Earthquakes: Steel Structures Performance and Design Code Developments

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INTRODUCTION

Major earthquakes occur several times each year throughout the world with heavy loss of life and property. Recent examples are the 1992 Cairo, Egypt earthquake with the loss of over 500 lives, and the Mexico earthquake of 1985 with the loss of 8,000 lives and the collapse of over 400 buildings.

The United States has experienced many large earthquakes, with the most seismic activity to date being located in California, i.e., Loma Prieta, California, 1989, 7.1 Richter magnitude and Landers, California, 1992, 7.5 Richter magnitude. It is evident from past occurrences of earthquakes that the highly seismic regions of the United States have a serious earthquake problem, and the less serious regions in the central and eastern parts of the country now realize that they have an earthquake problem which is being addressed through adoption of the latest seismic design provisions into the BOCA and SBCCI building codes. Some of these newly acquired seismic provisions are taken from the Building Seismic Safety Council program on improved seismic safety.

The Building Seismic Safety Council (BSSC) was established in 1979 under the auspices of the National Institute of Building Sciences (NIBS) as an entirely new type of instrument to develop and promulgate building earthquake hazard mitigation regulatory provisions that are national in scope. Its fundamental purpose is to enhance public safety by providing a national form that fosters improved seismic safety provisions for use by the building community in the planning, design and construction of buildings. To fulfill its purpose, the BSSC promotes the development of seismic safety provisions suitable for use throughout the United States. The BSSC believes that the regional and local differences in the nature and magnitude of potentially hazardous earthquake events require a flexible approach to seismic safety that allows for consideration of the relative risk, resources and capabilities of each community. The BSSC itself assumes no standards-making and promulgating role; rather, it advocates that code and standards formulation organizations consider BSSC recommendations for inclusion into their documents and standards. A recommendation that is taking place today in code writing.

The basic problem of earthquake design is to synthesize the structural configuration; the size, shape, and material of the structural elements along with the methods of fabrication, so that the structure will safely and economically withstand the action of earthquake ground motions. This of course requires a broad knowledge of the behavior of structures during earthquakes, and the final evaluation of the design will be made by a future earthquake. It is this ultimate test that has shown that steel-framed buildings and bridges have an excellent record of protecting life safety, as well as minimizing economic loss and business interruption.[1]

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STRUCTURES PERFORMANCE

Mexico Earthquake

On September 19, 1985, a magnitude 8.1 earthquake struck Mexico, followed by a magnitude 7.5 aftershock 36 hours later.[2] Data compiled by the Institute of Engineering of the National Autonomous University of Mexico revealed that a total of 330 buildings collapsed in central Mexico City. Of these, 12 were steel frame, and 318 were reinforced concrete or masonry. The majority of steel buildings that collapsed were built before 1957, while most concrete and masonry buildings were built between 1957 and 1976. Both years mark major revisions in the building code, adopted in response to past damaging earthquakes.

Steel structures constructed after 1957 fared much better than the norm, with only 6.8 percent of such buildings experiencing severe damage or collapse. Steel structures constructed after 1976 performed excellently; no cases of severe damage or collapse were noted in this group, and only four such buildings sustained any structural damage.[2]

Some steel buildings constructed from 1920 through the 1940s experienced severe damage. These structures were built prior to the adoption of earthquake codes, and used construction types that were abandoned following the 1957 earthquake. The most common type of steel building construction used over the last three decades has been highly redundant moment frames where almost every beam-column joint in these structures is moment resisting. The second most common lateral system for steel structures was moment-resisting frames with braced bays. The Pino Suarez complex accounted for all the reported failures of this system in the 1985 earthquake.

Analyses performed after the earthquake have provided an explanation of the Pino Suarez failures.[3] Very large axial loads, due to gravity and seismic overturning, overstressed the exterior columns in the braced bays. The bracing system being capable of resisting story shears several times higher than the code design level produced unanticipated large axial forces in the columns. As the result of these findings, provisions have been added to the 1988 edition of the Uniform Building Code to prevent overload of columns from overturning forces that exceed those calculated from the basic seismic provisions of the Code.

Structural steel was successfully used to strengthen reinforced concrete buildings prior to the 1985 earthquake.[4] The 12-story Durango Building is located in the heaviest damaged region of the city. After being heavily damaged in the 1979 earthquake, the building was retrofitted with steel frames, which added ductility as well as strength to the structure. In the 1985 earthquake the building performed excellently, sustaining no damage. The steel frames are believed to have carried over 80 percent of the total lateral force.

