Evolutionary Features of the New International Building Code (IBC-2000)

by

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Abstract

Starting in the year of 2000, the International Building Code (IBC-2000) replaces several US building codes including SBC, BOCA, and UBC. Thus IBC-2000 represents national uniformity in building codes. This paper outlines the Code's analysis procedures and related parameters with conclusion on the significant differences between UBC-97 and IBC-2000.

Introduction

Several seismic building codes such a Uniform Building Code-UBC (1997)[5], Standard Building Code-SBC (1994)[6] and BOCA (1996)[2] have been adopted in various regions in the U.S. Among these codes, the UBC is widely disseminated and recognized in the international engineering community. The UBC was first enacted by the International Conference of Building Officials in 1927. Since then, the code has been revised every three years (except during World War II). International Building Code of the year 2000, IBC-2000, was published as first and final draft in 1977 and 1998, respectively, and became the official code in May, 2000. It incorporates the above codes promulgated by representatives of BOCA, SBCCI, and ICBO. Thus IBC-2000 represents national uniformity in building codes and supersedes other codes such as UBC-97. In principle, the average annual risk of UBC is 10% probability of being exceeded in any 50-year period. This means that if minimum design lateral-force requirements in the codes are met, then normal building structures may be expected to resist an upper-level earthquake with a recurrence interval of 475 years without collapse and without endangering life. However, structural and nonstructural damages are anticipated which may not be acceptable for essential facilities, such as hospitals and communication centers. A higher-level design force is required to ensure their ability to function during and after an earthquake. With the same design philosophy as UBC, IBC-2000 has only a 2% probability of being exceeded in any 50-year period.

IBC-2000 classifies structures into two categories, seismically isolated and seismically non-isolated. For both seismically isolated and non-isolated structures, two procedures apply to finding seismic forces: (1) equivalent lateral force; and (2) dynamic lateral force including time-history analysis. This paper focuses the two procedures for non-isolated structures for which selecting the appropriate procedure depends on structural irregularities and on structural design categories A-F. These categories are determined by seismic use groups I, II, and III and design spectral response acceleration coefficients S_{DS} and S_{D1}, at short and 1 sec periods, respectively. The classification of seismic use groups is given in IBC-2000, Table 1604.5; the structural design categories classified in Tables 1616.3(1) and (2) for which S_{DS} and S_{D1} are found by using code Equations 16-16 through 16-19 to scale S_S and S_1 (maximum spectral response acceleration at short period and 1 sec period, respectively) from IBC-2000, Figures 1615(1) and (2). When design response acceleration at 1 sec period, S_{D1}, exceeds 0.75g, structures in groups I and II should be assigned to seismic design category E, and structures in group III should be assigned to seismic design category F.

IBC-2000 requires that structures assigned to seismic design category A use a simplified analysis defined as minimum lateral force (Section 1616.4.1), structures in categories B and C use equivalent lateral force or a more rigorous analysis, and structures in D, E, and F use the dynamic analysis procedure (Section 1616.6 and Table 1616.6.3).

(I) Equivalent Lateral-Force Procedure and Related Parameters

In IBC-2000 equivalent lateral-force procedure, the total base shear is specified according to design categories and can be summarized in (A) through (D) as follows.

(A) *For seismic design categories B through D*

$$V=\min\left\{\begin{array}{l}\dfrac{S_{D1}}{\left(\dfrac{R}{I_E}\right)T}W;\\ \max\left[\dfrac{S_{DS}}{\left(\dfrac{R}{I_E}\right)}W;\quad 0.044S_{DS}I_EW\right]\end{array}\right\} \qquad (1)$$

(B) *For seismic design categories E and F and structures for which* $S_1 \geq 0.6g$

$$V=\min\left\{\begin{array}{l}\dfrac{S_{D1}}{\left(\dfrac{R}{I_E}\right)T}W;\\ \max\left[\dfrac{S_{DS}}{\left(\dfrac{R}{I_E}\right)}W;\quad \dfrac{0.5S_1}{\left(\dfrac{R}{I_E}\right)}W;\quad 0.044S_{DS}I_EW\right]\end{array}\right\} \qquad (2)$$

where S_1 shall be determined from seismic maps.

(C) *For seismic design category A* – simplified analysis can be used as

$$V = 0.01W \qquad (3)$$

(D) *For seismic design categories B through F of Seismic Use Group I* – simplified analysis can be used for light-frame construction not exceeding three stories in height, and any construction other than light frame not exceeding two stories in height with flexible diaphragms at every floor (building is fixed at the base)(Section 1617.5.1)

$$V=\frac{1.2S_{DS}}{R}W \qquad (4)$$

S_{DS} and S_{D1} are design spectral response acceleration coefficients at short and 1 sec periods, respectively; I_E is the seismic importance factor given in Table 1604.5; R is the response modification factor given in Table 1617.6 and W is the seismic dead load.

