

Seismic Rehabilitation of Concrete Structures

Editors

Gajanan M. Sabnis
Avanti C. Shroff
Lawrence F. Kahn



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PREFACE

This volume brings out a wide spectrum of the state of research and practice in the rehabilitation of concrete structures for improved earthquake resistance from the symposium.

The symposium on Seismic Rehabilitation was held at the ACI Spring Convention in San Francisco, California, Spring 1994. Although the main session was organized by ACI 364, in cooperation with ACI 369, there were other sessions at which papers on seismic rehabilitation were presented. This volume includes papers from the main session, other sessions, and some that could not be presented due to lack of available time.

There were over 20 abstracts submitted in the main session, of which only eight could be presented. However, it was agreed to invite all the authors of these abstracts to develop papers for this volume. All papers were peer-reviewed according to ACI practice.

This volume covers many topics related to seismic rehabilitation in research as well as practice, and to elements of structures as well as the whole structures. We thank all authors and reviewers, as members of both ACI 364 and 369, as well as others who helped tremendously with their reviews to make this volume possible.

Finally, we acknowledge the help of ACI staff, who brought all efforts of the committee work to fruition with the publication of this volume.

Gajanan M. Sabnis, Chairman
Avanti C. Shroff
Lawrence F. Kahn
Symposium Subcommittee
ACI 364, Rehabilitation

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Review of Current and Proposed Seismic Design Provisions

by Jerome S. B. Iffland and Avanti C. Shroff

Synopsis: This paper summarizes the findings of a research project performed by the American Iron and Steel Institute entitled "Technical Review of Current and Proposed Seismic Design Provisions." In the last several years, both New York City and the Building Officials and Code Administrators International (BOCA) have proposed and drafted seismic provisions for their respective building codes. The purpose of this study was to compare the pertinent provisions of these proposed provisions to each other and to existing building codes (UBC, SEAOC, NEHRP, and ASCE 7). In addition, the American Institute of Steel Construction has adapted seismic provisions in their Load and Resistance Factor Design Specifications. BOCA has proposed a method of adapting these LRFD Seismic provisions into its building code. This study also reviewed these proposed revisions with respect to applicability to East Coast earthquakes, especially in New York City. The impact of both the NYC and BOCA provisions on design and construction costs are also addressed.

Keywords: Building codes; costs; earthquake-resistant structures; lateral pressure; loads (forces)

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Jerome S.B. Iffland was a founding principal and Chairman of Iffland Kavanagh Waterbury, P.C., an architectural-engineering firm based in New York City. Mr. Iffland was very active in professional societies, having served as director and officer of several societies and chairman of several prestigious committees. He was the author of over fifty technical publications in structural engineering, structural stability, longspan roof structures, slurry walls and folded plate structures and was a frequent lecturer on these topics in the United States and abroad.

ACI fellow Avanti C. Shroff is senior vice president and head of the structural division of Iffland Kavanagh Waterbury, P.C., New York, New York. He has published numerous papers on inspections and rehabilitation of buildings and bridges. He is a member of ACI Committee 437, Strength Evaluation of Existing Concrete Structures, and Chairman of ACI Committee 364, Rehabilitation.

1. INTRODUCTION

After the March 10, 1933 Long Beach, California earthquake, The City of Los Angeles introduced seismic provisions into its 1933 Building Code. These provisions, consisting of two paragraphs, were probably the first seismic building code provisions in the United States (1). The Los Angeles Building Code seismic provisions were subsequently included in the Uniform Building Code (UBC), which is a model code initiated in 1927 by the International Conference of Building Officials. A Joint Committee of the San Francisco Section of the ASCE and the Structural Engineers Association of Northern California drafted a lateral force provisions for building codes which was published in 1952 (2). While seismic provisions in these building codes grew over the years, the publication of the "Recommended Lateral Force Requirements and Commentary," by the Seismology Committee of the Structural Engineers Association of California (SEAOC) in 1967 was a major turning point and this document contributed substantially to future building code seismic provisions (3). This document, termed the "Blue Book" has subsequently been utilized for changes and revisions to the UBC document. Up into the early 1970's, the L.A. Code, the UBC, and the "Blue Book" were the major source of building code seismic provisions for the entire country.

A "Disaster Preparedness Study," published in 1972 by the U.S. Office of Emergency Preparedness, led to an August, 1972 "National Workshop on Building Practices for Disaster Mitigation" sponsored by The National Science Foundation (NSF) and The National Bureau of Standards (NBS). This workshop led to preparation of "Tentative Provisions for the Development of Seismic Regulations for Buildings" (4) prepared by the Applied Technology

Council (ATC) for NSF and NBS. This document, known as ATC-3-06, presented the state-of-the-art of earthquake engineering up to the mid 1970's. Unlike previous work, it addressed seismic building code provisions throughout the entire nation. This is still a major source document for building code writers.

The next major impetus to seismic provisions in codes was the 1977 enactment of the National Earthquake Hazards Reduction Program (NEHRP). This program is now in a current Five Year Plan for FY 1989-1993. This program is administered by the Federal Emergency Management Agency (FEMA), the U.S. Geological Survey (USGS), the NSF, and the NBS, now redesignated as the National Institute of Standards and Technology (NIST). This program led to the initiation and development of the NEHRP Recommended Provisions prepared by the Building Seismic Safety Council (BSSC) in 1986 and subsequently revised in 1988. The NEHRP also resulted in establishment of the National Center for Earthquake Engineering Research (NCEER) at the State University of New York-Buffalo.

NCEER has, and is, taking an active role in promulgating seismic provisions in building codes. The fact that New York City is considering seismic provisions for its Building Code is no accident. NCEER's strategy specifically targeted New York City (5), and as part of this strategy, organized a conference in Sterling Forest in 1987 (6) and participated in organizing one in New York City with The New York Academy of Sciences in 1989 (7) with the express purpose of influencing and promoting seismic provisions in The New York City Building Code. This activity sponsored by the NCEER, and the publicity resulting from an October 25, 1988 earthquake in Quebec, Canada and the December, 1988 Armenian earthquake, substantially aided The New York City effort. Relentless media attention on the Armenian earthquake with parallels drawn to U.S. cities, a well alerted audience at a hearing at the U.S. Embassy to The United Nations, and a study estimating damages in New York City for a moderately large earthquake (8) of 25 Billion U.S. Dollars, triggered The NYC Building Commissioner to establish an expert committee to draft seismic provisions for The NYC Code.

Publication of the ATC-3-06 report and the subsequent NEHRP provisions has stimulated considerable activity in the last few years in revising the seismic provisions in other building codes. This is mainly due to substantial differences in the approach of these two documents compared to the UBC provisions. A list and a brief description of other building codes in approximate historic order of their development that are reviewed in this report follows:

1. UBC (9):

UBC is a Model Building Code and is intended for nationwide use. However, it is used primarily in the West. The UBC document traditionally incorporates SEAOC provisions with some modifications. It

changes the local character of SEAOC to a national character by covering the areas of moderate seismicity (zone specific detailing requirements) by providing a map to cover the entire U.S.A. (The supplement has subsequently been adopted as the 1991 UBC).

2. SEAOC (2):

The SEAOC document addresses the various zones, structural systems, lateral load resisting systems and the detailing requirements. Because it is prepared by California structural engineers, it is revised very quickly after each seismic event to address possible or real problems in the California area and to incorporate new thinking and ideas. It does not address national seismic issues. (A 1991 edition of the Blue Book has been published).

3. NEHRP (11):

The NEHRP document addresses the needs of the different earthquake zones in the United States. The NEHRP document defines not only zone specific lateral forces, but it also defines zone-specific methods of analysis and zone specific detailing requirements.

4. NYC (12):

The NYC proposed code document addresses the various zones, structural systems, lateral load resisting systems and the detailing requirements. It is based on the Uniform Building Code with modifications for local conditions.

5. BOCA (13):

BOCA is a Model Building Code used mainly in the Eastern part of the U.S.A. The proposed 1991 BOCA draft now covers a comprehensive seismic provision in Section 1113.0. This proposed draft is based upon the NEHRP 1988 document which addresses the different earthquake zones in the United States. The BOCA draft defines seismic performance categories, each one depending on both the seismic hazard exposure group and a value of A_v . The BOCA draft not only defines zone specific lateral forces, but also defines zone specific methods of analysis and zone specific detailing requirements.

6. ASCE 7 (Formerly ANSI A58-1 - 1982) (14):

ASCE 7 (Formerly ANSI A58.1) is a reference document for building codes and is applicable nationwide. The proposed draft of Section 9 presents the earthquake provisions. The provisions have been adapted from the NEHRP 1988 provisions.

7. AISC - LRFD (15):

The seismic design provisions for Load and Resistance Factor Design Specifications for Structural Steel Buildings, American Institute of Steel

Construction are reviewed and compared to the appropriate sections of the other six codes being reviewed.

2. SCOPE

Both New York City and the Building Officials and Code Administrators International, have proposed and drafted seismic provisions for their respective Building Codes. The purpose of this study is to compare the pertinent provisions of these proposed provisions to each other and to other existing building codes or code reference documents covering seismic designs. In addition, the American Institute of Steel Construction has adapted seismic provisions in their Load and Resistance Factor Design Specifications. BOCA has proposed a method of adapting these LRFD Seismic provisions into its building code. This report reviews these proposed revisions with respect to applicability to the NYC Seismic Provisions. The impact of both the NYC and BOCA provisions on design and construction costs are addressed. This Report also addresses issues and possible future direction.

