structures. Nonstructural components experience accelerations at the point of attachment to the building structure which are amplified from the ground acceleration.

Structures which are not covered by this section include:

1. Transportation related structures including:

    - bridges
    - elevated roadways
    - tunnels

2. Marine Structures

    - piers
    - bulkroads
    - offshore platforms
    - dams

Non-building structures shall be designed for a lateral force as defined by section 2312(d) where the value of R_w is based on Table 23-Q (see page 28). For rigid non-building structures, those with a fundamental period including their anchorage of less than 0.06 seconds, equation 12-12 may be used to calculate the lateral seismic force. In general, R_w values assigned to non-building structures are less than those assigned to building structures. This is because buildings tend to have structural redundancy due to multiple bays and frame lines and curtain nonstructural and non-considered resisting elements which effectively give the building greater damping and strength during strong ground motion response.

3.4 Application to Building Additions and Alterations

The Technical Policy and Procedure Notice (PPN) #4/96 shown in Figure 3-1 and the flow chart in Figure 3-2 address how enlargements and additions to buildings are affected by the local law 17/95.

Local law 17/95 applies to alterations that:

> Enlarge the building area and require new or reinforced foundations.
>
> Enlarge the building area and cost more than 60% of the value of the building. For this specific case, the applicant may solicit a waiver from the commissioner.

Local law 17/95 does not apply to alterations that:

> Do not increase the building area regardless of: cost, change of use, and foundation work.
>
> Enlarge the building area, but do not require new or reinforced foundations, and the cost of the alteration is less than 60% of the building value.

The enlargement and the existing structure can be connected or seismically isolated. If they are isolated, only the new construction has to comply with the local law 17/95. If they are connected and the enlargement is small compared to the existing structure, only the new construction has to comply with the new law. If they are connected and the enlargement is big compared to the existing structure, both have to comply with the new code.

To define what is a big or small enlargement, the seismic shear force and the seismic overturning moment for the enlarged building are compared to the seismic forces of the existing building. If the difference between the forces is less than 20% the enlargement is considered small and only the new construction has to comply with the law. If the increase in forces exceeds 20% the enlargement is considered big and the existing structure and the enlargement have to comply with the law. The code recognizes the hardship of upgrading an existing building and limits the amount of upgrade that has to be done by reducing the seismic forces. Hence, the enlarged building has to be designed for the lesser of:

1 Two thirds of the force generated by the enlarged building but not less than the seismic force of the enlargement alone.
2. Twice the force of the enlargement alone.

The following rules should be used to select the R_w for computing the seismic forces. When the existing structure has an undefined or code disallowed lateral load resisting system, use an R_w=1 that represents the elastic forces generated by the seismic event. When the lateral load resisting system is unreinforced masonry, use R_w=1. When the enlargement and the existing structure have a different lateral load resisting system, use the lowest R_w of the two systems to determine the seismic forces of the enlarged building.

The following examples clarify the intent of the law and are done without calculations and assumed results for brevity and conciseness. Note that there is a strong correlation between building area, weight of the building and seismic forces.

Case 1 controls when the enlargement is much bigger than the existing building. Example: Adding 30,000 square feet to a 10,000 square foot building. The combined 40,000 foot structure has seismic forces (shear or overturning moment) that are more than 20% higher than the forces in the original building. In this case, the resulting building is designed for two thirds the force required for the 40,000 square foot building but not less than the forces required for the 30,000 square foot extension. Case (2) does not control as it requires the enlarged building to be designed for twice the force of the 30,000 square foot extension.

Case 2 controls when the enlargement is smaller than the existing building. Example: Adding an exterior stair tower of 7,000 square feet to 30,000 square foot building. The combined 37,000 square feet have seismic forces (shear or overturning moment) 20% greater than the original structure. In this case, the resulting building is designed for twice the forces from the 7,000 square foot addition. Case (1) does not control as it requires a force that corresponds to two thirds of the 37,000 square foot building that is not less than the force generated by the 7,000 square foot new extension.

