Impact of Seismic Design Provisions of 2000 IBC: Comparison with 1997 UBC

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Abstract

This paper examines the potential impact of the 2000 *International Building Code* (IBC) on seismic design in parts of the country where the *Uniform Building Code* is adopted. This is done through a generalized comparison of detailing requirements and seismic design forces at a number of specific locations where the UBC is used. The impact is also examined through comparative seismic designs of a number of reinforced concrete buildings with varying design parameters by the 2000 IBC and the 1997 UBC.

Introduction

The earthquake regulations of Sections 1613 through 1623 of the 2000 IBC (ICC 2000), based on the 1997 NEHRP Provisions (BSSC 1997), are substantitally different from the corresponding provisions of the 1997 UBC (IBCO 1997). The differences of relevance to this impact study are first discussed below. This is followed by a generalized comparison of detailing requirements and seismic design forces for a number of specific locations within UBC jurisdiction. Comparative seismic designs of buildings by the 2000 IBC and the 1997 UBC are next reported on.

Design Ground Motion Parameters

The biggest change from the 1997 UBC to the 2000 IBC is in the design ground motion parameters which are now S_{DS} and S_{D1}, rather than Z. S_{DS} and S_{D1} are 5%-damped design spectral response accelerations at short periods and 1 sec. period respectively. S_{DS} determines the upper-bound design base shear (the "flat-top" of the design spectrum) used in seismic design (see IBC Fig. 1615.1.4-1, reproduced here as Fig. 1, and Section 1617.4). S_{D1} defines the descending branch or the period-dependent part of the design spectrum (see IBC Fig. 1615.1.4-1 and Section 1617.4). The seismic zone map of the UBC has been replaced by contour maps giving two quantities from which S_{DS} and S_{D1} are to be derived. The mapped quantities are the Maximum Considered Earthquake spectral response accelerations S_S (at short periods) and S_1 (at 1 sec. period).

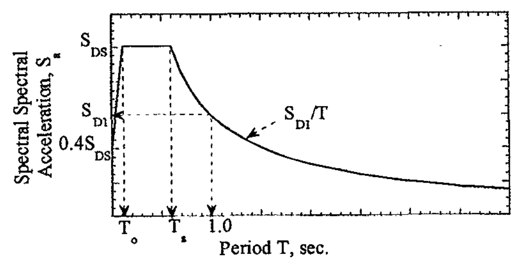


Fig. 1. Design Response Spectrum

The maximum considered earthquake is the 2500-year return period earthquake (2% probability of exceedance in 50 years) in most of the country, except that in coastal California, it is the largest (deterministic) earthquake that can be generated by the known seismic sources. The design earthquake of the IBC is two-thirds of the MCE, whereas the design earthquake of the 1997 UBC has a return period of 475 years (10% probability of exceedance in 50 years). The two-thirds is the reciprocal of 1.5 which is agreed to be the "seismic margin" built into structures designed by the UBC or older editions of the NEHRP Provisions. In other words a structure designed by the UBC or older editions of the NEHRP Provisions is believed to have a low likelihood of collapse under an earthquake that is one and one-half times as large as the design earthquake of those documents. The redefinition of the design earthquake in the IBC is intended to provide a uniform level of safety across the country against collapse in the Maximum Considered Earthquake. This was not the case before, because the MCE is only 50% larger than the design earthquake of the UBC in coastal California, while it can be four or five times as large as the design earthquake of the UBC in the Eastern United States. Z of the 1997 UBC indicates effective peak groiund acceleration (more exactly, the larger of effective peak acceleration or effective peak velocity-related acceleration) expected within a seismic zone corresponding to the design earthquake of the UBC on Type S_B soil or soft rock. The mapped MCE spectral response accelerations S_S and S_1 of the 2000 IBC are also mapped on Type S_B soil.