This comparison is hindered by the problem of comparing moving targets. The NYC Seismic Code has been under constant revision during this study and the committee preparing the draft did not forward it to the commissioner of the Department of Buildings until April 18, 1991. It may, and probably will, go through subsequent changes before it is approved. Similarly the proposed changes to the BOCA National Code have been undergoing continuous revisions. The drafters prepared a final version for public hearing at the April 8-13, 1991 Spring meeting. Final consideration is not until September 15-20, 1991. Additional changes can be anticipated. The UBC seismic provisions were also being changed during the process of writing this report. The supplement to the 1988 Code has been incorporated into the 1991 UBC. The 1991 SEAOC Blue Book corresponds to this version. At this writing, NEHRP, ASCE 7, and the AISC Specifications are all in the process of undergoing revisions. None of these revisions will be available this year (1991), so comparisons are based on the current published proposed versions that are available.

3. RELATIONSHIP AMONG CODES

Earthquake provisions in all code documents are related because the committee that writes one set of provisions is familiar with other existing earthquake codes and quite often holds joint committee memberships.

The 1988 Edition of UBC is basically the same as SEAOC. A few sections differ, e.g. the P-Delta requirements and some SEAOC sections are not incorporated in UBC, e.g., special inspection of wood construction. The UBC 1991 has been published incorporating 1989 supplements and proposed 1990 revisions.

SEAOC is based on the previous edition of SEAOC, ATC 3-06, and the 1985 Edition NEHRP. It includes expanded provisions for both analysis and detailing. Essentially new sections are the configuration requirements, the rules for selecting a lateral force procedure (including definitions of irregular structures), the dynamic lateral force procedure, and the steel detailing chapter.

The 1988 Edition of the NEHRP provisions represents a major product of the Building Seismic Safety Council's Multiyear, Multitask Program on Improved Seismic Safety Provisions.

The NYC Seismic Code Committee draft is essentially based on the UBC 1988, the 1989 Supplements, and the proposed draft of 1990 supplements. (Now published as UBC 1991).

A few of the NYC sections that differ are:

1. The equivalent static and dynamic analysis loading functions to properly account for the nature of NYC seismic hazard.
2. Extended and modified soils classifications and factors.
3. New provision to assess potential for soil liquefaction.
4. Revised occupancy classifications, and associated design requirements are included.
5. An extended list of qualified structural systems is provided.
6. Adapted and adopted provisions for material detailing including:
 - a. NFPA provisions for wood construction.
 - b. ACI 530-88/ASCE 5-88 and ACI 530.1-88/ASCE 6-88 Requirements for Masonry Structures to replace existing NYC Building Laws Reference Standards RS10-1 and RS10-2. This will apply to all masonry construction.
 - c. ACI 318-89 Requirements for Reinforced Concrete.
 - d. UBC Chapter 27 requirements for steel structures.

The proposed draft of BOCA 1991 is primarily based on the NEHRP 1988 Edition with some minor revision, e.g.; the reinforced concrete section is new and is based on ACI 318-89; the Steel Section reflects the 1989 AISC ASD and LRFD specifications; and the Masonry and Wood Sections slightly differ from the NEHRP provisions.

The proposed draft of Section 9 of ASCE 7 is based on the 1988 Edition of NEHRP provisions.

3.1 COMPARISON OF LATERAL FORCES

UBC, SEAOC, NEHRP, NYC, BOCA, and ASCE 7 use the terminology "for the effects of wind and earthquake". "Loads" is used for gravity effects (dead, live, snow). AISC-LRFD refers to wind and earthquake directly as loads.

There is an essential difference in determining lateral forces between ASCE 7, BOCA, NEHRP, NYC, SEAOC, and UBC earthquake codes. ASCE 7, BOCA, and NEHRP lateral forces are based on strength (limit state design) while NYC, SEAOC, and UBC internal forces are based on working stress design. This difference is in the level of lateral forces as well as in the load factors applied to the earthquake load. In ASCE 7, BOCA, and NEHRP a load factor for earthquake load equal to 1.0, is given in the section "Combination of Load Effects". SEAOC does not give load factors for general use but it does state in Section I-1, "The minimum design lateral forces prescribed in these recommendations are at a service level (rather than yield or ultimate level)". In Chapter 3 on reinforced concrete, SEAOC modifies the load factors given in ACI 318-83 and prescribes a load factor of 1.4 to be applied to the earthquake forces. Because of the difference in philosophy for lateral force levels, when comparing the base shear of ASCE 7, BOCA, NEHRP with that of the other documents, one must multiply the base shear given in any of these other documents by a factor of 1.4 to 1.5.

Figures 1 through 6 show the base shear coefficient, V/W , as a function of period for three typical framing systems. These figures from Luft (16) are modified to include NYC provisions. A comparison of lateral force requirements in ASCE 7, BOCA, NEHRP, NYC, SEAOC, UBC, and AISC-LRFD Design Specification is given in Table 1. This table, adapted from Luft (16), provides in Line 1, the formula for base shear and summarizes the items that go into it. Below are given some comments related to the lateral force requirements (16).

3.1.1 Period-Dependent Coefficient

The period-dependent numerical coefficient C or C_s , is a function of $T^{-2/3}$ (Line 2, Table 1), where T is the first natural period of vibration. The numerical coefficient in ASCE 7, in BOCA, in NEHRP in NYC, in SEAOC, and in UBC comes from the ATC 3-06 documents. The AISC-LRFD refers to the applicable building code.

All numerical coefficients are derived from smoothed earthquake response spectra. These response spectra have been found to depend on the peak ground displacement, peak ground velocity and peak ground acceleration. All documents place an upper limit on the numerical coefficient C or C_s (Line 3, Table 1). ASCE 7, BOCA, and NEHRP provide two upper limits for C_s ,

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depending on the soil factor and the zone. NYC, SEAOC, and UBC specify an upper limit on C equal to 2.75 and this may be used without regard to soil type or structure period.

All Code Documents and AISC-LRFD place lower limits in C or C_1 . T_u was obtained from the statistical analysis of period measurements of existing buildings. This limit on the maximum period T to be used for computing the base shear complicates the analysis because two levels of base shear must be used to comply with strength and drift provisions. The base shear used for establishing the required strength of structural members must be based on a period that does not exceed $C_1 T_u$, while for story drift determinations, a base shear computed with the calculated fundamental period T of the building may be used.

3.1.2 Zone Factor

A single zone factor Z is used in NYC, SEAOC, and UBC while ASCE 7, BOCA, and NEHRP use two coefficients, A_s and A_v , called the effective peak acceleration and the effective peak velocity rated acceleration (Line 5, Table 1). A_s and A_v enter into the determination of the numerical coefficient C_s (Line 2 and 3, Table 1). At most localities, the numerical values of A_s and A_v are the same.

The zones defined in SEAOC apply only to California. The Seismic Zone Map for California in SEAOC includes a Zone 2. UBC has no Zone 2 in California.

The zoning map included in NYC and UBC are based on the map published in the ATC 3-06 document, which in turn, is based on maps published by Algermissen and Perkins in the 1976 US Geological Survey open file Report 76-416 (17). The peak acceleration shown on any point of the Algermissen and Perkins map has a probability of 90% of not being exceeded in a period of 50 years. The zoning maps included in ASCE 7 and BOCA are based on the NEHRP provisions. The zoning factor proposed in NYC has been adopted from UBC 1988. In the NEHRP provisions, a new set of zoning maps are included in the Appendix to Chapter 1. AISC-LRFD refers to the Applicable Building Code.

3.1.3 Importance Factor

The importance of a building for post earthquake recovery is handled in NYC, SEAOC and UBC by an importance factor I and in ASCE 7, BOCA, and NEHRP by the seismic hazard exposure group. The importance factor I is a multiplier within the base shear formula for NYC, SEAOC, and UBC. This means that the importance factor increases the level of base shear applied to essential facilities. In ASCE 7, BOCA, and NEHRP, the seismic hazard

exposure group together with the seismicity index, or the value of A_v , defines the seismic performance category of a building. In ASCE 7, BOCA, and NEHRP the allowable story drift is dependent on the seismic hazard exposure group (a more restrictive drift limit is imposed for higher hazard) while the seismic performance category restricts the analysis procedures, the permissible framing systems and the required detailing. In other words, UBC, SEAOC, and NYC attempt to increase the safety of the essential facility by the level of lateral forces applied to the building while ASCE 7, BOCA, and NEHRP attempt to increase the safety of an essential facility by placing stricter requirements on drift, requiring more detailed analysis, placing restrictions on the type and height of framing to be used, and imposing more restrictive detailing requirements. The Importance Factor for NYC is essentially the same as the UBC requirements. AISC-LRFD refers to the Applicable Building Code.

3.1.4 Structural Framing Factor

The base shear formulas consider the type of structural framing system by a response modification coefficient R in all documents. All documents, have response modification coefficients that derive from ATC 3-06. As previously stated, the base shear in ASCE 7, BOCA and NEHRP is a limit states load, while the base shear in NYC, SEAOC, and UBC is a service level load. The difference between service state and limit state loads shows up in the magnitude of the response modification coefficients given respectively in the code documents.

The NYC code has modified and added several values of response modification coefficients over the UBC code documents. Some of these changes are outlined in Table 2.

AISC-LRFD refers to the Applicable Building Code.