In some cases, the new addition is placed on top of an existing building and it may result in a new construction designed seismically on top of an existing building that has no seismic capacity. Example: three stories with a small footprint and framed in steel are added to a ten story unreinforced masonry building with a large footprint. It is assumed that some foundation reinforcing is required. When the seismic moment and shear for the thirteen-story building based on R_w=1 is computed and compared to the ten-story unreinforced masonry building with R_w=1, a force increase smaller than 20% occurs. The addition is seismically designed as a three-story building sitting on top of an existing building with a fixed top. In accordance with the local law, the ten-story masonry building is not upgraded and reinforced, but other sections of the code require that the thirteen-story building must withstand the present day code wind loads.

Note that if, in the above example, the three-story addition is supported on new or reinforced columns that go through the existing building to the foundation, then these columns should be laterally braced and must be designed according to code for a thirteen-story tower.

Technical Policy and Procedure Notice #4/96

Date Issued: March 29, 1996

Subject: Earthquake Code - Local Law #17/95 - Enlargements

Purpose: To provide guidelines to Professional Engineers (P.E.) and Registered Architects (R.A.) in the interpretation and application of Reference Standard RS 9-6, §2312(a)(1) enacted with Local Law #17/95, the Earthquake Code.

Effective: Immediately

Specifics: The Earthquake Code, Local Law #17/95, was promulgated for new construction; it does not affect an alteration where there is no enlargement of the existing structure. When proposed work includes an enlargement of the existing structure, compliance with Local Law #17/95 is required when new foundations are constructed or when existing foundations are reinforced to carry the enlarged portion of the building. In addition, the Commissioner will require that any enlargement in and of itself comply with Reference Standard RS 9-6 when the cost of the enlargement exceeds 60% of the value of the building, pursuant to §27-119 of the Code, within any twelve-month period, unless a waiver is granted by the Commissioner.

When compliance with RS 9-6 is required, consideration must be given to the interface between the existing and enlarged portion of the building. If a seismic joint (separation) is utilized, then only the new construction must comply with RS 9-6.

When no seismic joint is provided, the computed seismic forces of the entire proposed structure, (existing building plus the proposed enlargement), must be compared to the computed forces for the existing portion alone. The forces referred to herein are either base shear or overturning moment. If the increase in any seismic force resulting from the enlargement is less than 20%, compliance with RS 9-6 is required only for the new construction. If either of the seismic forces attributable to the enlargement increase by more than 20%, then the design force shall be the lesser of the following:

1. Two-thirds (2/3) of the combined force resulting from the combination of the new and existing construction but not less than the seismic force resulting from consideration of only the new construction;
2. Twice the seismic force resulting from consideration of only the new construction.

This design force shall be resisted by either the enlargement alone or a combination of the existing building and the enlargement depending on the design professional's choice of load resisting system and force distribution.