Although steel construction in Mexico differs substantially from practice in the United States, dozens of modern steel buildings located in the badly shaken lake bed area of Mexico City received no damage. A good example is the 44-story Torre Latinoamericana, constructed in the early 1950s and designed for earthquake loads, which performed excellently in 1985 as it had in three previous earthquakes in 1957, 1978 and 1979.

Table 1.
Statistical Summary of Damage to Buildings
1985 Earthquake

Type Structure	Extent of Damage	Year When Built			
		Pre-1957	'57–'76	Post 1976	Total
Steel Frame	Collapse	7	3	0	10
	Severe	1	1	0	2
RC Frame	Collapse	27	51	4	82
	Severe	16	23	6	45
Waffle Slab	Collapse	8	62	21	91
	Severe	4	22	18	44

Whittier-Narrows, California Earthquake

The October 1, 1987 Whittier-Narrows earthquake of magnitude 5.9 (Richter Scale) was considered a moderate earthquake. Several aftershocks caused a few structures that were badly damaged on October 1 to collapse in an October 4 aftershock of 5.5 magnitude. USGS records show unusual high ground accelerations of 0.40g to 0.60g, and ground displacements of 1 to 2 inches.[5] According to the National Center for Earthquake Engineering Research (NCEER), most earthquake damage occurred in unreinforced masonry buildings, older homes and modern buildings in construction types lacking in ductility.[6]

Several reinforced concrete and shear wall buildings, bridges constructed according to pre-1971 engineering practice sustained heavy damage. Major shear damage was experienced by the supporting concrete columns of the overpass located at the junction of the I-5 and I-605 Freeways, 15 miles East of downtown Los Angeles. Whereas bridge abutments experienced moderate to minor damage by spauling of concrete underneath the supporting pads, no damage was noted in abutments, columns and piers of bridges that were retrofitted by cable restrainers.[6]

A two-story concrete parking structure built in 1964 and located in the Whittier Quad shopping center collapsed after shear failure of its columns. Large girders had much stronger sections than the supporting columns, thereby creating a strong-beam–weak-column situation. The requirement for a strong-column vs. a weak-beam design was first required for concrete structures in the 1985 Uniform Building Code. The Code requires that the sum of the column moments at a beam-column joint be a minimum of 20 percent greater than the sum of the girder moments. A similar design provision became a requirement for structural steel seismic design in 1988.[7]

The four-story steel-framed California Federal Savings Service Center relied upon braced (chevron) frames for lateral resistance and was designed in accordance with the 1979 Uniform Building Code. During the earthquake the building experienced a peak ground motion several times higher than the working stress design levels, Structural damage was limited to the buckling of a single wide flange bracing member on each of the second, third and fourth floors.[8] In spite of the severe ground motions that the building experienced, it was restored to service within a week. In contrast, an adjacent two-story precast concrete structure built in 1980 experienced such extensive damage that repair to the building took nine months.

Loma Prieta Earthquake

On October 17, 1989 an earthquake of 7.1 Richter magnitude occurred that was centered approximately 60 miles south of San Francisco. Among the seismic-induced events were the collapse of the elevated Cypress Street section of Interstate 880 in Oakland; the collapse of a section of the San Francisco-Oakland Bay Bridge; and major structural damage to modern buildings in Oakland, San Francisco and Burlinghame. Over 62 people died.

Some of the heaviest concentration of damage occurred in the city of Oakland, 60 miles north of the earthquake epicenter, where peak ground accelerations were only 0.2g to 0.26g.[9] A 15-story concrete shear wall structure in downtown Oakland suffered extensive damage when its lightweight concrete shear walls shattered at the first story. The presence of a redundant steel frame within the building saved the structure.[2]

The Hyatt Regency Hotel located in Burlingame, a midrise reinforced concrete structure, sustained extensive damage to its shear walls and floor slab around the elevator core. The structure was designed to the 1985 Uniform Building Code and construction completed just prior to the earthquake. Repair of damage resulted in closure of the hotel for more than eight months. In contrast, modern steel-framed buildings performed excellently in the Loma Prieta earthquake, as they have in the past.

Damage to steel structures was typically limited to cracking of cladding and interior partitions with wide-spread disarray of building contents. The nonstructural damage sustained by steel frame buildings may largely be attributed to their flexibility, which results in large displacements.[10]

Major transportation routes were affected by the Loma Prieta earthquake. Immediately after the earthquake 11 major highways and freeways were closed due to landslides, structural damage or bridge collapse. The collapse of the Cypress Street elevated section of I-880 (near downtown Oakland) was responsible for the majority of earthquake deaths. The double-deck highway system consists of box girder decks supported by concrete frames. The failure occurred at the connection of the support columns and the transverse beams, at the lower road level.

