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Recent and Impending Major Changes in U.S. Seismic Codes

S.K. Ghosh[1]

ABSTRACT

This paper discusses perhaps the most important of the changes that have been made in the seismic design provisions of the 1994 edition of the Uniform Building Code (UBC). This change introduces a more rational procedure for the design of reinforced concrete shearwalls in bending and axial load, which replaces the design procedure that is still in the ACI 318-89 (Revised 1992) Standard.

Following the widespread cracking of beam-column joints in special moment-resisting frames of steel in the Northridge earthquake of January 17, 1994, an emergency change to the 1994 UBC has been approved. Details of this important code change are given in the paper.

Being considered for the 1997 edition of the UBC is the replacement of the current soil factors with a much more elaborate set of soil factors that would depend not only on the soil profile at the location of the structure, but also on the seismicity of that location. This paper attempts to explain the proposed changes with major potential impact on seismic design.

DESIGN OF SHEARWALLS UNDER BENDING AND AXIAL LOAD

Shearwalls in buildings have been used by many engineers as the most efficient way of resisting lateral forces. In regions of high seismicity, shearwalls in buildings are even more beneficial than in regions of lower seismicity. Well-designed shearwalls have been found to reduce story drifts, and attract much of the lateral forces, thus reducing the demand on other structural components.

Shearwalls should be designed: 1) To obtain sections that are capable of developing the required strength in combined flexure and axial compression or tension, and 2) To ensure that shear strength is higher than the shear demand. In addition, a shearwall needs to be detailed for a certain degree of inelastic deformability.

Design requirements contained in ACI 318-89 for reinforced concrete shearwalls in Seismic Zones 3 and 4, which were adopted into the 1991 edition of the UBC, seek to meet the above objectives, but are seriously flawed. According to these requirements, a shearwall must be provided with boundary elements at the ends, if the extreme compression

[1] Director, Engineering Services, Codes and Standards, Portland Cement Association, Skokie, IL. 60076

fiber stress caused by the entire factored axial load tributary to the wall and the entire factored overturning moment at the base of the wall exceeds 0.2 f'_c. The stresses are calculated using a linearly elastic model of the shearwall and gross section properties. Boundary elements, where required, must be designed to carry the entire factored axial load tributary to the wall, and the entire factored moment at the base of the wall by tension and compression in the boundary elements. In other words, the boundary elements are required to be designed, completely ignoring the fact that the shearwall web even exists. This gives rise to oversized boundary elements typically containing a large number of longitudinal reinforcing bars. The code further requires that these bars be tied and cross-tied at a maximum spacing of 4 in. (100 mm), with every other bar laterally supported by a corner of a tie or crosstie. All these requirements result in boundary elements that are not only unnecessarily uneconomical, but are very difficult to construct. The result has been that buildings that could benefit from inclusion of shearwalls are being constructed without them. Also, the oversized boundary elements make the shearwalls over-strong in flexure, thereby potentially attracting larger shear forces to them in actual earthquakes than what the walls are designed for, making premature shear failure more likely.

Wood (1991a), in research carried out at the University of Illinois, examined the effects of boundary element transverse reinforcement amounts lower and higher than that required by ACI 318 on the deformability of shearwalls. It was found that higher confinement translated into higher deformability as long as a wall failed in flexure. However, the deformability of a wall failing in shear was independent of such confinement.

Wood (1991b), and later Wallace and Moehle (1993), also determined that the ACI/UBC boundary element confinement requirements are excessive, except when the ratio of shearwall cross-sectional area to floor plan area is very low (on the order of half a percent).

Design provisions for shearwalls subject to combined bending and axial loads have been made much more sensible and rational in UBC-94. The new provisions have the following primary features:

1. A shearwall is designed for flexure and axial load considering the entire cross-section, including web(s), to be effective, as in a short column . Shear resistance is still provided by the web, without any contribution from the overhanging flanges.

2. Wall is screened to eliminate cases where special boundary zone detailing is not required. Walls having $P_u \leq 0.10 A_g f'_c$ and either $M_u/V_u \ell_w \leq 1.0$ or $V_u \leq 3 \ell_w h \sqrt{f'_c}$ are exempt. Walls with $P_u > 0.35 P_o$ are not permitted to resist earthquake-induced forces.

3. Two options are provided for cases where boundary elements with special details are needed: (a) Conservative approach: provide boundary elements over 0.25 ℓ_w at each end; (b) Alternatively, determine compressive strains at wall edges when wall is subject to design earthquake displacements, using cracked section properties. Provide confinement wherever compressive strain exceeds 0.003.