Figure 3-1 Technical Policy and Procedure Notice (PPN) #4/96

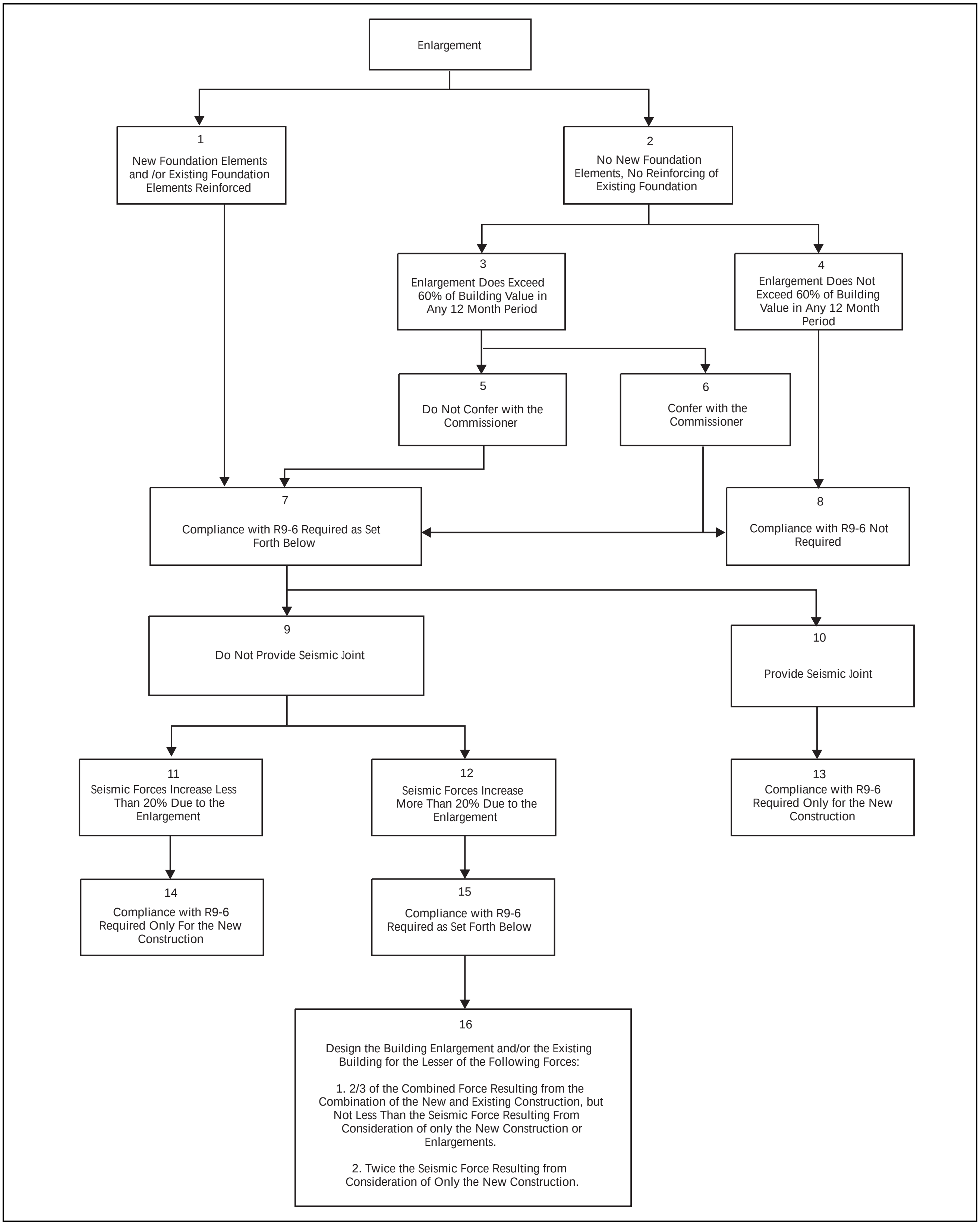

Figure 3-2 Proposed Seismic Regulations for Enlargements

3.5 Economic Impact Study

The following is a summary of an economic implication study of several buildings, located in New York City, based on conformance with New York City seismic regulations, as defined in Local Law 17 of 1995, effective February 21, 1995.

The study includes four basic types of construction; steel, reinforced concrete, masonry and precast. For steel and reinforced concrete construction, buildings heights ranging from low- to high-rise were considered. Masonry and precast construction were studied for low-rise buildings only.

3.5.1 Assumptions

In order to "fairly" compare the seismic impact on the buildings which are already designed for wind forces, the study was conducted based on the premise that the lateral system's configuration was not altered. The required additional weight was added without changing a member's depth or adding members, since adding a new member or increasing the depth of the existing member may intrude into architectural or functional space requirements. The review of those impacts are outside the scope of this study.

For this study, only the static lateral force method of the Code is used. The additional cost required for seismic conformance is evaluated as a percentage of the basic cost of the structural system only.

3.5.2 Results

A brief description of each type of building studied is given in the following sections. Table 3-1 summarizes the results of the study including the height, type of building and underlying soil type. It should be noted that the cost impacts were obtained for buildings where the lateral systems were selected solely based on wind criteria. However, if the structures were designed with wind as well as seismic criteria in mind from the conceptual phase of the project, the cost increases would most likely be lower than the reported values. Also, it should be mentioned that none of these buildings were highly sensitive to torsional effects, and as a result, the cost implications for torsional sensitivity or any other irregularity in general may not be reflected in these studies.

3.5.3 Conclusion

This study was limited to the so called “seismically simple buildings” which means that no set backs, soft stories, or any other type of irregularities so frequently in evidence in New York City were considered. As a result, the estimated additional cost impact does not reflect the effects of these irregularities.