Redesign of the Cypress Street roadway was completed in October of 1992, with reconstruction scheduled to begin in early 1993. Five sections of the new design of I-880 will be constructed of structural steel.

While a mile-long section of the Cypress Street overpass structure of I-880 collapsed, buildings of various types and vintage right next to the collapsed freeway experienced very little or no damage. It is of further interest that the collapsed portion of I-880 is located on man-made ground, whereas the surviving elevated section is located on a competent sand formation.[10]

The collapse of a section of the San Francisco-Oakland Bay Bridge greatly impacted bay area commuting. The Bay Bridge carries an average of 250,000 vehicles per day between San Francisco and the cities of the East Bay. The Bay Bridge is a double-decked steel bridge about 8.5 miles long. Its west bay crossing is a suspension span, while the east bay crossing consists of deck trusses and through trusses. About two miles west of the Oakland toll plaza, 50-foot horizontal spans, situated along the top and bottom decks and located above a main pier, serve to link the bridge's deck-truss section to the east with its through-truss section to the west. The anchor bolts that attached the bridge's deck-truss section to the pier were the only constraint that prevented the two deck-spans from displacing longitudinally with the bridge's deck-truss section to the east. During the earthquake, large longitudinal and lateral seismic forces caused these bolts to fail in shear. Following this failure, the earthquake-induced longitudinal deformations along the length of the deck-truss section were sufficiently large (7 in.) to result in collapse of the upper and lower spans. The cause of the bridge failure was easily understood, and the 50-foot section was repaired in one month and opened to traffic again.[11]

Landers, California Earthquake

On June 28, 1992 a magnitude 7.5 earthquake, epicentered near Landers, California in the Southern Mojave Desert, occurred at 4:58 a.m. At 8:04 a.m. a second earthquake, of 6.5 magnitude and centered near Big Bear Lake 20 miles to the West of Landers in the San Bernardino mountains, occurred.

Both earthquakes occurred near the so-called "Big Bend" of the San Andreas Fault, causing scientists to speculate about the possibility of a larger earthquake on this conspicuously quiet stretch of the longest fault in California.

Accelerations of as much as 1.0g were recorded in Lucerne Valley, and 0.55g in Big Bear, although most epicentral stations reported peak accelerations of 0.3g or less.[12] Caltrans had instrumented one of the tall (70 ft.) bridge columns on Interstate 10, near the city of Colton, after retrofitting the concrete column with a steel plate jacket as a result of the 1989 earthquake. Although the Landers earthquake showed a ground acceleration of only 0.1g, acceleration at the top of the column was 0.8g in the longitudinal direction and 1.02g in the transverse direction. The column experienced no damage which can be attributed in part to the retrofit method of wrapping the concrete column in steel.

The Landers earthquake sequence appears to have occurred in a northerly northwest direction, striking the Camp Rocks, Emerson and Johnson Valley faults. It appears that the Big Bear seismic event was initiated by movement on the Camp Rock-Emerson Fault. Two sets of 500kV and two sets of 220kV transmission lines crossed the Camp Rock-Emerson Fault near the north end of the rupture. The fault passed directly between the legs of a bolted steel frame 220kV tower, moving two of the legs approximately 9 feet. This movement resulted in substantial deformation of the steel tower and failure of several braces. No damage was sustained by the lines or ceramic insulators and the tower continued to provide adequate support until repaired.[13]

EARTHQUAKE LEGISLATION

Earthquake Hazard Reduction

A review of the history of seismic code development in the United States[14] helps to more fully understand the lethargy in bringing modern seismic code requirements into the building codes. Much like other areas of the world the early seismic design codes in the United States were the result of disastrous earthquakes, primarily in California. Major milestones in seismic code development closely follow many of our significant earthquakes. California's Earthquake Reduction Act of 1986 was signed into law shortly after the 1985 Mexico Earthquake.[15]

This Act is sponsored by the Seismic Safety Commission which has the responsibility of preparing and administering the California Earthquake Hazard Reduction Program. It is a multidisciplinary commission consisting of 17 commissioners and 12 staff. The commission's goal is to significantly reduce earthquake risk in California by the year 2000. The responsibility for meeting this goal must be shared by State, City, and County agencies as well as the private sector.

The first step is an advisory document which is based on initiatives to improve seismic safety. Between 1987 and 1992 there were 72 initiatives passed by the legislature, with another 42 initiatives scheduled for the 1992–96 period. The advisory document contains 150 milestones to measure progress and record accomplishments.