The detailing requirements for confinement reinforcement, when needed, are much less stringent than in ACI 318-89 (Revised 1992) and UBC-91. The maximum spacing is 6 in. (150 mm) or 6 times the longitudinal bar diameter, whichever is smaller, rather than 4 in. (100 mm).

High-strength concrete shearwalls have not been tested under reversed cyclic lateral loading in this country so far. A significant number of them have been so tested in Japan in recent years. The UBC-94 confinement requirements need to be examined in the light of results from such tests, as they become available.

SPECIAL MOMENT-RESISTING FRAMES OF STEEL

Performance in the Northridge Earthquake

The biggest story to emerge out of the Northridge earthquake is the surprisingly poor performance of many steel buildings. The most serious reports of damage concern brittle failures in connections of special moment-resisting steel frames.

Approximately 110 buildings with welded steel moment frames are known to have suffered unexpected, and often extensive, fractures in or near girder-to-column welds. The fractures generally occurred at the heel of the beam flange to column flange weld, and in many cases continued through the column flange/web or the beam flange/web right above the weld.

Current design procedures presume that these joints can maintain their structural integrity through numerous cycles well into the inelastic range of deformation. As such, the failures invalidate the very basis of current design and construction practices for steel frames. Buildings of one to 27 stories have been affected, with the majority in the range below six stories. Most of the damage occurred to structures of recent construction, although buildings up to 20 years old have also experienced weld fractures.

Failure of 50 to 80 percent of the moment resisting connections in a building has not been uncommon. The ability of the steel joints to resist rotation of the beams relative to the columns, and thereby withstand seismic forces, has diminished with the failure of the welds. The decreased stiffness resulted in structures more susceptible to damage during after-shocks and future earthquakes, making them highly vulnerable and possibly posing a severe life-safety hazard.

The damage to steel frame structures from the Northridge earthquake is apparently unprecedented. However, this is one of the first major earthquakes that occurred in an urban area containing a significant number of modern steel structures. The fact that damaged buildings showed few outward signs of distress and the difficulty and costs involved in detecting internal damage suggest that similar damage may have occurred in the past and gone undetected.

An Emergency Code Change

The Board of Directors of the International Conference of Building Officials (ICBO) have approved an emergency change to the 1994 edition of the Uniform Building Code (UBC), deleting Section 2211.7.1.2. This section permitted the design and construction of welded steel moment frame connections without testing or much calculation, provided a few simple rules were followed.

Steel moment frames in Seismic Zones 3 and 4 will still be allowed under the new Section 2211.7.1.2 (previously Section 2211.7.1.3), which now reads:

"Connection configurations utilizing welds or high strength bolts shall demonstrate by approved cyclic test results or calculations the ability to sustain inelastic rotation and develop the strength criteria in Section 2211.7.1.1 considering the effect of steel over-strength and strain-hardening."

The Seismology Committee of the Structural Engineers Association of California (SEAOC) is in the process of developing a formal interpretation to explain the above functional requirement. The "ability to sustain inelastic rotation," in particular, requires explanation.

Continuing Developments

Although the UBC provision governing welded steel moment frame connections has been set aside, agreed-upon construction alternatives, even on an interim basis, have not evolved. The American Institute of Steel Construction (AISC) issued interim guidelines in March 1994 for adding steel plates to beam flanges and adhering closely to current welding specifications. AISC has since updated the recommendations, partly on the basis of tests at the University of Texas at Austin. The tests concluded at the University of Texas have so far indicated that the most cost-effective way to reinforce a new beam-to-column connection involves spending nearly three times as much as what used to be needed before the earthquake to create a typical moment connection.

The National Science Foundation has awarded a number of grants in support of further research. A SAC Joint Venture of SEAOC, the Applied Technology Council (ATC), and the California Universities for Research in Earthquake Engineering (CUREE) has been formed. The joint venture has proposed to undertake an unprecedented $18.4 million, three-year research effort. It was funded in late 1994 for a four-month effort leading to the development of interim recommendations.

While interim guidelines will no doubt be developed, probably by several sources, it may be several years before all the research now starting will be sufficiently advanced to permit a consensus by the structural engineering community on appropriate use, deployment, and detailing of steel moment frame systems in the aftermath of the Northridge earthquake.

AMPLIFICATION/ATTENUATION OF GROUND MOTION DUE TO SITE SOIL CONDITIONS

The United States has three model codes, one of which is adopted by almost every local jurisdiction. The seismic design provisions of one, the Uniform Building Code, are based on the so-called SEAOC Blue Book (SEAOC, 1990). The seismic design provisions of the other two are based on the NEHRP Provisions (BSSC 1991), which is a successor document to ATC 3-06 (ATC 1978).