3.1.5 Soil Factor

The definition of Soil Factors in all documents are derived from the soil factor given in ATC 3-06. Following the Mexico City earthquake a soil factor equal to 2.0 was added to the 1988 Editions of NEHRP, UBC, and SEAOC. The factor 1.0 applies to rocks or stiff soil condition, and for stable deposits of sands, gravels or stiff clays of a depth of less than 200 ft. The factor 1.2 applies to deep cohesionless or a stiff clay soil condition exceeding 200 ft. of depth where the soil types overlying rock are stable deposits of sands, gravels or stiff clays. The factor 1.5 applies to soft to medium stiff clay and sands. The factor 2.0 is for a soil profile containing more than 40 ft. of soft clay (UBC 1988) or more than 70 ft. of soft clays or silts (NEHRP).

The definitions for soil factors S_o to S_4 for the NYC code (see Table 3) are different from all documents. They are related to 11 classes of materials (1-65

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through 11-65) as defined by the Unified Soil classification system which is already part of the current NYC Building Code for specifying allowable bearing pressures.

The soil profile definitions in ASCE 7 and BOCA are the same as given in NEHRP.

The in AISC-LFRD refers to the Applicable Building Code.

3.1.6 Weight

The weight to be used in the base shear formula given in Table 1 is in all documents: the total dead load, partition loads, 25% of the live load in storage and warehouse occupancies and in some cases snow load. The UBC and SEAOC documents specify that the snow load be considered only when it is greater than 30 lbs/sq. ft. SEAOC, UBC, and NYC require that the partition load be not less than 10 lbs/sq ft and that all permanent equipment be included in the weight. NYC, SEAOC, and UBC allow the snow load to be reduced up to 75% when permitted by the building officials while ASCE 7, BOCA, and NEHRP states that the effective snow load shall be 70% of the full snow load. ASCE 7, BOCA, and NEHRP also state that where the snow load is less than 30 lbs/sq. ft, the snow load is not required to be included in the seismic loading. NYC code includes 25% live load in parking structures in weight computations.

In base shear formula, AISC-LRFD considers the weight, W . The weight can be calculated based on the various load and load combinations. In AISC-LRFD, separate load and load combinations are given for earthquake loads and are based on the ASCE 7-88 which is the same as ANSI A58.1, 1982.

3.2 COMPARISON OF ANALYSIS PROVISIONS

There are some differences among the code documents. The comparison of analysis provisions for various requirements is summarized in Table 4 (16). In Table 4, AISC-LRFD does not appear, because the AISC-LRFD provision is a design specification and it does not address any analysis provisions except the base shear formula.

3.2.1 Direction of Base Shear

Earthquake codes have traditionally defined the base shear as the total of the horizontal forces assumed to come from any horizontal direction. The lateral seismic forces are assumed to act non-concurrently in the direction of the main axes of the structure. This concept is used in NYC, UBC, and SEAOC codes. Also, these codes contain certain exceptions. Provisions must

be made in Zone 2, 3, and 4 for the effects of earthquake forces acting in a direction other than the principal axes when the structure has a plan irregularity defined as Type A or Type E or when a column of a structure forms a part of two or more intersecting lateral force resisting systems. For these exceptions, orthogonal effects may be incorporated by designing the structural elements for 100% of the prescribed seismic forces in one direction plus 30% of the prescribed forces in the perpendicular direction or alternatively, by using a square root of the sum of the squares approach for combining two orthogonal directions. ASCE 7, BOCA, and NEHRP requirements are based on the seismic performance category. For category B or C buildings, the design seismic force may be applied separately in each of two orthogonal directions, while for Category D or E buildings, structural components and foundations must be designed for 100% of the prescribed lateral forces applied for the one direction plus 30% of the prescribed forces applied for the other direction. In ASCE 7, BOCA, NEHRP, SEAOC, and NYC, when combining orthogonal effects, the combination requiring the maximum component strength must be used.

3.2.2 Vertical Acceleration

SEAOC and UBC prescribe that for Seismic Zones 3 and 4, the vertical component of seismic forces be considered for horizontal cantilever components and for horizontal prestressed components which must be designed using not more than 50% of the dead load. ASCE 7, BOCA, and NEHRP include the effect of vertical earthquake accelerations by increasing or reducing the dead load in the section on combination of load effects. The vertical acceleration is used to increase the dead load or to decrease the resistance to overturning from dead load. Thus, the load factors for dead load are $(1.1 + 0.5 A_v)$ or $(0.9 - 0.5 A_v)$. For the areas of highest seismicity, the load factor for dead load increases from 1.1 to 1.3 over that used in areas of low seismicity. NYC provisions for Zone 2 are similar to SEAOC and UBC.

3.2.3 Determination of Period

The period is determined in all codes by use of empirical formulas or by the results of an elastic dynamic analysis. All code documents have a formula that relates to empirical period to the height of the building raised to a $3/4$ power and multiplied by a coefficient C_t ($T = C_t h^{3/4}$). The coefficient C_t equals 0.035 for steel frames in the UBC and SEAOC codes. For reinforced concrete frames, C_t equal 0.03 in all codes. In ASCE 7, BOCA, and NEHRP an alternate approach is given to determine the approximate fundamental period for concrete and steel Moment Resisting Frame buildings not exceeding 12 stories in height and having a minimum story height of 10 feet. The fundamental period is defined as the number of stories divided by 10 ($T = 0.1N$). Also, in ASCE 7, BOCA, and NEHRP the approximate fundamental period for all other buildings is defined as $T = 0.05 h_n/L^{1/2}$. In ASCE 7,

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BOCA, and NEHRP, the length used for the empirical period calculation is the overall dimension of the building at the base in the direction under consideration. NYC, SEAOC, and UBC, for buildings other than Moment Resisting Frames, use the same expression as for Moment Resisting Frames with C_t equal to 0.02. Alternatively, the value of the C_t for structures with concrete or masonry shear walls may be a $0.1/\sqrt{A_c}$ when the value of $A_c = \Sigma A_c [0.2 + (D_c/h_n)^2]$. The value of $D_c/h_n \leq 0.9$. The NYC Code has a modified value of $C_t = 0.035$ for reinforced concrete Moment Resisting Frames and $C_t = 0.03$ for Eccentric or Concentric Braced Frames and shear wall dual frames where the building height exceeds 160 feet. The empirical period is important in ASCE 7, BOCA, and NEHRP because the period used for base shear determinations may not exceed C_s times the empirical period. In NYC, SEAOC, and UBC, there is an indirect upper limit to the period: the numerical coefficient C determined with a period from Method B cannot be less than 80% of the coefficient C value obtained by using T from Method A. (Lines 2 and 4, Table 1).

3.2.4 Analysis Procedure

Conventional earthquake analysis is performed using a static equivalent system of lateral forces even though programs to perform dynamic analyses have been available for many years. Dynamic analysis is required by all building codes only for irregular structures. UBC allows the static equivalent lateral load analysis for all buildings in Seismic Zone 1, for standard occupancy buildings in Zone 2, for regular structures that are under 240 ft in height, and for irregular buildings not more than 5 stories under 65 ft high (this includes all wood frame buildings). Irregular structures are defined in great detail in SEAOC. The dynamic force procedure must be used for buildings that do not qualify for the equivalent static force procedure. SEAOC requires a dynamic analysis for buildings with period longer than 0.7 second on soil profile Type 4. NYC allows the static equivalent lateral load analysis for all buildings, regular or irregular, in Occupancy IV for all structures that are under 400 ft in height. NYC requires the dynamic force procedure be considered for structures over 400 feet in height. In ASCE 7, BOCA, and NEHRP, the prescribed analysis procedures are dependent on the seismic performance category. For seismic performance Category A, no analysis of the overall building is required. For Categories B and C, the static equivalent analysis procedure may be used for both regular and irregular buildings and for buildings in Categories D and E. The static procedure is also permissible for regular structures. A dynamic analysis must be performed for irregular structures over five stories or 65 ft in height. For buildings in Categories D and E that have only vertical irregularities, a dynamic analysis is required. ASCE 7, BOCA, and NEHRP have no height restriction for applying the static force procedure.

The NYC and UBC codes require that irregular structures be analyzed considering the dynamic characteristics of the structure. NYC, SEAOC, and

UBC have a section on dynamic lateral force procedures. This section defines the ground motions and the acceptable mathematical models, and states that the response spectrum analysis and time history analysis are both acceptable. For response spectrum analysis, Codes require that a sufficient number of modes be considered so that at least 90% of the participating mass of the structure is included in the calculation of response for each principal horizontal direction. Codes do not specify the number of torsional modes. ASCE 7, BOCA, and NEHRP include a section on the modal analysis procedure. This section requires that the modal analysis include at least the three lowest modes of vibration or all modes of vibration with periods greater than 0.4 seconds for each of two mutually perpendicular axes.

3.2.5 Vertical Distribution of Base Shear

When the static analysis procedure is to be used, the base shear must be distributed along the height of the building. This vertical distribution of lateral loads is intended to simulate the combined natural mode shapes of the building. NYC, SEAOC, and UBC divide the base shear into two parts, one part called F_t is applied as a concentrated force at the top of the building and the second part is distributed throughout the height in proportion to the product of the story weight times the height of the story over the base (Line f, Table 4). The linear part of the load distribution models the first natural mode of vibration of the building, while the concentrated force at the top accounts for the higher modes of vibration. ASCE 7, BOCA, and NEHRP have adopted a different vertical distribution of seismic forces. In ASCE 7, BOCA, and NEHRP, the fraction of the base shear to be applied at a story is proportional to the weight of the story times the height of the story above the base raised to an exponent k (Line f, Table 4). The exponent k equals 1 for buildings having a period of 0.5 seconds or less, and equals 2 for buildings having a period of 2.5 seconds or more. An interpolated value of k may be taken for buildings having a period between 0.5 seconds and 2.5 seconds. If such interpolation is not performed, k must be taken equal to 2.0.