Conversely, it should be emphasized that this research was done with regard to design of "yesterday's buildings" for "tomorrow's forces." In other words, if seismic and wind criteria are implemented from inception of the project and as the Architectural/Developer group became familiar with the cost implications of irregularities, etc., most likely the cost impact would be somewhat lessened although it would still exist.

Since there are numerous parameters and criteria involved in shaping the outcome of the study, the tabulated results should not be taken as "definitive". Variations from this mean can easily run ± 50%. Indeed, the study of a 3-story bearing wall project on the direct substitution basis proved impossible economically and was abandoned.

Based on the limited study of this three-story masonry building, it would appear that for six- to ten-story masonry buildings, the cost increment will be in the range of 15%.

3.5.4 Summary of the Study

A. Steel structures:

1. 2-Story Building

   Two industrial buildings are studied. Building 1A has dimensions of approximately 150' x 194' x 38'. The lateral load resisting system in short direction is a series of one bay Ordinary Moment Resisting Frames, R_w=6. In the long direction it is a Braced Frame, R_w=8. Soil types of S_2 and S_4 were considered. The estimated cost increases are about 11.4% and 15.8% for S_2 and S_4, respectively. The ceiling value of 2.75 for "C" was reached for S_4.

   Building 1B is a 2-story structure with overall dimensions of 127 x 82 x 28'. The lateral system in the short direction is a Moment Resisting Frame, and in the long direction is a Concentric Braced Frame. The wind system is designed to resist a lateral wind pressure of 44 psf. The seismic study indicated that there is no need for additional material for either of the soil types S_2 or S_4.

2. 5-Story Building

   A 5-story office building utilizing an Ordinary Moment Resisting Frame in both lateral directions, R_w=6. The estimated cost increases are about 8%, 12%, and 15% for soil types S_0, S_1, and S_2, respectively.

3. 11- Story Building

   An 11-story office building utilizing a dual lateral force resisting system comprising a Concentric Braced Frame and Special Moment Resisting Frame in both directions, R_w=10. The estimated cost increases are about 8% and 12% for soil types S_1 and S_3, respectively.

4. 25-Story Building

   The lateral force resisting system in both directions is Concentric Braced Frame, R_w=8. The estimated cost increases are about 2.5% and 5% for soil types S_1 and S_3, respectively.

5. 32-Story Building

   In one direction, Ordinary Moment Resisting Frame with R_w=6 is used, and in the perpendicular direction, Concentric Braced Frames with R_w=8 are utilized. The estimated cost increases are about 1.6% and 5% for soil types S_1 and S_3, respectively.

B. Concrete Structures:

1. 3-Story Building

A 3-story hospital is studied. The building dimension is 100' x 125'. The structural system comprises a 9" Flat Slab with 20" square columns a 25' o.c. grid with 34" x 34" x 7" drop panels. This is considered as an Intermediate Moment Resisting Frame system with R_w=7. The required additional materials are about 1.3 psf of rebars for soil types S_1 and S_4, respectively, which may be translated to an additional cost of 4.3% and 9% for S_1 and S_4, respectively.

2. 15-Story Building

The lateral force resisting system is a combination of shear wall core and perimeter Ordinary Moment Resisting Frame in both directions, R_w=5. The estimated cost increases are about 8% and 11% for soil types S_1 and S_3, respectively.

3. 24-Story Building

The lateral force resisting system is a combination of shear wall core and perimeter Ordinary Moment Resisting Frame in both directions, R_w=5. The estimated cost increases are about 5% and 9% for soil types S_1 and S_3, respectively.

4. 57-Story Building

The wind design was controlled by the wind tunnel test results. The lateral force resisting system is a combination of shear wall core and flat slab. The flat slab was considered both as an Ordinary Moment Resisting Frame with R_w=5 and alternately as an Intermediate Moment Resisting Frame with R_w=9. The minimum cost increase of the two alternate framing of R_w=5, and R_w=9 is reported as follows. The estimated cost increase are about 3.2% with R_w=5 for S_0 soil, 4.8% with R_w=5 for S_1 soil, 5.8% with R_w=9 for S_2 soil, and 5.8 for R_w=9 for S_3 soil. The lateral system with R_w=9 and soil S_3 is controlled by the floor limit of .075 x R_w for parameter "C". Therefore, the floor limit would be the controlling value for all the soil types stronger than S_3, such as S_2, S_1 and S_0.