ATC 3-06 as well as UBC-82 considered three Soil Profile Types to be different enough in seismic response to warrant separate seismic coefficients (S factors). Experience from the September 1985 Mexico earthquake prompted the addition of a fourth Soil Profile Type. The need for improvement in codifying site effects was discussed at a 1991 National Center for Earthquake Engineering Research (NCEER) workshop (Whitman, 1992) which made several general recommendations. A committee was formed during that workshop to pursue resolution of pending issues and develop specific code recommendations. The committee collected information, guided related research, discussed the issues, and organized a November 1992 site response Workshop in Los Angeles (Martin, 1994). This workshop discussed the results of a number of empirical and analytical studies and approved consensus recommendations that formed the basis of extensive modifications to the consideration of site effects in the 1994 NEHRP Provisions.

The design base shear equation in 1994 NEHRP is going to change from:

$$V = \left(\frac{1.2A_vS}{RT^{2/3}}\right)W \le \left(\frac{2.5A_a}{R}\right)W \qquad (1)$$

to:

$$V = \left(\frac{1.2C_v}{RT^{2/3}}\right)W \le \left(\frac{2.5C_a}{R}\right)W \qquad (2)$$

where:

$$C_a = F_a \ A_a \qquad (3)$$

and:

$$C_v = F_v \ A_v \qquad (4)$$

The values for site coefficients F_a and F_v are as indicated in Tables 1a and 1b, respectively. Seismic coefficient C_a based on Soil Profile Type and A_a is determined from Table 2a, while seismic coefficient C_v based on Soil Profile Type and A_a is determined from Table 2b.

Five different soil profile types have been defined in the 1994 NEHRP Provisions, as follows:

A. Hard rock with measured shear wave velocity, $\bar{v}_s$ < 5000 ft/sec (1520 m/s).

B. Rock with 2500 ft/sec (760 m/s) < $\bar{v}_s \le$ 5000 ft/sec (1520 m/s).

C. Very dense soil and soft rock with 1200 ft/sec (365 m/s) < $\bar{v}_s \le$ 2500 ft/sec (760 m/s) or with either $\bar{N}$ > 50 or $\bar{s}_u$ > 2000 psf (95.8 kPa).

D. Stiff soil with 600 ft/sec (183 m/s) $\le \bar{v}_s \le$ 1200 ft/sec (365 m/s) or with either $\bar{N} \le$ 50 or 1000 psf (47.9 kPa) $\le \bar{v}_s \le$ 2000 psf (95.8 kPa).

E. Any profile with more than 10 ft (3m) of soft clay defined as soil with PI > 20, w >

40 percent and $\bar{s}_u \leq$ 1000 psf (479 kPa) or a soil profile with $\bar{v}_s <$ 600 ft/sec (183 m/s).

F. Soils requiring site-specific evaluations:
1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils.

2. Peats and/or highly organic clays (H > 10 ft or 3 m of peat or highly organic clay).

3. Very high plasticity clays (H > 25 ft or 7.6 m with PI > 75 percent).

4. Very thick soft/medium stiff clays (H > 120 ft or 37 m).

When soil properties are not known in sufficient detail to determine the Soil Profile Type, Type D is to be used. Soil Profile Type E need not be assumed unless the building official determines that Soil Profile Type E may be present at the site or in the event that Type E is established by geotechnical data.

The above site classification system recognizes the primary importance of the types of soil materials and especially their shear wave velocities in the top 100 ft (30.5 m) of the site profile, as well as the need to consider the difference in response of soils of different stiffnesses. The low- and high-period spectral site coefficients F_a and F_v in Tables 1a and 1b are functions of both site class and level of shaking.

The above scheme of site effects consideration is being studied by the Seismology Committee of the Structural Engineers Association of California who may recommend adoption of the scheme, with necessary modifications, in the 1997 edition of the Uniform Building Code.

TABLE 1a, Values of F_a as Function of Site Conditions and Shaking Intensity

Shaking Intensity	A_a	A_a	A_a	A_a	A_a
Soil Profile Type	≤0.1	=0.2	=0.3	=0.4	>0.5
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	a
F	a	a	a	a	a

TABLE 1b, Values of F_V as Function of Site Conditions and Shaking Intensity

Shaking Intensity / Soil Profile Type	$A_V \le 0.1$	$A_V = 0.2$	$A_V = 0.3$	$A_V = 0.4$	$A_V > 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	a
F	a	a	a	a	a

NOTE: Use straight line interpolation for intermediate values of A_a or A_V.
[a] Site specific geotechnical investigation and dynamic site response analyses should be performed.