The hazard reduction program is based on five criteria: (1) lives saved, (2) damage reduction, (3) socioeconomic continuity, (4) opportunity (ease of implementation), (5) cost. These priorities must pass the common sense test of will the decision maker and the general public consider the initiative as being practical, sensible, and feasible? The program has 42 initiatives integrating actions needed in the public and private sector.

The size of the earthquake and the amount of damage greatly influence safety legislation. For instance, during the 1987–88 session of the California legislature the Whittier, California earthquake, M 5.9, occurred with 23 seismic safety bills being introduced and 11 of the bills passed by the legislature but only six bills finally being signed into law by the Governor.

Two years later during the 1989–90 legislative session the Loma Prieta earthquake of M 7.1 occurred with 443 bills being introduced. Of these bills, 164 passed the legislature and 137 of those were signed into law.

If a historical comparison of legislative action is made for the 1906–1989 period, 112 seismic safety bills were signed into law during that 83-year period. But from 1990 to present, a short two-year period, 206 seismic safety bills were made law. Obviously the impact of the Loma Prieta earthquake.

How seismic safety translates into dollars is shown in Table 3. Out of $670 million for fiscal year 1991–92 the bulk of the money, 63 percent, went to the California Department of Transportation (Caltrans), again as the result of the major road and bridge damage inflicted by the Loma Prieta earthquake.

A review of the 12,500 California State highway bridges after the 1971 Sylmar California earthquake (6.6 Richter magnitude) showed that ten percent of the bridges constructed prior to 1971 would need to be strengthened. The initial portion of the 1973 program was aimed at retrofitting bridge hinges. Inexpensive joint restraining devices were developed and installed. This portion of the program focused on 1,249 bridges statewide and was scheduled to be completed in 1988. Despite these safeguards, the possibility of bridge damage was not eliminated. The 1987 earthquake on the Whittier Fault verified the action Caltrans started in 1973 because although there was the expected damage during the earthquake no bridges collapsed. The program is currently focused on wrapping a steel reinforcement shield around the concrete column in bridges with single-column designs.

Table 2.
California at Risk 1992–1996

Category	Number of Initiatives	Focus
1	20	Existing Facilities
2	5	New Facilities
3	9	Emergency Management
4	5	Disaster Recovery
5	3	Research and Education

Table 3.
California Seismic Safety Activities
FY 1991–1992, $670 Million Total

Agency	$ Millions
Department of Conservation	6.6
Office of Emergency Services	192.6
General Services	17.0
Public Utilities Commission	0.6
Seismic Safety Commission	1.4
Department of Water Resources	1.5
University of California	3.4
Office Statewide Health Planning & Development	16.6
Department of Transportation (Caltrans)	420.3

Seismic Design Provisions

It is impossible to predict the location and magnitude of earthquakes accurately. It is therefore essential to adopt a preventive design philosophy in order to avoid repeating errors in rebuilding after an earthquake and in planning new construction. This goal is best achieved through code adoption where state-of-the-art seismic design criteria is specified.

One such specification is the AISC Seismic Provisions for Structural Steel Buildings. First published in 1990 for Load & Resistance Factor Design, it has been updated in a 1992 version to encompass both LRFD and ASD design procedures.[16] A significant change in the 1992 edition of the seismic provisions is the conversion to the loads and design format recommended by the 1991 National Earthquake Hazards Reduction Program (NEHRP) document.[1]

Whereas the provisions contained in the AISC seismic document are to be used in conjunction with the AISC Load & Resistance Factor Design (LRFD) Specification, the load provisions have been modified from those in the LRFD in order to be consistent with the load provisions contained in the BOCA, SBCCI Codes, and the ASCE 7-93 Minimum Design Loads for Buildings and Other Structures.[17] All these new seismic load provisions are modeled after the 1991 NEHRP earthquake provisions.

Table 4. Seismic Hazard Exposure Groups	
Group III	Buildings having essential facilities that are necessary for post-earthquake recovery and requiring special requirements for access and functionality.
Group II	Buildings that constitute a substantial public hazard because of occupancy or use.
Group I	All buildings not classified in Groups II and III.