3.2.6 Overturning Moment

In developing earthquake codes, vertical distributions of base shear are arrived at by performing modal dynamic analyses of many buildings, superimposing the several modes, and computing the resulting story shears and story overturning moments. A particular vertical distribution of the base shear leads to a good representation of an envelope of the story shear diagram or of an envelope of the overturning moment but not of both. Codes have usually taken the approach that the distribution of base shear should model the story shear envelope. As a result of this choice, the overturning moments along the height of the building are overestimated. The NYC, SEAOC and UBC documents do not permit reductions in overturning moment to account for the overestimate. In ASCE 7, BOCA, and NEHRP, a multiplier κ is applied

to the overturning moment computed at each story. The value of κ is 1.0 for the top ten stories, 0.8 for the twentieth story from the top and below, and a value interpolated between 1.0 and 0.8 for stories in between. The NYC code includes overturning provisions for column design in structures with discontinuous lateral-load-resisting [2312(e)7B, UBC 1988] elements for structures in Occupancy Categories I, II and III only. Also, the NYC code has modified the requirement of 2312 (c) 7B (iii) and C, of UBC.

3.2.7 Drift

Drift requirements, together with the specified base shear, have the largest impact on member sizing, especially in moment frames. Steel structures are more sensitive to drift requirements than reinforced concrete structures. The allowable story drift in ASCE 7, BOCA, and NEHRP depends on the seismic hazard exposure group and on the building height. The allowable story drift for a 5 or more story building is 0.015 times the story height for exposure groups 1 and 2 and 0.01 times the story height for exposure group 3. Higher drift limits are allowed for building 4 stories or less in height without brittle finishes and for all types of single story buildings. The drift requirements in ASCE 7, BOCA, and NEHRP cannot be compared directly with those of NYC and UBC because (a) ASCE 7, BOCA, and NEHRP contain a deflection amplification factor C_d to be applied to the deflections determined by an elastic analysis; (b) the base shear level in ASCE 7, BOCA, and NEHRP is based in a strength design level; (c) the forces used to calculate drift are not constrained by an upper limit of the building period, as are the forces used to size members; and (d) the analysis used to obtain the lateral deflections in ASCE 7, BOCA, and NEHRP must include the P-Delta effect, if applicable. The ratio of C_d to R goes from a low of 0.5 to a high of 1.0; therefore, the deflection amplification factor C_d does not necessarily negate the effect of the response modification factor R as would occur if C_d were equal to $1/R$. ASCE 7, BOCA, and NEHRP do not contain an alternate provision allowing higher drift when it can be demonstrated that greater drift can be tolerated.

In the NYC, SEAOC, and UBC codes, the drift limits depend upon a controlling criteria of fundamental period less than 0.7 sec. The drift limits in NYC, SEAOC, and UBC code may be exceeded when it is demonstrated that greater drift can be tolerated by both structural elements and non-structural elements that could affect life safety. The lateral forces used in SEAOC to compute drift are not restricted by the 80% limitation that applies when the calculated period exceeds the empirical period. In SEAOC, there are no separate deflection amplification factors but rather the limits $0.03/R_w$ and $0.04/R_w$ negate the response modification factor R_w used to compute the base shear. Table 5 provides a comparison of lateral force and drift requirements of seismic codes for steel structures (18).

3.2.8 P-Delta

SEAOC requires that the P-Delta effect be considered only when the drift exceeds $0.02/R_w$ times the story height. The NYC and UBC codes require that P-delta effects be considered when the ratio of secondary moment to primary moment exceeds 0.1. UBC does not require a P-delta analysis if drift limits are met in Seismic Zones 3 and 4. ASCE 7, BOCA, and NEHRP require that the P-delta effects be considered for all buildings that require an overall analysis when the stability coefficient exceeds 0.10. The P-delta effects in ASCE 7, BOCA, and NEHRP are based on linear elastic stability theory. As such, they ensure that the static equivalent lateral forces in combination with gravity loads will result in a stable structure. The approach is valid and consistent with the static equivalent procedure. The ASCE 7, BOCA, and NEHRP approach has been criticized because the P-delta effects are based on elastic deformations as opposed to amplified deformations. During an earthquake, equilibrium in the structure exists between inertial forces, elastic or inelastic member forces, and P-delta forces under amplified deflections. The effect of P-delta under dynamic conditions in buildings that may deform inelastically is complex and not yet at the stage where it can be incorporated by a simple expression in design codes. The linear elastic approach of ASCE 7, BOCA, and NEHRP will probably continue to be used until further information is developed.

3.2.9 Building Separation

There is no specific requirement in ASCE 7, BOCA, and NEHRP for the separation of buildings. However these codes suggests that all portions of structures shall be designed and constructed to act as an integral unit in resisting horizontal forces unless separated structurally by a distance sufficient to avoid contact under deflection from seismic effect.

SEAOC and UBC suggest that all structures shall be separated from the adjoining structures by $3 (R_w/8)$ times the displacement due to seismic forces. This means the response modification factors have an important role regardless of the seismic zone. (i.e. for high seismicity or moderate seismicity areas, the building separation would be the same).

3.3 COMPARISON OF DETAILING REQUIREMENTS

Detailing requirements, also called ductility requirements, are essential to provide structures the ability to deform beyond the elastic limit and to undergo many cycles of extreme stress reversals. NYC, SEAOC, and UBC codes address detailing requirements for steel and concrete in both high and moderate seismicity zones. The ASCE 7, BOCA, and NEHRP provisions contain detailing requirements for Intermediate Ductility Concrete Frames and

requirements for reinforced masonry. For concrete, SEAOC, ASCE 7, and NEHRP code are based on ACI 318-83, while UBC and NYC are based on ACI 318-89. For steel, ASCE 7, BOCA, and NEHRP provisions are based on the AISC-ASD/LRFD, Specifications with only BOCA incorporating AISC-LRFD seismic provisions, and these only for category D and E buildings and for Hazard Exposure Group III with $A_v \geq 1.0$. In this section, the discussion and various provisions is generally limited to the steel detailing requirements.

3.3.1 Reinforced Concrete

All codes contain detailing requirements for reinforced concrete. ASCE 7 and NEHRP base their requirements on ACI 318-83 with modifications given in appropriate chapter. ASCE 7 and NEHRP recognize three types of moment frames; ordinary moment frames, intermediate moment frames, and special moment frames. Ordinary moment frames are designed in accordance with the body of ACI, intermediate moment frames are designed in accordance with Section A.9 of ACI; and special moment frames are designed in accordance with Sections A.2 through A.7 of ACI. Ordinary moment frames may only be used for buildings in seismic performance Category A and for some buildings of seismic performance Category B. Intermediate moment frames may be used in seismic performance Category C, and wherever ordinary frames are permitted. Special moment frames must be used in buildings assigned to Categories D or E and may of course be used for buildings in Categories A to C. When designers have the option of using an ordinary or an intermediate moment frame, they have the option to compare the increased cost associated with (a) higher forces in ordinary frames (Figures 5 and 6), and (b) the more stringent detailing requirements in intermediate frames. Similar comments apply to the choice of intermediate versus special frames.

BOCA bases its requirements on ACI 318-89. BOCA recognizes three types of moment frames; ordinary moment frames, intermediate moment frames and special moment frames. These frames are designed in accordance with the ACI 318-89 code. Ordinary moment frames may only be used for buildings in seismic performance Category A and for some buildings of seismic performance Category B. Intermediate moment frames may be used in seismic performance Category C, and wherever ordinary frames are permitted. Special moment frames must be used in buildings assigned to Categories D or E and may of course be used for buildings in Categories A to C. When designers have the option of using an ordinary or an intermediate moment frame, they must compare the increased cost associated with (a) higher forces in ordinary frames (Figures 5 and 6), and (b) the more stringent detailing requirements in intermediate frames. Similar comments apply to the choice of intermediate versus special frames.

NYC draft follows the UBC and requirements for ACI 318-89.

SEAOB requirements are also based on ACI 318-83 with extensive modifications. SEAOB does not define or allow the use of Ordinary Moment Frames. Intermediate Moment Frames may be used in Seismic Zones 1 and 2 for normal buildings and for certain non-building structures in Seismic Zones 3 and 4.

3.3.2 Steel

Framing systems used in structural steel are Ordinary Moment Frames, Ductile or Special Moment Frames, Concentrically Braced Frames and Eccentrically Braced Frames. Frames containing damping devices, friction plates, and steel plates designed to yield are currently being researched but are still not used widely. Currently, there are no provisions for Intermediate Moment Resisting Frames.

AISC-LRFD Seismic Specification is a comprehensive document providing earthquake provisions for Seismic Zones 3 and 4 and in Zone 2 having an importance factor, I , greater than 1.0. It includes the requirements for Ordinary Moment Frames, Special Moment Frames, and Concentrically and Eccentrically Braced Frames. It also includes drift and building separation requirements. In AISC-LRFD, Ordinary Moment Frames must contain beam-to-column connections that are adequate to develop the strength of the member framing into the connection as specified in the beam-to-column joint requirements for Special Moment Frames unless the connection meets the strength requirements based on the load combinations. Special Moment Frames must meet the requirements of beam-to-column joints that are adequate to develop the flexural strength of the beam or the panel zone shear strength. AISC-LRFD also contains extensive requirements for Concentrically Braced Frames and Eccentrically Braced Frames.