S_{DS} and S_{D1} are two-thirds of S_{MS} and S_{M1} which are the soil-modified (Maximum Considered Earthquake) spectral response accelerations at short period and 1 sec. period, respectively. S_{MS} is obtained by multiplying the mapped MCE spectral response acceleration S_S (at short periods) by F_a, the acceleration-related soil factor. S_{M1} is similarly obtained by multiplying the mapped MCE spectral response acceleration S_1 (at 1 sec. period) by F_v, the velocity-related soil factor. F_a and F_v are analogous to C_a/Z and C_V/Z of the 1997 UBC, respectively.

Both the 2000 IBC and the 1997 UBC having adopted the soil classification and the associated site coefficients first introduced in the 1994 NEHRP Provisions, a correlation of ground motion parameters between the two codes is possible. If S_{DS} of the 2000 IBC is equal to $2.5C_a$ and S_{D1} of the 2000 IBC is equal to C_v for a particular location, then the soil-modified seismicity for that site has not changed from the 1997 UBC to the 2000 IBC.

It may be of interest to note that the 1997 edition of the UBC for the first time introduced two near-source factors: acceleration-related N_a and velocity-related N_v, the purpose of which is to increase the soil-modified ground motion parameters C_a and C_v when there are active faults capable of generating large-magnitude earthquakes within 15 kilometers, or 9 miles of a Seismic Zone 4 site. These factors became necessary in view of the artificial truncation of Z-values at 0.4 in UBC Seismic Zone 4. These near-source factors are not in the 2000 IBC because the artificial truncation of ground motion is not a feature of that code. Both S_S and S_1 attain high values in the vicinity of seismic sources that are judged capable of generating large earthquakes.

Seismic Design Category

In the Uniform Building Code (ICBO 1997), the Seismic Zone in which a structure is located determines permissible structural systems including the level of detailing required for structural members and joints that are part of the lateral-force-resisting system and for the structural components that are not, limitations on height of structure and structural irregularity, the type of lateral load analysis that must be performed as the basis of design, as well as nonstructural component requirements. The 1994 and prior editions of the NEHRP Provisions (BSSC 1994) as well as the predecessor document, ATC 3 (ATC 1978), used a Seismic Performance Category (SPC) for the above purposes. The SPC was a function of occupancy (called Seismic Hazard Exposure Group in the documents being discussed) and of the seismic risk at the site of the structure in the form of the peak velocity-related acceleration coefficient, A_V. Through this device, a hospital in an area of low seismic risk was required to be detailed like, and be subject to the same restrictions as, an office building in an area of high seismic risk.

The 2000 IBC and the 1997 NEHRP Provisions have replaced the Seismic Performance Category (SPC) with a Seismic Design Category (SDC) which plays the same role as the SPC used to play. The SDC is a function of occupancy (called Seismic Use Group in the 2000 IBC and the 1997 NEHRP Provisions) and of **soil-modified** seismic risk at the site of the structure in the form of the design spectral response acceleration at short periods, S_{DS}, and the design spectral response acceleration at 1 sec. period, S_{D1}. For a structure, the SDC needs to be determined twice - first as a function of S_{DS} by IBC Table 1616.3-1 and a second time as a function of S_{D1} by IBC Table 1616.3-2. The more severe category will govern.

When ATC 3 in 1978 made the level of detailing (and other restrictions concerning permissible structural systems, height, irregularity and analysis procedure) a function of occupancy, that was a major departure from UBC practice. The departure was continued in all the NEHRP Provisions through the 1994 edition. Now, in the 2000 IBC and the 1997 NEHRP Provisions, the level of detailing and the other restrictions have been made a function of the soil characteristics at the site of a structure. This is a further major departure from UBC practice, and indeed from current practice across the country - a move that is likely to have significant impact on the economic and other aspects of earthquake-resistant construction.

Importance Factor

ATC 3, the predecessor document to NEHRP, deliberately decided to drop the importance factor that has been used in seismic design by the UBC for a long time. ATC 3 chose to institute two other requirements instead to ensure enhanced performance of structures in higher occupancy categories. Very importantly, it made drift limits tighter for structures in higher occupancy categories. Secondly, as mentioned, it made the level of detailing and other restrictions a function not only of the seismic risk at the site of a structure, but also of the occupancy of the structure. With these provisions still in place, the IBC has also chosen to bring the importance factor back. The highest value of the importance factor in the IBC is 1.5, whereas the UBC has used a maximum value of 1.25 since 1988.