Table 5. Seismic Performance Categories

Value of A_v	Seismic Hazard Exposure Group		
	I	II	III
$0.20 \le A_v$	D	D	E
$0.15 \le A_v < 0.20$	C	D	D
$0.10 \le A_v < 0.15$	C	C	C
$0.05 \le A_v < 0.10$	B	B	C
$A_v < 0.05$	A	A	A

Table 6. Load Combinations

$1.4D$	(3-1)
$1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R')$	(3-2)
$1.2D + 1.6(L_r \text{ or } S \text{ or } R') + (0.5L \text{ or } 0.8W)$	(3-3)
$1.2D + 1.3W + 0.5L + 0.5(L_r \text{ or } S \text{ or } R')$	(3-4)
$1.2D \pm 1.0E + 0.5L + 0.2S$	(3-5)
$0.9D \pm (1.0E \text{ or } 1.3W)$	(3-6)

The requirements for analysis and design of buildings under the 1991 NEHRP and the 1992 AISC Seismic Provisions are based on a seismic hazard criteria, Table 4, that reflects the relationship between the use of the building and the level of earthquake to which it may be exposed. This relationship primarily reflects concern for life safety and, therefore, the degree of exposure of the public to the hazard based on a measure of risk.

The purpose of the NEHRP seismic ground acceleration maps and corresponding seismic hazard exposure groups is to provide the means for establishing the measure of seismic risk/performance for a building of any use group, and in any area of the United States, Table 5.

Seismic performance design requirements get progressively more stringent as the categories proceed from A through E. The seismic hazard exposure groups listed in Table 4 are defined in detail, with examples of buildings in each type, in ASCE 7-93.[17]

The most frequently used load combinations given in the LRFD Specification are repeated in the AISC Seismic Provisions publication in order to reduce the amount of cross-referencing by the engineer. The load combinations, Table 6, have been modified to be consistent with the anticipated ASCE 7-93 document.

The most notable modification is the reduction of the load factor applied to the earthquake load, E, to 1.0. This results from the limit states load model used in ASCE 7-93. The earthquake load and load effects E in the ASCE 7-93 are composed of two parts. E is the sum of the seismic horizontal load effects and one half of A times the dead load effects. The second part adds an effect simulating vertical accelerations concurrent to the usual horizontal earthquake effects. An amplification factor to earthquake load E of $0.4R$ is prescribed. The amount of this amplification was assumed to be two times the deflections generated by forces specified for a building with $R = 5$. This amplification factor is thus $2R / 5$ or $0.4R$. The added complication that would be required to consider orthogonal effects with the amplified force is not deemed necessary.

Base Shear and the *R* Factor

The equivalent lateral force procedure for a Special Moment Resisting Frame is greatly influenced by the R or R_w factor, a numerical coefficient commonly referred to as a response modification factor.

NEHRP	*UBC*
$V = C_s W$	$V = ZICW / R_w$
$C_s = \dfrac{1.2A_v S}{RT^{2/3}}$	$C = \dfrac{1.25S}{T^{2/3}}$
For Map Area 7, $A_v = 0.4$	Seismic Zone 4, $Z = 0.4$
Soil/Site Coefficient $S = 1.0$	Importance Factor $I = 1.0$ Site Coefficient $S = 1.0$
$V = \dfrac{1.2(0.4)SW}{8T^{2/3}}$	$V = \dfrac{0.4(I)1.25(S)W}{(12)T^{2/3}}$
$V = 0.06W$	$V = 0.04W$ (for equal T values)

Basically, the 50 percent difference in the base shear values is due to the different response modification factor, R, used by the Uniform Building Code and NEHRP. The R value depends on the degree to which the system can be allowed to go beyond the elastic range, its energy dissipation in so doing, and the stability of the vertical load carrying system during inelastic response due to maximum expected ground motion. It is recognized that the assigned R values must be periodically reviewed as earthquake performance is observed and more data on material and system performance becomes available.[18]

Under NEHRP design provisions, the design of a structure

Table 7.
Comparison of 1991 NEHRP and 1991 UBC Drift Limits

Single Story Buildings (Assumed to have a $C = 2.75$ [UBC] and a $C_s = 2.5\ A_a / R$ [NEHRP])
$[Z = A_v]$