In ASCE 7, BOCA, and NEHRP, the steel detailing requirements depend on the seismic performance category of the building. In seismic performance categories A and B, a building may be of any type of steel construction permitted by AISC, AISI, and SJI. In seismic performance Category C, Ordinary Moment Frames designed in accordance with Part 1 of the AISC-ASD specification may be used. In seismic performance Category D and E, Moment Resisting Frames must be Special Moment Frames conforming to requirements given in the code. Key requirements for Special Moment Frames are; that they be of Type 1 construction, that the axial forces in columns not exceed 0.6 times P_y , and that column splices not be placed in areas in which a plastic hinge may form unless the splice fully develops the column section. Requirements are also given for beam-column joint panel zone areas and beam to column connections.

SEAOB also is a comprehensive document providing earthquake provisions for steel structures in seismic Zones 2, 3 and 4. Based on the definition, an

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Intermediate Ductility Steel Moment Frame may be classified as an Ordinary Moment Frame. This means the requirements for both Ordinary Moment Frames and Intermediate Ductility Steel Moment Frames are the same. SEAOC has provisions for the girder-to-column connection ductility, column-girder joint panel zone, and a strong-column/weak-beam requirement. For Moment Frame drift calculations, SEAOC requires the inclusion of bending and shear contributions from clear column spans, plus the joint rotation resulting from the clear girder span, plus the distortion of the panel zone, plus the effect of column axial stresses when they amount to more than 10% of the total frame drift. SEAOC also contains extensive requirements for Braced Frames and for Eccentrically Braced Frames.

In UBC - 1990 supplement, Ordinary Moment Frames must contain a girder to column connection that is adequate to develop flexure strength of the girder or the panel zone shear strength 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. Special Moment Resisting Frames for Zone 1 and 2, must meet the requirements of Ordinary Moment Frames and the additional requirements on the panel zone, strong-column/weak-beam concept, beam bracing and others. For Moment Frame drift calculations, UBC requires the inclusion of bending and shear contribution from clear column spans, plus the joint rotation resulting from the clear girder spans, plus the distortion of the panel zone, plus the effect of column axial stress when it amounts to more than 10% of total frame drift.

The NYC requirements for Steel Structures Resisting force induced by Earthquake Motions in seismic Zones 2 are very similar to the requirements of UBC with some modifications and editorial changes. NYC sets a maximum limit of 50 ksi yield strength for structural steel. For drift calculations, NYC also requires the inclusion of bending and shear contribution from clear column spans, the joint rotation resulting from the clear girder spans, plus the distortion of the panel zone, plus the effect of column axial stress when it amounts to more than 10% of total frame drift.

The following is a comparative review of the design requirements for steel structures contained in the various code documents and in the AISC-LRFD design specifications. These documents best represent the range of consensus among the proposed seismic documents. The structural systems discussed are Special and Ordinary Moment Resistant Frames, Concentrically and Eccentrically Braced Frames. There are no provisions for Intermediate Moment Frames, however, the Ordinary Moment Frame, as defined by SEAOC may be considered an Intermediate Moment Frame. A brief summary of comparison of lateral force and drift requirements of various seismic documents is given in Table 5.

3.3.2.1 Special Moment Resisting Frames: Recently, Special Moment Resistant Frames (SMRF) have received the greatest attention among earthquake resistant structural steel systems. The detailing requirements have evolved mostly from the results of the research rather than the evidence of earthquake damage. Few multistory steel SMRF's have been built in accordance to modern seismic codes. Steel framed buildings performed well during the recent Loma Prieta earthquake in California.

Beam-to-Column Joints: ASCE 7, and NEHRP do not clearly distinguish the beam-to-column joints requirements but they require that shear in frame beams and columns and their connections shall be determined on the basis of dead and live loads acting in combination with a moment equal to the member flexural strength at critical sections. However, these shears shall not be less than the shears resulting from elastic distribution of the specified forces. BOCA refers to AISC-LRFD Seismic Specification for Category D and E buildings with some modifications.

AISC-LRFD requires that:

- (a) The required flexural strength, M_u , shall be the lesser of the (i) plastic bending moment, M_p , or (ii) the moment resulting from the panel zone nominal shear strength V_n , where:

$$\phi_v V_n = 0.55 \phi_v F_y d_c t_p \left[1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_p} \right]$$

- (b) The required shear strength, V_u , shall be based on the load combination $1.2D + (0.5L \text{ or } 0.2S)$ plus the shear resulting from M_u .
- (c) The design strength ϕR_n of a beam-to-column joint may be considered to be adequate to develop the required flexural strength of the beam if the LRFD criteria have been met and
- (d) The design strength of the joint shall be 125 percent of design strength of connecting elements when welds or high strength bolts are used.

NYC, SEAOC and UBC have somewhat similar requirements to ASCE 7, and NEHRP. The flange detail limitations and joint configurations are essentially similar to ASCE 7, and NEHRP in calculating shear strength (Gravity loads plus $3(R_w/8)$ times the girder (beam) shear resulting from the prescribed seismic forces).

Panel Zone: ASCE 7, and NEHRP require that the panel zone area shall be designed to resist the shear, f_v , based on the capacity of the members framing into the joint, but need not exceed shears produced by deforming

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AISC-LRFD requires that the beam-to-column connection must satisfy one of the following requirements:

$$\frac{\sum Z_c (F_{yc} - \frac{P_{uc}}{A_g})}{\sum Z_b F_{yb}} \geq 1.0 \quad \text{where } P_{uc}(\text{in comp}) \geq 0,$$

or

$$\frac{\sum Z_c (F_{yc} - \frac{P_{uc}}{A_g})}{V_a d_b [\frac{H}{(H-d_b)}]} \geq 1.0 \quad \text{where } P_{uc}(\text{in comp}) \geq 0.$$

The above requirements need not apply:

- (a) For column with $P_{uc} \leq 0.3 F_y A_g$,
- (b) For columns in any story that has a total design lateral shear strength 50% greater than that of story above and
- (c) For any columns whose design shear strength is not included in the design to resist the required seismic shear.

NYC, SEAOC and UBC require that moment frame joints must satisfy the following:

$$\frac{\sum Z_c (F_{yc} - f_s)}{\sum Z_b F_{yb}} > 1.0$$

or

$$\frac{\sum Z_c (F_{yc} - f_s)}{1.25 \sum M_{ps}} > 1.0 \quad \text{where } f_s > 0$$

The above requirement need not apply:

- (a) For columns with f_s less than $0.4 f_y$ for all load combinations,
- (b) For columns in any story which has lateral shear strength 50% greater than that of the story above and
- (c) For any column whose lateral shear strength is not included in the design to resist code required seismic shear.

In addition to above requirements, AISC-LRFD has two additional requirements covering restrained and unrestrained connections.

These requirements are Beam-to-Column Connection Restraint and Lateral Support of Beams. NYC, SEAOC and UBC require five additional requirements and these are:

- (i) Trusses in SMRSF,
- (ii) Girder-Column Joint Restraint,
- (iii) Flange Width-thickness Ratio,
- (iv) Beam Bracing, and
- (v) Moment Frame Drift Calculations.

ASCE 7, and NEHRP do not mention any of above requirements, however they do require the compliance to AISC-ASD/LRFD the Design Specifications.

3.3.2.2 Ordinary Moment Frames: ASCE 7, and NEHRP require that when OMF are used, the proportioning members and their connection must be based on the requirements of AISC-ASD/LRFD Design Specification.

AISC-LRFD requires that OMF shall have the design strength to resist the factored load and load combinations. The design strength must be based on the LRFD specifications. It must also meet the joint requirement:

- (a) Beam-to-Column joints requirements are similar to SMF.
- (b) The design strengths of the connection shall be based on the load combinations and code required seismic load combinations. AISC-LRFD permits the use of FR (fully restrained) and PR (partially restrained) connections.

NYC, SEAOC and UBC require that the Girder-to-Column connections of OMF shall meet the requirements of Girder-to-Column connection of SMRF unless it has a capacity to resist the combination of gravity load and $3(R_w/8)$ times the design seismic forces.

3.3.2.3 Concentrically Braced (CBF) and Dual System Buildings: ASCE 7, and NEHRP requirements are based on the AISC-ASD/LRFD Design Specifications. When bracing is provided to control drift, the beams and columns incorporating the bracing shall be designed to carry the brace forces in accordance to the AISC-ASD/LRFD Design Specifications in addition to the appropriate forces from the gravity and seismic loads. Briefly the provisions include the following requirements:

(a) Bracing Member:

- (i) The tensile strength shall equal the tensile yield strength and the compression strength shall equal the member's buckling strength.
- (ii) The lateral force capacity of bracing shall not exceed the capacity in the reverse direction by more than 10%.

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Beams with bracing members connected at location not at the beams ends shall be designed for the gravity forces.

(c) Columns:

The column design shall be based on the gravity loads plus maximum forces received from the braces and the moment capacity of the associated beams.

(d) Bracing Connections:

In Seismic Performance Categories D and E, connections shall be designed to develop the full tensile yield capacity of the member.