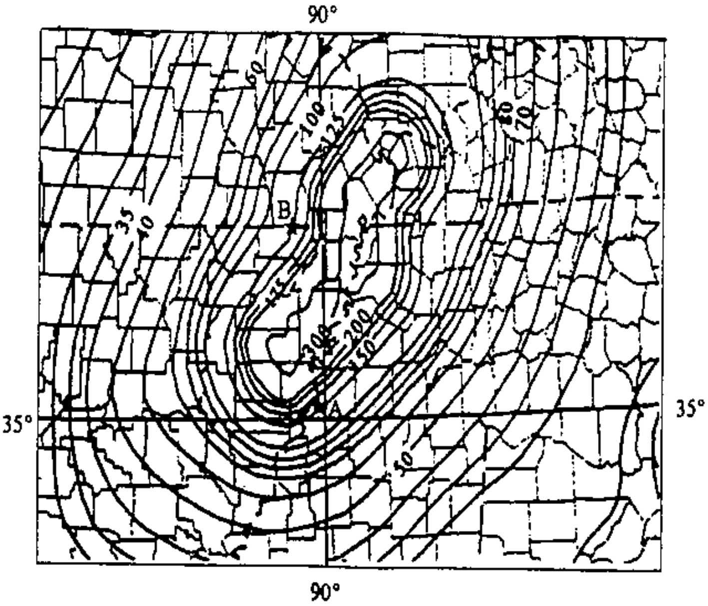

Fig. 1 S_S for 0.2 sec spectral response acceleration (%g) site class B, ★A: City of Memphis

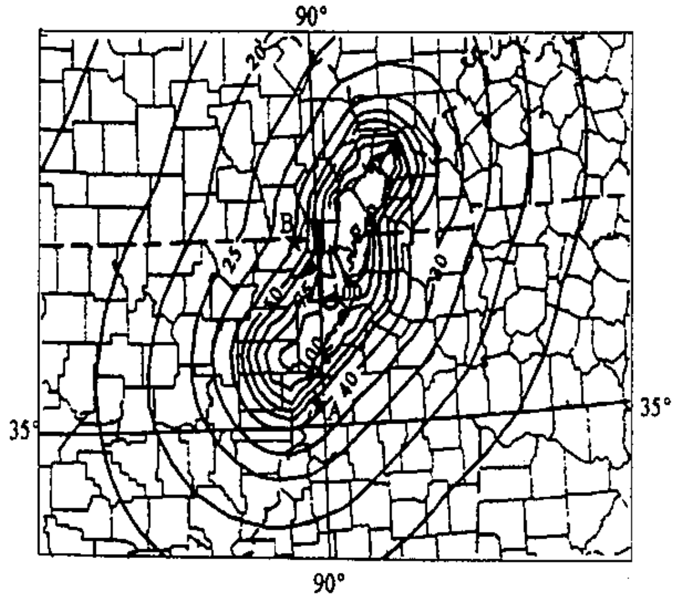

Fig. 2 S_1 for 1 sec spectral response acceleration (%g) site class B, ★A: City of Memphis

(E) *Design spectral response acceleration coefficients* S_{DS} *and* S_{D1} – Regionalization of the maximum considered earthquake ground motion for each site class is given in IBC-2000. From the regionalization maps, 0.2 sec and 1.0 sec spectral response acceleration (S_S and S_1) can be obtained in terms of percentage of g. Figures 1 and 2 show a partial regionalization map near the New Madrid fault for site class B. These two figures illustrate typical contours of maximum earthquake spectral response accelerations at short period, S_S, and 1 sec period, S_1. The adjusted maximum considered earthquake spectral response acceleration for short period,

S_{MS}, and 1 sec period, S_{M1}, can then be obtained from the effects of site class (Section 1615.1.2) as

$$S_{MS} = F_a S_s \tag{5}$$

$$S_{M1} = F_v S_1 \tag{6}$$

where F_a and F_v are site coefficients defined in Table 1615.1.2(1) and 2(2).

The design spectral response acceleration at short period, S_{DS}, and 1 sec period, S_1, can be obtained as

$$S_{DS} = \frac{2}{3} S_{MS} \tag{7}$$

$$S_{D1} = \frac{2}{3} S_{M1} \tag{8}$$

which are used in Eqs. (1), (2) and (4) to determine base shear.