TABLE 2a, Seismic Coefficient C_a

Soil Type	A_a <0.05	A_a =0.05	A_a =0.10	A_a =0.20	A_a =0.30	A_a =0.40	A_a ≥0.5
A	Aa	0.04	0.08	0.16	0.24	0.32	0.40
B	Aa	0.05	0.10	0.20	0.30	0.40	0.50
C	Aa	0.06	0.12	0.24	0.33	0.40	0.50
D	Aa	0.08	0.16	0.28	0.36	0.44	0.50
E	Aa	0.13	0.25	0.34	0.36	0.36	a

Table 2b, Seismic Coefficient C_v

Soil Type	A_V <0.05	A_V =0.05	A_V =0.10	A_V =0.20	A_V =0.30	A_V =0.40	A_V ≥0.5
A	A_V	0.04	0.08	0.16	0.24	0.32	0.40
B	A_V	0.05	0.10	0.20	0.30	0.40	0.50
C	A_V	0.09	0.17	0.32	0.45	0.56	0.65
D	A_V	0.12	0.24	0.40	0.54	0.64	0.75
E	A_V	0.18	0.35	0.64	0.84	0.96	a

NOTE: For intermediate values, the higher value or straight-line interpolation shall be used to determine the value of C_a or C_V.
[a] Site specific geotechnical investigation and dynamic site response analyses should be performed.

NOTATION

A_a - Effective peak acceleration coefficient
A_g - Gross cross-sectional area

A_v - Effective peak velocity related acceleration coeffienct
Ca - Seismic coefficient based on Soil ProfileType and value of Aa
Cv - Seismic coefficient based on Soil Profile Type and value of Av
Fa - Acceleration-based (short-period) site factor
Fv - Velocity-based (long-period) site factor
f'_c - Specified compressive strength of concrete
h - Overall thickness of member
H - Thickness of soil
ℓ_w - Length of wall in direction of shear force considered
M_u - Factor w moment at section
$\bar{N}$ - Average field standard penetration resistance for the top 100 ft (30.5 m)
P_o - Nominal axial load strength at zero eccentricity
P_u - Factored axial load at given eccentricity
PI - Plasticity index, ASTM D4318
R - Response modification factor
$\bar{s}_u$ - Average undrained shear strength in top 100 ft (30.5 m), ASTM D2166 or ASTM D2850
S - Site coefficient for soil characteristics
T - Fundamental period of vibration of structure in direction under consideration
$\bar{v}_s$ - Average shear wave velocity in top 100 ft (30.5 m)
V_u - Factored shear force at section
w - Moisture content (in percent), ASTM D2216
W - Total seismic dead load

REFERENCES

ACI 318-89 (Revised 1992). 1992. Building Code Requirements for Reinforced Concrete. American Concrete Institute, Detroit, Michigan.

ATC 3-06. 1978. Tentative Provisions for the Development of Seismic Regulations for Buildings. Applied Technology Council, Palo Alto, California.

Martin, G.M., Editor, 1994. Proceedings of the NCEER/SEAOC/BSSC Workshop on Site Response During Earthquakes and Seismic Code Provisions. University of Southern California, Los Angeles, California.

NEHRP. 1991. Recommended Provisions for the Development of Seismic Regulations for New Buildings. Building Seismic Safety Council, Washington, D.C.

SEAOC. 1990. Recommended Lateral Force Requirements and Commentary. Structural Engineers Association of California, Sacramento, California.

UBC. 1994. Uniform Building Code. International Conference of Building Officials, Whittier, California.

Wallace, J.W., Moehle, J.P. 1993. Ductility and Detailing Requirements of Bearing Wall Buildings. Journal of Structural Engineering. ASCE. Vol. 118, No. 6, 1625-1644.

Whitman, R., Editor. 1992. Proceedings of the Side Effects Workshop, October 1991.

Report NCEER-92-0006. National Center for Earthquake Engineering Research, Buffalo, New York.

Wood, S.L. 1991a. Observed Behavior of Slender Reinforced Concrete Walls Subjected toCyclic Loading. Publication SP-127. American Concrete Institute, Detroit, Michigan. 453-477.

Wood, S.L. 1991b. Performance of Reinforced Concrete Buildings During the 1985 Chile Earthquake. Implications for the Design of Structural Walls. Earthquake Spectra. EERI. Volume 7, No. 4, 607-638.