C. Masonry Structures:

1. 3-Story Building

A 3-story nursing home is studied. The floor system is made of 8" precast plank plus 3" topping. Eight inch solid concrete block is placed perpendicular to the exterior walls and spaced at 27 feet on center with openings for the central corridor. The 8" unreinforced solid concrete block is adequate for both gravity and wind loadings. Two different approaches are followed for seismic design of the long direction (parallel to the corridor) of the building. First approach: design the cross walls in the weak axis as a frame in conjunction with slab to resist the lateral forces. This requires thickening the masonry blocks to 12" and adding vertical reinforcing at an average of #4 @ 10'-0" #4 @ 16" for soil S_1 or #6 @16 for soil S_4. Also, plank joint reinforcement should increase accordingly. This yields an economically unacceptable solution. In a second approach, a 9 foot long section of exterior wall (between windows) for each 27 foot length of building is used with 8" reinforced concrete block to resist the seismic forces. Thus, the interior bearing wall reverted to its original design (8" solid concrete block). However, vertical reinforcing is required, at a minimum of 1 #4 @ 10'-0". The 9 foot long exterior walls also require vertical reinforcing at an average of 4 #6 for soil type S_1, and 6 #6 for soil type S_4. The estimated cost increases are about 9.7% for soil type S_1 and 11.3% for soil type S_4.

D. Precast Structures:

1. A 2-story parking structure is studied. Lateral forces are resisted by shear walls in both directions. Soil types of S_2 and S_4 are considered. The ceiling value of 2.75 for "C" is reached for both types of soils. The seismic force is 4 to 5 times higher than wind force. The cost difference is in wall reinforcement (minimal increase above normal) as well as increase in shear wall foundation, which is estimated as about a 3% cost increase for both types of soils.

2. 8-Story Building

   An 8-story parking structure is studied. Lateral forces are resisted by shear walls in both directions. Only soil type S_3 is considered. The C value was close to the maximum value of 2.75. Seismic force is about 5 times larger than the wind force. The cost difference would be in wall reinforcement as well as increase in shear wall foundations, which is estimated as about a 3% cost increase for soil type S_3 as well as S_4 since "C" value was close to maximum value of "C".

3.5.5 Acknowledgments

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Table 3-1 Summary of Cost Impacts (Ratio of Structural Cost Only)

Building Type	Lateral Load Resisting System	Number of Floors	Soil Condition S_0	S_1	S_2	S_3	S_4
S1A Steel	OMRF, R_W=6 CBF, R_W=8	2			1.11		1.16
S1B Steel[1]	OMRF, R_W=6 CBF, R_W=8	2			1.00		1.00
S2 Steel	OMRF, R_W=6	5	1.08	1.12	1.15		
S3 Steel	CBF + SMRF R_W=10	11		1.08		1.12	
S4 Steel	CBF, R_W=8	25		1.03		1.05	
S5 Steel	OMRF, R_W=6 CBF, R_W=8	32		1.02		1.05	
C1 Concrete	IMRF, R_W=7	3		1.04			1.09
C2 Concrete	Shear Wall+ OMRF, R_W=5	15		1.08			1.11
C3 Concrete	Shear Wall+ OMRF, R_W=5	24		1.05			1.09
C4A Concrete	Shear Wall+ Flat Slab	57	R_W=5 1.03	R_W=5 1.05	R_W=9 1.06	R_W=9 1.06	
C4B Concrete[2]	Shear Wall+ Flat Slab	57	R_W=5 1.01	R_W=9 1.03	R_W=9 1.04	R_W=9 1.06	
M1 Masonry	Reinforced Masonry Wall R_W=5	3		1.10			1.11
P1 Precast	Shear Wall R_W=6	2			1.03		1.03
P2 Precast	Shear Wall R_W=6	8				1.03	1.03

Note: The base value of 1.00 is for non-seismic design structure.

1 Designed for twice the New York City wind pressure.

2 C min = 0.05 x R_W.

Section 4
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