(a) Vertical and Horizontal Distribution of Lateral Forces

After base shear, V, is determined, the vertical distribution of the shear over the height of a structure as lateral forces at each floor level can be obtained as

$$F_x = C_{vx} V \tag{9}$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{10}$$

where w_x and w_i are weight at h_x and h_i are height of a particular level above the base; the k factor takes into account a flexible structure's higher mode shapes.

The horizontal shear distribution at any floor level x is derived by summation of the lateral forces distributed above that level as

$$V_x = \sum_{i=1}^{n} F_i \tag{11}$$

The overturning moment at x level, M_x, is the sum of the moments caused by lateral forces above that level. At lower levels, M_x needs to be reduced through multiplying the overturning moment by a reduction factor τ as (Section 1617.4.5):

$$M_x = \tau \sum_{i=x}^{n} F_i (h_i - h_x) \tag{12}$$

(b) Deflection and Story Drift

IBC-2000 (Section 1617.4.6.1) requires that design story drift, Δ, be computed as the difference between the deflections of the mass center at the top and at the bottom of the story under consideration. For structures assigned to seismic design category C, D, E or F having a plan with torsional irregularities, design story drift Δ is computed as the largest difference between deflections along any edge of the structure at the top and bottom of the story under consideration.

At level x, deflection δ_x is determined as

$$\delta_x = \frac{C_d \delta_{xe}}{I_E} \tag{13}$$

where δ_{xe} is deflection found by elastic analysis of the seismic force-resisting system. Hence δ_{xe} multiplied by deflection amplification factor, C_d, gives inelastic deflection within the code's ductility range. Designed story drift should not exceed allowable story drift Δ_a (i.e. $\Delta \le \Delta_a$). Δ_a is given in Table 1617.3 and is mainly based on construction material, characteristics of a structural system, and seismic use group. The P-Δ effect on story shears and moments, the resulting member forces and moments, and story drifts thus induced need not be considered when the stability coefficient, θ, is equal to or less than 0.1 (Section 1617.4.6.2).

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d} \tag{14}$$

$$\theta_{max} = \frac{0.5}{\beta C_d} \le 0.25 \tag{15}$$

where θ cannot exceed θ_{max}. β is the ratio of shear demand to shear capacity for the story between level x and x-1. When this ratio is not calculated, a value of 1.0 is used. If θ is greater than θ_{max}, then a structure is potentially unstable and must be redesigned.

(II) Dynamic Analysis Procedure

For dynamic analysis of non-isolated building structures, a modal analysis procedure is provided in detail; time-history analysis is permitted but only general guidelines are given. While modal analysis procedure is applicable to all kinds of building structures, the structures in seismic design categories D, E, and F (Tables 1616.3(1), (2) and Table 1616.6.3) must be designed by using the procedure. General guidelines for using modal analysis can be abstracted as follows:

(A) All structures, regular or irregular, over 240 ft in height.

(B) Structures with all of the following characteristics:
1. located in an area where $S_{D1} \ge 0.2$; 2. located in an area assigned to site class E or F; 3. having natural period $T \ge 0.7$ sec, as determined by the approximate method (Section 1617.4.2).

For structures described in (B), a site-specific response spectrum must be used but design base shear cannot be less than that determined by equivalent lateral-force procedure.

Some IBC requirements for dynamic lateral-force procedure are similar to those for static lateral-force procedure. Only the design criteria related to spectrum analysis are summarized below. Step-by-step dynamic analysis must be interwoven with certain criteria required in equivalent lateral-force procedure.

(A) IBC-2000 requires a sufficient number of modes to be used so that 90% of structural mass contributes to system response (Section 1618.2).

(B) SRSS and CQC can both be used for modal, force, and displacement combination (Section 1618.7). For the SRSS method, SEAOC recommendations in Section c106.4.1.2 should be maintained.[7]

(C) When design base shear V calculated by equivalent lateral-force procedure (Section 1617.4.1) is larger than modal base shear V_t, the design story shears, moments, drifts and floor deflections are multiplied by modification factor C_m (Section 1618.7) defined as

$$C_m = V/V_t \tag{16}$$

(D) A mathematical model should be constructed to represent spatial distribution of mass and stiffness throughout the structure. For regular structures with independent orthogonal seismic-force resisting systems, independent two-dimensional models may be constructed to represent each system. For irregular structures without independent orthogonal systems, a three-dimensional model must be constructed. At minimum, three dynamic d.o.f. consisting of translation in two orthogonal plan directions and torsional rotation about the vertical axis must be included at each level of the building. Additional dynamic d.o.f. are required to represent diaphragm flexibility where the diaphragm is not rigid relative to the rigidity of the vertical elements of the lateral force resisting system. This model must include the effect of cracked sections on stiffness properties of concrete and masonry elements, and the contribution of panel zone deformations to overall story drift for steel moment frame systems.