The AISC-LRFD provisions require that the brace member of CBF be sized for axial force resulting from the design seismic lateral load. Briefly the provision includes:

(a) Bracing Members:

- (i) A limit of $720/\sqrt{F_y}$, slenderness ratio L/r .
- (ii) Compression strength $\leq 0.8 \phi_c P_n$.
- (iii) In Zones 3 & 4 the Horizontal Components of seismic force in tension or in compression $\leq 0.7 \times$ total force.
- (iv) Width-thickness ratios must meet the LRFD specification
- (v) Built-up member requirements must meet the LRFD specification.

(b) Bracing Connections:

The brace connection shall develop a tensile strength equal to the load combination given in the AISC-LRFD Specifications and the maximum force that can be transferred to the brace by the system.

(c) Special Bracing Configuration Requirements:

- (i) V-Bracing shall be designed 1.5 times the required strength.
- (ii) K-Bracing: K-Bracing is prohibited in Seismic Zone 3 and 4. For other seismic zones, K-Bracing requirements are similar to V-bracing.

NYC, SEAOC and UBC requirements are similar to the AISC-LRFD requirements. Briefly the provisions includes:

(a) Bracing members:

- (i) A limit of $720/\sqrt{F_y}$, slenderness ratio L/r for Zones 3 and 4.
- (ii) A cyclic reduction factor of $1/[(1 + kl/r)/C_c]$ to be applied to the allowable compressive stress.
- (iii) The horizontal components of seismic force in tension or in compression $\leq 0.7 \times$ total force.

- (iv) Built-up members and width-thickness ratio must meet the AISC-ASD/LRFD Design.
- (b) Bracing Connections
The brace connection shall develop a tensile strength equal to $3(R_w/8)$ times the force in the brace or the tensile strength of the brace, whichever is smaller.
- (c) Bracing Configuration
 - (i) Chevron bracing member shall be design 1.5 times the required strength.
 - (ii) K-Bracing is generally prohibited.

3.3.2.4 Eccentrically Braced Frames and Dual Systems: Eccentrically Braced Frames (EBF) are both quite stiff and capable of large ductility and energy dissipating capacity. The inelastic action is localized in a shear or flexural yielding link and the balance of the frame is designed to develop that mechanism.

ASCE 7, and NEHRP design requirements are similar to the AISC-LRFD specifications. Some of the more important provisions are as follows:

- (i) The compression strength of the brace should exceed 1.5 times the axial load corresponding to the link yielding mechanism.
- (ii) The columns should remain elastic for the given mechanism calculated for 1.25 times of the material yield strength.
- (iii) The maximum link end rotation should not exceed 0.08 rad.
- (iv) Brace to beam connections must develop the brace capacity.

AISC-LRFD requirements are based on the AISC-ASD/LRFD Design Specifications. Some of the requirements are as follows:

- (i) The compression strength of the brace should be greater than 1.5 times the axial load corresponding to the link yield mechanism.
- (ii) The columns should remain elastic for the given mechanism calculated for 1.25 times of the material yield strength
- (iii) The maximum link end rotation should not exceed 0.09 radians at E'/E times the drift determined using the specified base shear, V .
- (iv) Brace to beam connection must develop the brace capacity.
- (v) Link should be laterally braced at their ends and have sufficient web stiffeners to transfer the brace force and insure adequate ductility for cyclic inelastic load reversals.

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NYC, SEAOC and UBC requirements are based on the AISC-ASD/LRFD Design Specifications. Some of the more important provisions are as follows:

- (i) The compression strength of the brace should exceed 1.5 times the axial load corresponding to the link yield mechanism.
- (ii) The columns should remain elastic for the given mechanism calculated for 125% of the material yield strength.
- (iii) The maximum link end rotation should not exceed 0.06 radians at $3R_w/8$ times the design drift.
- (iv) Brace to beam connections must develop the brace capacity.
- (v) Links should be laterally braced at their ends and have sufficient web stiffeners to transfer the brace force and insure adequate ductility for cyclic inelastic load reversals.

3.4 OTHER CODE PROVISIONS

In addition to sections that have been compared with the several building codes, the BOCA and NYC provisions on Modal Analysis Procedures; Soil Structures Interaction; and on Architectural, Mechanical and Electrical Components and Systems are based on the ATC 3-06 document. Minor modifications except the soil structures interaction in NYC. Site Coefficients for the NYC Code are shown in Table 3 since this represents a substantial change from the UBC and ATC 3-06 documents.

4. GENERAL COMMENTS:

4.1 NYC and BOCA Seismic Provisions

NYC and BOCA seismic provisions for steel structures are well detailed and reasonably fair. They are consistent with other codes, except for a few additional stringent requirements as for the drift. The provisions address all important issues of steel seismic design. For both codes, for ordinary structures, detailing requirements are not required and the only design impact is the requirement to include seismic forces in the analysis.

4.2 Economic Studies

There are several economic studies reported in the literature that are significant and pertinent to this report. These include studies by the Building Seismic Safety Council (BSSC) (19), studies by the Naval Civil Engineering Laboratory, Port Hueneme, California (20) and studies by Weidlinger Associates, Inc. (21). Each of these studies is briefly discussed.

4.2.1 Building Seismic Safety Council Studies

The Building Seismic Safety Council, by contract with local Consulting Engineering Firms, conducted trial designs (and also partial designs) of 46 structures located in 9 cities (Los Angeles, Memphis, Phoenix, Seattle, Charlestown, Chicago, Fort Worth, New York, and St. Louis). These structures included all building types (wood, masonry, concrete, steel, combinations) and ranged from 1 story to 30 stories. Each structure was designed twice - once to conform to the existing applicable building code with or without local seismic provisions as the case may be, and once to meet the applicable local building code supplemented or replaced by the proposed NEHRP provisions for seismic design. Each consultant was requested to estimate the increase in design time required to analyze and design for the seismic provisions and to estimate the increase in structural costs when utilizing the NEHRP provisions over the local applicable provisions.

As expected, the results showed considerable scatter. Cities with codes including seismic provisions understandably reported very little if any change in construction costs and in design costs when the NEHRP provisions were used. Cities without seismic provisions and in higher seismic zones naturally reported the highest changes in construction and design costs. Overall, structural costs increased from 1 to 15 percent (The structure may be considered approximately 20 percent of the overall building cost). The average increase is 5 percent or about 1 percent of the total building cost. Of particular interest for this report are the studies on steel buildings conducted in New York City. The New York Studies on seven buildings by two consultants includes 1 - 5 story steel moment frame, 1 - 30 story steel moment frame, and a 2 story steel frame with masonry walls. The results of the study were as follows:

<u>Building</u>	<u>Increase in Structural Cost</u>	<u>Increase in Building Cost</u>
NY-27A (5 story steel)	< 2%	< 0.5%
NY-28A (30 story steel)	< 0.5%	< 0.125%
NY-41A (2 story steel and masonry)	< 2.5%	< 1.5% to 2.5%

The NYC Consultants estimated a 75% increase in design time for a first time application but estimated that this would be reduced to 25% or less on repeated seismic design assignments. The BSSC committee concluded from all studies that basic fees for structural design of concrete and steel buildings would increase from 10 to 15 percent. The report also noted that the structural engineer's role would probably be increased to review non-structural components for seismic code conformance and therefore the higher number of 15 percent was probably a better estimate of increased design costs.

4.2.2 Naval Civil Engineering Laboratory Studies

The US Navy has studied the cost of providing seismic designs for buildings of new construction, for five different lateral force systems:

- (1) steel moment frame;
- (2) steel braced frame;
- (3) steel frame and concrete shear wall;
- (4) concrete moment frame; and
- (5) concrete moment frame and shear wall.

All studies are for 3 story buildings 185 feet by 185 feet in plan. The elements of the various framing systems were proportioned using the strength design provision contained in the AISC specifications and ACI Building Code Requirements. Design forces in the individual members were obtained by applying suitable load factors to the loads obtained from the dynamic response analysis.

The design spectra used in the study was based on the elastic design spectrum developed by Newmark and Hall. Six levels of base accelerations; which include 10% g, 15% g, 20% g, 25% g, 30% g, and 35% g; form the basis for six load conditions. The design spectra for these six values of base acceleration were constructed using the applicable amplification factors for 5% of structural damping. These spectra were digitized so as to represent the spectrum in a linear manner in the dynamic response analysis. This data was then used in the modal response analysis.

The detailed structural costs estimated were based on the results of the six design loading cases for the five different lateral force systems. Fig. 7 show the increase cost for providing seismic designs. Fig. 8 shows the first mode periods of the structure for the several seismic loadings and lateral force systems. Fig. 9 shows the damping used for analysis, based on engineering practice; damping increases with the ratio of applied load to design level.

For New York City the base acceleration is 0.15 g. From Figure 7 it is seen that a steel frame and shear wall structure is the most economical design for the 3 story building being studied. The total increase in construction cost is approximately 0.5 percent. This agrees quite well with the BSSC study. The equivalent building would be NY-41A (2 story steel and masonry) where the structural increase in cost was less than 2.5 percent. Using 20% for structural cost. The increase in total cost would also be 0.5 percent.

Figure 8 illustrates how the building period and hence the calculated base shear, varies with different lateral force systems. This stresses the importance of including seismic loading in cost studies to evaluate the structural systems. A poor choice of a lateral force system can result in a substantial difference in construction costs.

Figure 9 has been included to illustrate, that at least for three story buildings in the New York City area, where the ratio of applied load to design load is low, damping is not an issue. This is not true as building height increases.