UBC Seismic Zone versus IBC Seismic Design Category

For a number of chosen specific locations in UBC territory, Tables 1 and 2 show the values of design spectral response acceleration at short period, S_{DS}, and the design spectral response acceleration at 1 sec. period, S_{D1}, respectively. With the Table 1 values of S_{DS}, Seismic Design Categories for the various locations and site classes were determined using the IBC Table 1616.3-1. Similarly, with the Table 2 values of S_{D1}, Seismic Design Categories for the various locations and site classes were redetermined using the IBC Table 1616.3-2. The more severe of the two SDC's for each location and site class is shown in Table 3, along with the 1997 UBC Seismic Zone for the location.

For the purposes of detailing requirements and other restrictions, the UBC Seismic Zones and the corresponding IBC Seismic Design Categories shown in Table 4 may be considered to be approximately equivalent. SDC F structures are Occupancy Category III structures (essential facilities) that are within 15 km of a fault that is capable of generaating large-magnitude earthquakes; SDC E structures are Occupancy Category I and II structures (non-essential facilities) that are similarly located.

Table 3 shows that in five out of the six Zones 3, 4 locations examined, site-specific geotechnical investigation will be required on Site Class E. This would currently not be required by the 1997 UBC. In Sacramento, on firmer soils, only the equivalent of Zone 2 detailing will be required under the 2000 IBC. This is because the seismicity at the location has been judged to be lower by the IBC. In two of the three low-seismicity locations examined: Denver (Seismic Zone 1) and Houston (Zone 0), the equivalent of Zone 2 detailing will be required to be done on Site Class E.

UBC versus IBC Seismic Design Forces

Table 5 shows the ratio of upper-bound seismic design forces (for short-period structures) by the IBC and the UBC. It should be noted that the ratio has three components to it: $S_{DS}/2.5C_a$ (basically, the ground motion component), R_{UBC}/R_{IBC}, and I_{IBC}/I_{UBC}. R_{UBC}/R_{IBC} is typically 1.07 (=1.5/1.4), subject to some difference caused by round-off. I_{IBC}/I_{UBC} has a maximum value of 1.2 (= 1.5/1.25) for essential and hazardous facilities. Thus, $(R_{UBC}/R_{IBC})*(I_{IBC}/I_{UBC})$ has a maximum value of 1.29. Table 5 shows the values of $(S_{DS}/2.5C_a)$ for the locations selected earlier for different site classes. There are significant decreases in this ratio for Sacramento for all site classes; there are significant increases in Seattle on firm soil. Otherwise, the values are close to one. It should be remembered, however, that the IBC design base shears are liable to be up to 29% higher than the values indicated by Table 6.

Comparative Seismic Designs

At the beginning of 1999, BSSC/FEMA funding was provided to S. K. Ghosh Associates Inc., Northbrook, IL, and J. R. Harris & Company, Denver, CO, for a joint project to produce approximately 40 comparative designs using the seismic design provisions of 2000 IBC and 1997 UBC and other codes. Concrete, masonry, steel and wood buildings of different occupancies and heights, employing different structural systems, and located in areas of varying seismicity and on different types of soil, have been designed (Table 7). The results for the concrete buildings designed using the IBC and the 1997 UBC are reported herein.

Twelve-Story Reinforced Concrete Office Building

A typical plan and elevation of the structure considered (Buildings #7-#11 of Table 7) are shown in Figs. 2(a), (b) and 2(c), respectively. The columns and shearwalls have constant cross-sections throughout the height of the building, the bases of the lowest story segments being assumed fixed. The uniformity in member dimensions is adopted mainly for simplicity. The beams and the slabs also have the same dimensions on all floor levels. Although the element dimensions in this example are within the practical range, the structure itself is a hypothetical one. It has been adapted from (Ghosh 1996).