(E) The distribution of horizontal shear is the same as for the equivalent lateral force method except that explicit amplification due to torsion is not required because torsional d.o.f. are included in the modal analysis model (Section 1618.8).

(F) The P-Δ effect on story drifts and shears is determined in the same way as equivalent lateral force procedure (Section 1618.9).

(G) Soil-structure interaction effects, although not required, can be considered on the basis of ASCE 7-98, Section 9.5.5 [1].

(a) Importance of EPA and EPV as Parameters for Seismic Zone Maps

Design ground-shaking should have an intensity, which can be characterized by two parameters: effective peak acceleration (EPA) and effective peak velocity (EPV). The relationship between EPA and S_a (smoothed spectrum of peak ground acceleration) and between EPV and S_v (smoothed spectrum of peak ground velocity) with a proportional value of 2.5 is shown in Fig. 3. This figure shows that EPA is proportional to S_a for periods between 0.1 and 0.5 sec, while EPV is proportional to S_v at a period of about 1 sec. EPA is related to but not necessarily proportional to peak ground acceleration (S_a) as is EPA to peak ground velocity (S_v). In fact, with ground motion at very high frequencies, EPA may be significantly less than peak acceleration (see Fig. 3 for period < 0.1). On the other hand, EPV is generally greater than peak velocity at long distances from a major earthquake. Ground motions, then, increase in duration and become more periodic with distance, which tends to increase EPV. Post-earthquake studies reveal that two motions of different duration but similar response spectrum cause different degrees of damage. This damage is usually less with motion of shorter duration. Thus EPV is an important parameter in developing a seismic zone map.

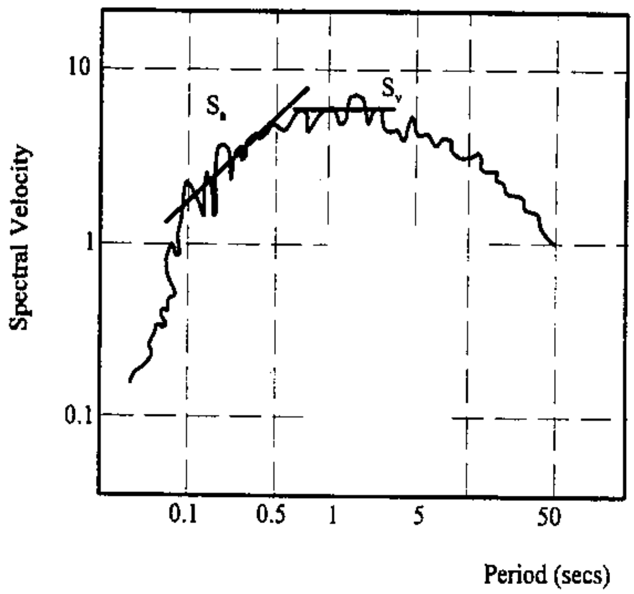

Fig. 3 Schematic representation of S_a and S_v

In IBC-2000, design spectral accelerations, S_{DS} and S_{D1} are control parameters of design response spectra. S_{DS} and S_{D1} are based on mapped spectral accelerations, S_S, and S_1, respectively, for which S_S is at 0.2 sec and S_1 is at

1 sec periods. The value of 0.2 sec is about the average period for EPA, and the value of 1.0 sec is about the average period for EPV.

Conclusions

Major differences among UBC-97 and IBC-2000 are summarized as follows:

(A) The response spectra in IBC-2000 are similar to UBC-97. Fault location and characteristics of strong ground motion associated with a fault are roughly estimated in tabulated form for UBC-97, but comprehensively considered in regionalization maps in IBC-2000.

(B) IBC-2000 seismic zone maps contain greater detail than those in UBC, featuring acceleration and velocity-related acceleration as well as highly defined regions. Regionalized seismic zones are more comprehensive and display more contours between zones.

(C) IBC-2000 considers higher modes with distribution exponent, k, related to a range of building periods; UBC-97 use a force, F_t, applied to the top of any building with a period greater than 0.7 sec.

(D) Reduction factor is used for overturning moment in IBC-2000 but not in UBC-97.

It should again be noted that the organization of sections, figures, tables and the like in IBC-2000 differs from UBC-97. Therefore, comparisons and step-by-step numerical procedures are given in parallel layout with the hope that the reader can easily grasp the specific requirements of various sections of these two codes[3].

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