					UBC Drift		UBC Drift		NEHRP Allowable Elastic Drift				Ratio of NEHRP to UBC		
	NEHRP		UBC	Force Amplifier	(0.005h or [0.04 / R_w]h)		Scaled to NEHRP by 0.91R_w/ R		(Delta / C_d)				SHEG I	SHEG II	SHEG III
Framing System	Cd	R	R_w	$0.91R_w/R$	I = 1.0	I = 1.25	I = 1.0	I = 1.25	0.025 Hsx	0.020 Hsx	0.015 Hsx	0.010 Hsx	NA	0.020 Hsx	0.015 Hsx
Bearing Wall System															
Light framed w / sp	4	6.5	8	1.12	0.0050	0.0040	0.0056	0.0045	0.0063	0.0050	0.0038	0.0025		1.12	0.84
CBF	3.5	4	6	1.37	0.0050	0.0040	0.0068	0.0055	0.0071	0.0057	0.0043	0.0029		1.05	0.78
Building Frame System															
EBF	4	8	10	1.14	0.0040	0.0032	0.0046	0.0036	0.0063	0.0050	0.0038	0.0025		1.37	1.03
Light Framed w / sp	4.5	7	9	1.17	0.0044	0.0036	0.0052	0.0042	0.0056	0.0044	0.0033	0.0022		1.07	0.80
CBF	4.5	5	8	1.46	0.0050	0.0040	0.0073	0.0058	0.0056	0.0044	0.0033	0.0022		0.76	0.57
Moment Resisting Frame System															
SMF Steel	5.5	8	12	1.37	0.0033	0.0027	0.0046	0.0036	0.0045	0.0036	0.0027	0.0018		1.00	0.75
OMF Steel	4	4.5	6	1.21	0.0050	0.0040	0.0061	0.0049	0.0063	0.0050	0.0038	0.0025		1.03	0.77
SMF Conc.	5.5	8	12	1.37	0.0033	0.0027	0.0046	0.0036	0.0045	0.0036	0.0027	0.0018		1.00	0.75
IMF Conc.	3.5	4	8	1.82	0.0050	0.0040	0.0091	0.0073	0.0071	0.0057	0.0043	0.0029		0.78	0.59
OMF Conc.	2	2	5	2.28	0.0050	0.0040	0.0114	0.0091	0.0125	0.0100	0.0075	0.0050		1.10	0.82
													Average	1.03	0.77
													Avg. for all moment frames	0.98	0.74

(sizing of members, connections, etc.) is based on the internal forces resulting from a linear elastic analysis using the prescribed forces. It assumes that the structure as a whole, under the prescribed forces, will not deform beyond a point of significant yield. The elastic deformations then are amplified to estimate the real deformations in response to the design ground motion.[1]

Earthquake load combinations in the AISC Provision[16] are:

$$1.2D + 0.5L + 0.2S \pm 0.4R \times E \qquad (3\text{-}7)$$

$$0.9D \pm 0.4R \times E \qquad (3\text{-}8)$$

The amplification factor ($3R_w$ / 8) was derived by using the similar assumptions that were used in deriving the factor for ASCE 7-93. The same building type with $R = 5$ in ASCE 7-93 has a Structural System Coefficient $R_w = 8$ in the 1991 Uniform Building Code. The deflection determined by this R_w was used as the value to be amplified by 3. Thus ($3R_w$ / 8)E.

Drift Limits

Model Codes and resource documents such as NEHRP contain specific seismic drift limits, but there are major differences among them, i.e., UBC drift allowable is ⅓ greater than that allowed by NEHRP for a Special Moment Frame in steel, Seismic Hazard Exposure Group I for "All other buildings" category, Table 7.[19]

There are many reasons for controlling story drift in a building. Stability considerations dictate that flexibility be controlled. The stability problem is resolved by limiting the drift of the building columns and the resulting secondary moments commonly referred to as P-Δ effects. Buildings subject to earthquakes also need drift control in order to limit damage to partitions, emergency stair towers, exterior curtain walls and other fragile nonstructural elements. The design story drift limits of NEHRP take into account these needs, and in order to provide a higher performance standard for essential facilities the drift limit for Seismic Hazard Exposure Group III is more stringent than that for Groups I and II, Table 4 and Table 8.

The story drift limitations of ASCE 7-93 and NEHRP provisions are applied to an amplified story drift that estimates the story drift that would occur during a large earthquake. For determining the story drift the deflection deter-

Table 8. Tentative Allowable Story Drift			
Building	**Seismic Hazard Exposure Group**		
	I	**II**	**III**
Single-story buildings without equipment attached to the structural resisting system and with interior walls, partitions, ceilings, and exterior wall system that have been designed to accommodate the story drifts.	No limit	$0.020h_{sx}$	$0.015h_{sx}$
Buildings with four stories or less with interior walls, partitions, ceilings, and exterior wall system that have been designed to accommodate the story drifts.	$0.025h_{sx}$	$0.020h_{sx}$	$0.015h_{sx}$
All other buildings.	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$
Where h_{sx} is the story height of the story drift calculated.			

mined using the earthquake forces E is amplified by a deflection amplification factor, C_d (5½ for a SMF of steel) which is dependent on the type of building system.

The 1991 Uniform Building Code[7] drift provisions are numerically specific and require that story drift shall be calculated including the translational and torsional deflection resulting from the application of unfactored lateral forces. There are no drift limits on single-story steel-framed structures with low occupancies.