The structure employs a Moment Resisting Frame System for resistance to lateral forces in the longitudinal direction (Buildings #7-9). It employs a Dual System consisting of moment frames and shearwalls in the orthogonal or transverse direction (Buildings #10, 11). The building in the source document uses two shearwalls in the transverse

Table 1: Values of S_{DS}, Design Spectral Response Acceleration at Short Periods for Chosen Locations

LOCATION	S_S	S_{DS} for Site Class A	B	C	D	E
West La[1]	2.06	1.10	1.37	1.37	1.37	**
San Franciso[2]	1.50	0.80	1.00	1.00	1.00	**
Berkeley[3]	2.00	1.06	1.33	1.33	1.33	**
Denver	0.26	0.14	0.17	0.20	0.27	0.43
Sacramento	0.60	0.32	0.40	0.46	0.53	0.60
St. Paul	0.11	0.06	0.07	0.08	0.11	0.18
Seattle	1.50	0.80	1.00	1.00	1.00	**
Portland	1.20	0.64	0.80	0.80	0.82	**
Houston	0.11	0.06	0.07	0.08	0.11	**

Table 2: Values of S_{D1}, Design Spectral Response Acceleration at 1 sec. Period for Chosen Locations

LOCATION	S_1	S_{D1} for site class A	B	C	D	E
West La[1]	0.81	0.43	0.54	0.70	0.81	**
San Franciso[2]	0.65	0.34	0.43	0.56	0.65	**
Berkeley[3]	0.93	0.50	0.62	0.81	0.93	**
Denver	0.06	0.03	0.04	0.07	0.10	0.14
Sacramento	0.26	0.14	0.17	0.26	0.32	0.41
St. Paul	0.015	0.01	0.01	0.02	0.02	0.04
Seattle	0.60	0.32	0.40	0.52	0.60	**
Portland	0.39	0.21	0.26	0.37	0.42	0.63
Houston	0.05	0.02	0.03	0.05	0.12	0.18

Table 3: Seismic Design Category of 2000 IBC vs. UBC Seismic Zone

LOCATION	Zone	Site Class A	B	C	D	E
West La[1]	4[4]	E*	E*	E*	E*	**
San Franciso[2]	4	D	D	D	D	**
Berkeley[3]	4[5]	E*	E*	E*	E*	**
Denver	1	A	B	B	B	C
Sacramento	3	C	C	D	D	D
St. Paul	0	A	A	A	A	B
Seattle	3	D	D	D	D	**
Portland	3	D	D	D	D	**
Houston	0	A	A	A	B	C

[1]On Newport - Inglewood Fault
[2]Downtown, 4th & Market
[3]UC Memorial Stadium
[4]$N_a = 1.3$, $N_v = 1.6$
[5]$N_a = 1.5$, $N_v = 2.0$

*F in the case of Seismic Use Group III buildings
**Site-specific geotechnical investigation and dynamic site response analysis should be performed

Table 4: Approximate Equivalency between UBC Seismic Zones and IBC Seismic Design Categories

1997 UBC Seismic Zone	0, 1	2A, 2B	3, 4
2000 IBC Seismic Design Category	A, B	C	D, E, F

Table 5: Ratio of Design Base Shear - 2000 IBC vs. 1997 UBC

$$\frac{V_{IBC}}{V_{UBC}} = \frac{S_{DS} I_{IBC}}{R_{IBC}} \times \frac{R_{UBC}}{2.5 C_a I_{UBC}}$$

$$= \frac{S_{DS}}{2.5 C_a} \cdot \frac{R_{UBC}}{R_{IBC}} \cdot \frac{I_{IBC}}{I_{UBC}}$$

$$R_{UBC}/R_{IBC} \simeq 1.5/1.4 = 1.07$$

$$\frac{R_{UBC}}{R_{IBC}} \cdot \frac{I_{IBC}}{I_{UBC}} \rightarrow 1.07 \frac{1.5}{1.25} = 1.29$$