5.2.3 Weidlinger Associate Studies

Weidlinger Associates, Inc., in a study sponsored by the National Science Foundation, studied simple prismatic buildings of both reinforced concrete and steel frames and compared the NYC code wind force with the UBC (static) lateral force requirements. The studies are similar to those reported by Nordensen (22) reported in section 5.3. The Weidlinger studies showed that steel frames needed some strengthening over wind design to accommodate seismic forces up to 400 feet in height and that concrete frames, regardless of height, must always be strengthened (21). The report concluded that implementation of seismic provisions would increase total construction costs from 0 percent to 2.5 percent. This conclusion is consistent with the NEHRP and Naval Civil Engineering Laboratory Studies.

An important economic concept that developed from the Weidlinger studies was that it is not always advisable to use the code option to determine the fundamental period (i.e., stiffness) of the structure. Two buildings studied (both reinforced concrete) showed the following natural frequencies:

<u>Building</u>	<u>Analysis (3D Model)</u>	<u>Code (UBC)</u>
NY1	5.8	2.7
NY2	3.4	1.8

These results indicate a substantial reduction in seismic base shear when the natural frequency is computed. This conclusion of the importance of calculating the natural frequency agrees with the Naval Civil Engineering Laboratory Studies. Weidlinger Associates also concluded that it was economically advantageous to use a site dependent spectra instead of the code spectra.

The NYC Economic Impact Sub-Committee also prepared an economic study of incorporating seismic provisions (23). This report is unpublished. The study covering 12 building types showed considerable scatter but generally confirmed the studies previously cited.

It can generally be concluded that for regular buildings with an Importance Factor of 1, increased total building costs will run at the lower bound of 0.5% while for irregular buildings with higher Importance Factors, the total building costs will increase in the order of 2.5%. These are very average values and many buildings could be exceptions to the rule.

4.3 Wind Versus Seismic Design

There has been considerable debate among building designers whether modern structures designed according to the code-specified wind load could prove safe in the moderate-level earthquake. The NYC code requires wind pressures up to 30 and even 40 pound per square foot for building over 20 stories. Nordenson (22) has analyzed and compared results for building heights up to 1000 ft., using NYC - moderate seismic versus NYC- wind base shears. For the study, he uses the unit base shear density values of 5 and 15 pound per cubic foot (Figs. 10 and 11) and 5 pounds per cubic foot with code reduction factor $R = 2$ (Fig. 12) for the seismic analysis. He concluded the following:

- o The ratio of values, representing the effects of a "maximum credible event" to those on which the building code is based, is on the order of 1.5 to 2 for wind along the eastern seaboard and for earthquakes in the western United States. The equivalent value (or "marginal hazard") is closer to 5 to 7 for earthquakes in the East. This reflects the rare occurrence of large earthquakes in the East, particularly in relation to the return period on which the currently proposed seismic codes are based. This suggests that structures designed for earthquake resistance in the East should be provided with even greater energy-dissipation than is the practice in the West (if this is possible) or that the average return period chosen as the basis for the code-prescribed loads should be increased to reduce the magnitude of the marginal hazard.
- o A comparison of unit base shears calculated by the code procedures for wind and seismic design demonstrates that for a representative range of building height, slenderness, and density the elastic seismic demands (allowing no load reduction for "ductility") in all cases exceeds the prescribed wind loading.
- o If the possible frequency content for the type of moderate earthquake used as the design basis event for NYC is incorporated in the comparison (e.g., by modifying the response spectrum shape) the relative seismic demands are substantially reduced. This is especially true for structures over 200 feet in height.
- o Clearly a further reduction in the difference is possible if even a nominal degree of energy-dissipating capacity is allowed for.

Hwang, et al (24) have also carried out a study of a low-rise building under loads imposed by nature such as wind or earthquake. Based on the study the following conclusions were drawn:

- o Seismic hazard appears to be more serious than the hazard imposed by wind, even when Zone 2 design is implemented. Thus, a low-rise

structure designed for wind loading without consideration for earthquake loading may require further strengthening for horizontal seismic force.

- o The seismic hazard curve approaches zero very slowly. Consequently, the limit state probability due to earthquake is rather insensitive to changes in the structural capacity. Thus, modeling of the seismic hazard needs special attention. Also, the seismic hazard curve used in the paper expresses the seismic input in limited terms. This is obviously not an adequate indicator of the seismic input.
- o The annual limit state probabilities due to wind are quite small. Thus, if the design is to be made for an ultimate limit state such as collapse of the structure, the load and resistance factors specified in ACI 318-83 associated with the load combination involving wind may be reduced for low-rise RC buildings.

4.4 AISC-LRFD Provisions

For the detailing requirements for steel in the AISC-LRFD document, the Load and Load Combinations Section 2 is adopted from the American National Standard Minimum Design loads for Buildings and other structures, ANSI A58.1. This section differs from all other documents. In the AISC-LRFD, the earthquake load E , has been modified from the Load Factor. This load has been converted into the amplified earthquake E' , based on a ANSI base shear formula. In the amplified earthquake load, the traditional 1.5 multiplier reduces to 1.0 multiplier. (i.e., $1.0 E' = 1.5 E$). By referring to ANSI provisions, the earthquake provisions in AISC-LRFD becomes consistent with the main AISC-LRFD specifications. To simplify, the AISC-LRFD specifications have suggested the formula for amplified earthquake load, E' for other documents.

(a) NYC, SEAOC and UBC.

$$E' = V = (3/8) ZICW$$

(b) ASCE 7, BOCA and NEHRP

$$E' = V = C'W$$

Fig. 13 compares the calculation of E' using the three sets of provisions presented assuming $S=1.2$; $I=1.0$ and Z or A_v = the value for the maximum seismic zone. From Fig. 13, it can be seen that an approximately 18% to 20% base shear value increase in AISC-LRFD over UBC when the fundamental period of vibration is more than 2.0 seconds. When the fundamental period of vibration is little over 0.7 seconds, the base shear value becomes the same amount in both AISC-LRFD and UBC requirements. If the load and load combination formulas were considered, the differences would not be significant.

This has been shown in Appendix "A" by a sample of calculation comparing the UBC and ANSI requirements.

4.5 Issues

(i) Drift

The Drift requirement in the present BOCA 1990 code is limited to 0.005 times the story height. This requirement will change in the proposed BOCA Earthquake code. The allowable story drift depends on the seismic Hazard exposure group and on the building height. The allowable story drift for a five or more story building is 0.015 times the story height for exposure groups 1 and 2 and 0.01 times the story height for exposure group 3.

(ii) R_w - Numerical Coefficient for Structural System. (Table No. 2).

In ASCE 7, BOCA, NEHRP, NYC, SEAOC and UBC codes, it is noted that a constant value is assigned to each type of structural system and seismic resisting system. The constant R_w factor reflects an unknown and an unspecified ductility ratio that the structure is allowed to reach under the design of earthquake load.

The numerical coefficient in NYC, SEAOC and UBC documents differ significantly from the ASCE 7, BOCA and NEHRP documents. The main reason is working strength design versus limit-state design. The R_w values appear to be inconsistent. For example, in ASCE 7, BOCA, and NEHRP, for reinforced concrete shear walls in a bearing wall system, $R_w = 4 \frac{1}{2}$, while in NYC, SEAOC and UBC, this value is 6. If a reinforced concrete shear wall in a building frame system is considered, BOCA, ASCE 7 and NEHRP have values of $R_w = 5 \frac{1}{2}$ and NYC, SEAOC and UBC have values of $R_w = 8$. This means that there is no common simple multiplier factor. The recent study by Gergely (25) of the R_w factor notes that it depends on at least nine factors such as damping, duration, compatibility of systems, stability of elements, redundancy, energy absorption, reliability, period increase, and performance in earthquake. Hwang and Jaw (26) has suggested that R_w depends on the duration of the seismic event. Hwang et. al. (24) also studied the R_w factor and concluded that the R_w is a function of maximum ductility ratio, the various damping ratios and the earthquake structure period ratio, T ($T = T_s/T_g$; T_s = fundamental period of structure; and T_g = dominant period of earthquake motion = $2T/W_g$). In this study, it was noted that when T is small, the R_w factor is also small. A structure with a small value of T is a very stiff structure.

The R_w factor varies with T significantly, especially in the case of a large ductility ratio. Therefore, the earthquake structure period ratio, T , is a important parameter to be considered in the determining the R_w value. It is necessary to have some research in this field, since it involves the

computation of Design Base Shear in the Static Force Procedure, Load Combinations, Story Drift Calculation, and Building Separations.

(iii) The Load and Load Combinations for the seismic effects on structures have very little or insignificant difference in sizing the steel members, when using either AISC-LRFD specifications (ANSI A58.1) or the UBC code and its supplements. This is indicated in the sample computations in Appendix A. The above conclusion is also verified by the US/Japan test structure. This structure was a full scale two-bay by two-bay in both directions, six-story high. (The test building is 15.0m x 15.0m x 21.5m).

4.6 Possible Future Direction

There is a trend to standardize seismic design provisions throughout the country with the NEHRP provisions serving as the base document. This is probably several years away but it will result in elimination of several present philosophical differences that prevent easy comparisons between the several seismic codes.