The AISC Seismic Provisions do not specify specific drift limits but defer to the governing design code, stating that the story drift shall be calculated using the appropriate load effects consistent with the structural system and method of analysis.

Ordinary Moment Frames

Ordinary moment frames (OMF) of structural steel are moment frames which do not meet the requirements for special design and detailing required of the Special Moment Frame. OMF of structural steel exist in all areas of seismic activity throughout the country, and experience has shown that this type of building has responded without significant structural damage.

The 1992 AISC Seismic Provisions for OMF have beam-to-column joint requirements that allow the use of either fully restrained (FR) or partially restrained (PR) connections, contrary to the Uniform Building Code. But the beam-to-column connection must meet *one* of three criteria depending on whether it is a fully restrained (FR) or partially restrained (PR) connection:

1. If fully restrained then the connection may conform to the requirements for SMF except that the required flexural strength of a column-to-beam *joint* is not required to exceed the nominal plastic flexural strength of the connection
2. If fully restrained with a connection design strength meeting the requirements of Load Combinations 3-1 through 3-8
3. If either FR or PR connections meeting all the following:
   a. The design strengths of the members and connections shall have a design strength to resist Load Combinations 3-1 through 3-6.
   b. The connections have been demonstrated by cyclic tests to have adequate rotation capacity at a story drift calculated at a horizontal load of $0.4R \times E$.
   c. The additional drift due to PR connections shall be considered in design.

The provision requiring a demonstration of rotation capacity is included to permit the use of connections not permitted under the provisions for SMF, such as top and bottom angle joints, in areas where the additional drift is acceptable.

Column Strength

As the result of the reduction in the actual lateral forces for use in a code elastic analysis of the structure, overturning forces are underestimated and are amplified by unaccounted-for concurrent vertical accelerations. These two load combinations account for these effects:

Axial compression loads:

$$1.2P_D + 0.5P_L + 0.2P_S + 0.4R \times P_E \leq \phi_c P_n \qquad (6\text{-}1)$$

where the term $0.4R$ is greater or equal to 1.0.

Axial tension loads:

$$0.9P_D - 0.4R \times P_E \leq \phi_t P_n \qquad (6\text{-}2)$$

where the term $0.4R$ is greater or equal to 1.0.

These load combinations are to be applied without consideration of any concurrent flexure forces on the member.

Column Splices

Column splices, as a minimum, must be able to transmit the prescribed design code forces, but more stringent provisions are required for column splices in frames that due to seismic forces are required to transmit net tension forces.

The AISC Seismic Provisions require partial penetration welded joints that are subject to net tension to be designed for forces in excess of the code forces (150 percent of the required strength) and that the column splice be located three feet from the beam-to-column connection.

For column splices in seismic design, using either complete or partial penetration welded joints, beveled transitions as given in AWS D1.1, Section 9.20,[20] are not required when changes in thickness and width of flanges/webs occur.

The possibility of developing high tensile stresses in partial penetration welded column splices during a maximum probable seismic event is real and the use of splice plates welded to the lower part of the column and bolted to the upper part should be considered.

The designer should always review the conditions found in columns in tall stories, large changes in column sizes at the splice, or where the possibility of a single curvature exists on a column over multiple stories to determine if special design strength or special detailing is necessary at the splice.

Panel Zone Design

Cyclic tests of beam-to-column joints has shown the ductility of shear yielding in column panel zones.[21] The usual Von Mises shear limit of $F_y/\sqrt{3}$ did not accurately predict the actual panel zone behavior. Tests have shown that strain hardening and other phenomena have enabled panel zone shear strengths in excess of $1.0F_y dt$ to be developed.

In calculating the required panel zone shear strength for AISC LRFD Seismic Provisions, the typical Load Combinations 3-5 and 3-6 are used with the nominal web shear strength defined as $0.6F_y dt$. In order to provide the same level of safety as determined by tests and as contained in the 1991 Uniform Building Code, a lower resistance factor of 0.75 was selected:

$$\phi_v V_n = 0.6\phi_v F_y d_c t_p \left[1 + \frac{3b_{cf} t_{cf}^2}{d_b d_c t_p}\right] \qquad (8\text{-}1)$$

where for this case $\phi_v = 0.75$

where:

t_p = Total thickness of panel zone including doubler plates, in.
d_c = Overall column section depth, in.
b_{cf} = Width of the column flange, in.
t_{cf} = Thickness of the column flange, in.
d_b = Overall beam depth, in.
F_y = Specified yield strength of the panel zone steel, ksi.