Table 6: Ratio of Design Base Shear - 2000 IBC vs. 1997 UBC

		SDS for site class				
LOCATION	Zone	A	B	C	D	E
West La[1]	4[4]	1.05	1.05	1.05	0.96	**
San Franciso[2]	4	1.00	1.00	1.00	0.91	**
Berkeley[3]	4[5]	0.88	0.89	0.89	0.81	**
Denver	1	0.93	0.85	0.89	0.90	0.91
Sacramento	3	0.53	0.53	0.56	0.59	0.67
St. Paul	0					
Seattle	3	1.33	1.33	1.21	1.11	**
Portland	3	1.07	1.07	0.97	0.91	**
Houston	0					

[1]On Newport - Inglewood Fault
[2]Downtown, 4th & Market
[3]UC Memorial Stadium
[4]$N_a = 1.3$, $N_v = 1.6$
[5]$N_a = 1.5$, $N_v = 2.0$

**Site-specific geotechnical investigation and dynamic site response analysis should be performed

Table 7. Selection of Buildings for Comparison of Designs by UBC 1997 and IBC 2000

Building#	Occupancy	Story #	Material	System	Seismicity	Soil	Importance
1	Office	7	Steel	MRF	5	D	1
2	Office	7	Steel	MRF	4	D	1
3	Office	7	Steel	MRF	2	D	1
4	Office	7	Steel	CBF	4	D	1
5	Office	7	Steel	CBF	2	D	1
6	Office	7	Steel	Dual	4	D	1
7	Office	12	CIP conc.	MRF	5	D	1
8	Office	12	CIP conc.	MRF	4	D	1
9	Office	12	CIP conc.	MRF	2	D	1
10	Office	12	CIP conc.	Dual	5	D	1
11	Office	12	CIP conc.	Dual	4	D	1
12	Residential	5	CIP conc.	BF	5	D	1
13	Residential	5	CIP conc.	BF	4	D	1
14	Residential	5	CIP conc.	BF	2	D	1
15	Residential	5	CIP conc.	BF	1	C	1
16	Residential	5	CIP conc.	BF	1	E	1
17	Residential	5	RM+PC	BW	4	D	1
18	Residential	5	RM+PC	BW	2	D	1
19	Residential	5	RM+PC	BW	1	C	1
20	Residential	5	RM+PC	BW	1	E	1
21	Residential	12	RM+Steel	BW	4	D	1
22	Residential	12	RM+Steel	BW	2	D	1
23	Residential	3	Wood	LF	4	D	1
24	Residential	3	Wood	LF	2	D	1
25	School	3	CIP conc.	MRF	4	D	1.25
26	School	3	CIP conc.	MRF	2	D	1.25
27	School	3	CIP conc.	MRF	1	C	1.25
28	School	3	CIP conc.	MRF	1	E	1.25
29	Warehouse	1	RM+Wood	BW	5	D	1
30	Warehouse	1	RM+Wood	BW	4	D	1
31	Warehouse	1	RM+Wood	BW	2	D	1
32	Warehouse	1	Tilt-up	BW	4	D	1
33	Parking	4	PC*	BF	4	D	1
34	Parking	4	PC	BF	2	D	1
35	Parking	4	PC	BF	1	C	1
36	Parking	4	PC	BF	1	E	1
37	Parking	4	PC	BF	1	D	1
38							
39	Hospital	3	Steel	EBF	4	D	1.5
40	Hospital	3	Steel	EBF	2	D	1.5

* The Shearwalls are CIP for Building #33

Summary Counts:

Type of Building	Number	Material	Number	Earthquake Intensity	Number
Office	11	Steel	8	5	5
Residential	13	CIP conc.	14	4	14
School	4	Masonry	9	3	0
Warehouse	4	Wood	5	2	12
Parking	5	Precast concrete	4	1	9
Hospital	2				

Note: The portions marked by (shading) were designed by S.K. Ghosh Associates Inc.
The remaining buildings were designed by J.R. Harris & Company