5. EASTERN EARTHQUAKES

The subject of earthquakes in Eastern North America has received increasing attention over the past several years. A symposium was held at the Sterling Forest Conference Center, Tuxedo, New York on October 20-22, 1987 on "Seismic Hazards, Ground Motions, Soil-Liquefaction and Engineering Practice in Eastern North America," (6) sponsored by the National Center for Earthquake Engineering Research. A similar meeting was held at the New York Academy of Sciences, New York, New York on February 24-26, 1988 on "Earthquake Hazards and The Design of Constructed Facilities in the Eastern United States," (7) sponsored jointly by the Academy and the NCEER. In the Fall of 1990 a workshop on "Problems Associated with Earthquakes in Eastern North American" was held at the University of Illinois, Urbana, sponsored by the National Science Foundation.

These earthquake conferences all attest to the following points:

1. The threat of earthquakes in Eastern North America is real. Most scientists agree that there is a better than 50% chance that a very severe earthquake will strike somewhere in the United States within the next thirty years. Exactly when and where is not known but the where includes Eastern North America.

2. While earthquakes in Eastern North America may not be expected to have the intensity of those in Western North America, the potential damage may be as large and possibly even larger. This is because the potential damage area in the East is much larger than in the West since the seismic energy is transmitted over larger areas due to differences in the geological formations. Eastern rocks are older and more dense than are Western rocks and there are fewer fault zones in the East to disrupt energy transmission. For example, the damage area for the 1811 New Madrid (Missouri) earthquake, which was about the same in magnitude as the San Francisco earthquake of 1906, resulted in an area of similar damage that was 15 times larger. This problem is compounded by the fact that very few structures in the East have been designed for earthquakes, while earthquake-designed structures have been standard practice in the West since 1933. When one considers the amount of construction that has taken place in the East over the past 75 to 100 years, damage estimates for Eastern earthquakes can become very large indeed.
3. Earthquakes in Eastern North America are significantly different than those in Western North America; they are characterized by (a) long-wave acceleration, versus short-wave acceleration in the West; (b) substantially different ground geology--they occur in the interior of a tectonic plate rather than at the edge of the plate; and (c) deep and scattered fault patterns, versus surface and well defined fault patterns in the West. These differences result in significantly different types of earthquakes.
4. Structural design technology for seismic forces is not generally transferable from the West Coast to the East coast. While analysis and design techniques may be transferable, the ground accelerations and the forces generated, as well as the durations of the earthquakes, are different and specification provisions need to be modified accordingly.

It is concluded that seismic provisions for building codes covering Eastern North America are still in a development stage and considerable research is necessary to resolve some of the problems. For this reason, it is believed that the New York City Seismic Provisions should have been based on the NEHRP Provisions which specifically address and were written for all regions of the country. Many specification changes are reactions to specific problems that develop or to specific events that occur. Each new earthquake has resulted in modifications to code seismic provisions. SEAOC reacts to incorporate modifications for the West Coast. UBC follows the SEAOC changes. These changes may not apply to the East Coast and the base document for the NYC Provisions gets changed for NYC without NYC participation. It will then be up to NYC Code Writers to determine what direction to take and how any modifications apply. If NYC Provisions were based on the NEHRP Provisions, these situations would not develop since modification to NEHRP would automatically address country wide and specifically East Coast problems. It should be noted that ASCE-7 and BOCA, both of which are clearly written for the Eastern United States, have elected to adopt the NEHRP provisions. While the UBC provisions are commendable, the fact remains that New York City would be fairly represented on ASCE-7 and BOCA code committees. The same is not true to the same extent for the UBC committee.

Notwithstanding the author's preference for the NEHRP provisions for the above reasons, this study concludes that the NYC and BOCA Seismic Provisions are adequate, fair and will have a fairly small impact on both design costs and construction costs. As previously noted, except for occupancies for essential features, hazardous facilities and special structures, the special seismic detailing provisions do not apply for standard occupancy structures. Most structures fall into this latter category. Earthquake loading must be considered in the analysis, but once the forces have been accounted for, that is the end of any seismic design. However, as also noted above, seismic code provisions are in a state of flux and constantly undergoing revisions, so once they are adopted, future changes could have substantial impact on both design and construction costs.

Seismic provisions should be included in the building codes of Eastern United States and specifically in the New York City code. Practically, it really does not make much difference which model code provisions are utilized or, in fact, the specific details of these provisions. The important point is that with these provisions, New York City will commence building an inventory of structures that have been designed for earthquake forces. In 50 to 100 years, that inventory will represent a substantial portion of the existing structures. The sooner the process starts, the more protection that will be available in the future.

6. RECOMMENDATIONS

- a. P-Delta Effect - Earthquakes in Eastern North America are characterized by long-wave phenomena as compared to the short wave phenomena of earthquakes in western North America. Because of this, the P-Delta effect on framed structures could be substantially different in the East than in the West. The P-Delta effect directly influences the natural frequency and consequently affects the dynamic forces and response behavior. The defining of the P-Delta forces constitutes a major seismic specification requirement, which has not yet been satisfactorily established for

Eastern North America. P-Delta is a very important, perhaps the major, factor affecting earthquake-type instability.

Since P-Delta is such an important factor affecting earthquake-type instability, research should be undertaken to improve our knowledge of the P-Delta affect and the design specification requirements relating to it.

- b. Dynamic Effects - For the same reason, the parametric response or dynamic instability of individual columns could be substantially different in the East than in the West. Under dynamic horizontal forces, individual columns vibrate laterally under loads well below the Euler Load, while the amplitude of the vibration could reach very large values. These amplitudes control the magnitude of the axial load the columns can sustain. This is essentially a problem associated with small P-Delta and member stability as compared to Item a which is concerned with large P-Delta and frame stability.

The effect of small P-Delta values on single columns in higher seismic zones should be studied.

- c. Connections - Modifications on how to design the structural frame and the component columns must also be applied to the connections since the structure is a single unit and all components participate.
- d. Drift - As the proposed drift requirements for seismic design are more stringent than generally used for comfort, studies should be undertaken to determine the reasonableness of the proposed requirements. Should the same drift limits apply to all occupancy groups? Should the limits vary based on building height? Should the limits be the same for different structural systems?
- e. Steel Frames - At present steel frames are categorized as "ordinary" and "special", terms which appear to have origin in the Uniform Building Code. The many restrictions on steel framing should be more clearly differentiated by degrees of restraint, ordinary, intermediate and special, eg. Efforts should be made to have codes and specification reflect such terminology.

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TABLE 1—LATERAL FORCE REQUIREMENTS (AFTER LUFT, 1989A)

	CODE DOCUMENTS			AISC-LRFD Design Specifications
	ASCE 7, BOCA and NEHRP	SEAOC and UBC	NYC	
(1) Base shear formula	$V = C_s W$	$V = \frac{ZIC}{R_w} W$	$V = \frac{ZIC}{R_w} W$	$V = E'(V) = ZIKCSW$ OR, when required $V = 3ZIKCSW$
(2) Coefficient C or C_s	$C = \frac{1.2A_s S}{RT^{2/3}}$	$C = \frac{1.25S}{T^{2/3}}$	$C = \frac{1.25S}{T^{2/3}}$	Per Applicable Code
(3) Upper limits	$C_s \leq \frac{2.5A_a}{R}$ OR $C_s \leq \frac{2.0A_a}{R}$ for S_2 or when $A_s \geq 0.30$.	$C \leq 2.75$	$C \leq 2.75$	$C \leq 0.12$ AND $CS \leq 0.14$ ($CS < 0.11$ for S_3)

TABLE 1—LATERAL FORCE REQUIREMENTS (CONTINUED)

	CODE DOCUMENTS			AISC-LRFD Design Specifications
	ASCE 7, BOCA and NEHRP	SEAOOC and UBC	NYC	
(4) Lower limits	$C_s \geq C_s(C_a T_a)$	$\frac{C}{R_w} \geq 0.075$ OR for SEAOOC only, $C \geq 0.8CT_a$ where $T_a = T$ from Method A.	$\frac{C}{R_w} \geq 0.050$	Per Applicable Code
(5) Zone factor	A_s and A_v	Z	Z	Per Applicable Code
(6) Importance factor	Seismic Hazard Exposure Group	I	I	Per Applicable Code
(7) Structural system factor	R	R_w	R_w	Per Applicable Code
(8) Soil factor	$S_1 = 1.0$ $S_2 = 1.2$ $S_3 = 1.5$ $S_4 = 2.0$	$S_1 = 1.0$ $S_2 = 1.2$ $S_3 = 1.5$ $S_4 = 2.0$	$S_0 = 0.67$ $S_1 = 1.0$ $S_2 = 1.2$ $S_3 = 1.5$ $S_4 = 2.5$	Per Applicable Code

TABLE 2—STRUCTURAL SYSTEMS (TABLE NO. 23-0, N.Y. CITY CODE)

BASIC STRUCTURAL SYSTEM	LATERAL LOAD-RESISTING SYSTEM DESCRIPTION	R_w
A. Bearing Wall	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 Fr.	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

Notes:

1. Basic structural systems are defined in Section 2312(d)6.
2. See Section 2312(c)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 2—STRUCTURAL SYSTEMS (CONTINUED)

BASIC STRUCTURAL SYSTEM	LATERAL LOAD-RESISTING SYSTEM DESCRIPTION	R_w
C. Moment-Resisting Fr.	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
D. Dual System	b. Concrete	4
	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	
	g. Reinforced masonry with concrete IMRF	6
		7
	2. Steel eccentric braced frame	
	a. With steel SMRF	12
	b. With steel OMRF	6
	3. Concentric braced frame	
	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(c)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.