Eccentrically Braced Frames (EBF)

Whereas concentrically braced frames (CBF) are braced systems whose worklines essentially intersect at points with no eccentricities, the EBF is composed of columns, beams, and braces in which at least one end of each bracing member connects to a beam at a short distance (eccentricity) from a beam-to-column connection.

Research[22] has shown that buildings using the EBF system possess the ability to combine high stiffness in the elastic range together with excellent ductility and energy dissipation capacity in the inelastic range. In the elastic range, the lateral stiffness of an EBF system is comparable to that of a CBF system, particularly when short link lengths are used.

In the inelastic range, EBF systems provide stable, ductile behavior under severe cyclic loading, comparable to that of a SMF system. The design purpose of an EBF system creates a system that will yield primarily in the links. The special provisions for EBF systems are intended to satisfy this criterion and to ensure that cyclic yielding of the links can occur in a stable manner.

Upon publication of the first research report[22] on EBF, several important applications of this design concept were employed in the design of major buildings. Ten years later the Structural Engineers Association of California developed recommended seismic design provisions for the EBF which were accepted for inclusion into the 1988 Uniform Building Code. It is to be noted that the SEAOC and UBC design provisions for EBF are essentially identical and are based on the allowable stress design approach, whereas the NEHRP and AISC Provisions are based on the strength design approach.

Eccentrically braced frames are designed so that under earthquake loading, yielding will occur primarily in the links. The diagonal braces, the columns, and the beam segments outside of the links are designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded and strain hardened links.

EBF have become a well established structural steel system for seismic resistant construction. Sustained research since 1975 combined with experience from many buildings that employed the system has provided the database for proper design of eccentrically braced frames. The most recent EBF code provisions are contained in the 1992 AISC Seismic Provisions.[14] This document represents the most up-to-date and comprehensive code requirements for EBFs currently available in the United States.

Conclusion

Several years ago, it was not uncommon for local jurisdiction to each have their own building code. However, this did little to promote efficient construction, nor to promote uniform safety. Today the system has evolved to where most cities adopt one of three model codes: the Uniform Building Code, promulgated by the International Conference of Building Officials (ICBO) and used throughout the western United States; the National Building Code, promulgated by the Building Officials and Code Administrators International (BOCA) and used in the northeastern United States; the Standard Building Code, promulgated by the Southern Building Code Congress International (SBCCI) and used in the southeastern United States.

Since 1957 the Seismology Committee of the Structural Engineers Association of California has published its seismic design recommendations. They have also acted as an effective bridge between seismic research and the application of their recommendations by assuring that the provisions were adopted into the UBC in a timely manner. The last major rewrite of the SEAOC recommendations occurred in 1988, which formed the basis for the seismic provisions in the 1988 UBC. The SEAOC Seismology Committee is beginning the preparation of a code change to convert the seismic provisions in the UBC to a limit state design basis. Their goal is for a completion time to allow the changes to be incorporated in the 1997 UBC.

Whenever possible BOCA and SBCCI prefer adopting design standards by reference.

Unfortunately, the national seismic standard adopted was ANSI A58.1 / ASCE 7, which was made up of UBC criteria that was developed by SEAOC. The delay that results from this technology transfer resulted in the 1987 NBC and 1988 SBC being based on 18 and 14 year old SEAOC recommendations respectively.

But in 1991 BOCA approved seismic code changes based on NEHRP provisions from its 1988 publication and updated that to the 1991 NEHRP provisions in 1992. The SBCCI followed a similar path to code update and the 1993 Standard Building Code Supplement will contain seismic provisions based on the 1991 NEHRP.

Within a few months of publication of the June 1992 AISC Seismic Provisions for Structural Steel Buildings,[16] both BOCA and SBCCI approved the provisions which will be referenced in the 1993 NBC, and will appear in the 1993 Amendments to the SBC. Could uniformity in code seismic design criteria be just around the corner for the United States?

The provisions contained in the AISC Seismic Provisions for Structural Steel Buildings,[14] are to be used in conjunction with the AISC LRFD Specification in the design of buildings in the areas of moderate, high seismicity. The First Edition of the LRFD Specification was published in 1986. It did not contain seismic design criteria.

Load and Resistance Factor Design (LRFD) is an improved approach to the design of structural steel for buildings. The method involves explicit consideration of limit states, multiple load and resistance factors, and implicit probabilistic determination of reliability. The designation LRFD reflects the concept of factoring both loads and resistance. The LRFD method was devised to offer the designer greater flexibility, more rationality of design, and possible overall economy.

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