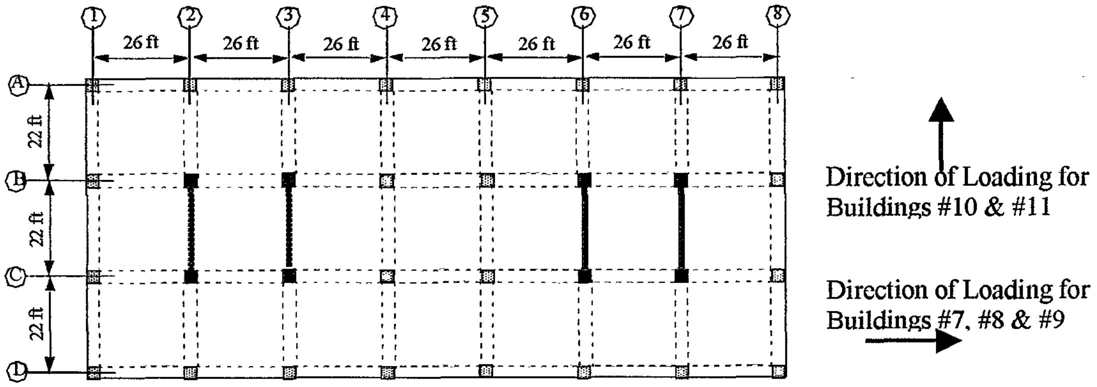


Fig. 2(a). Floor Plan of Office Building Studied – Buildings #7 and #10

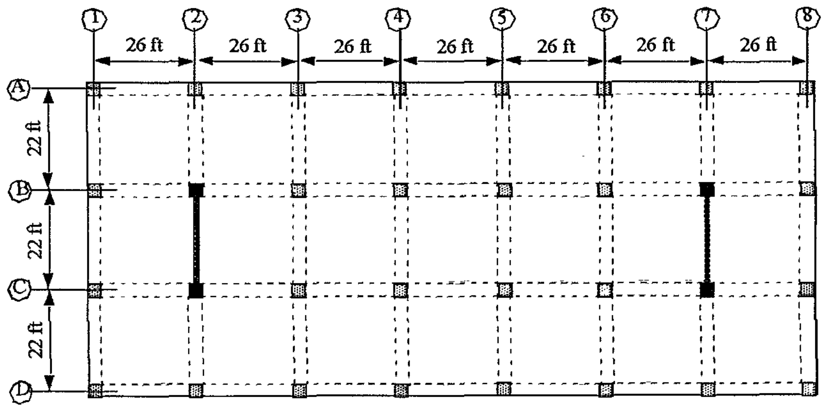


Fig. 2(b). Floor Plan of Office Building Studied – Buildings #8, #9 and #11

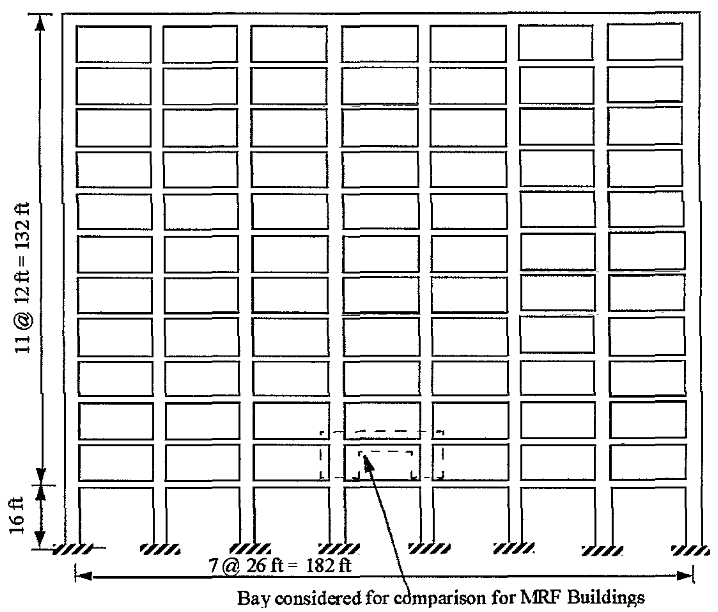


Fig. 2(c). Elevation of Office Building Studied

direction along lines 2 and 7 (Fig. 2b). This investigation started with the same configuration. However, two more shearwalls had to be added along Lines 3 and 6 (Fig. 2a) for Buildings # 7 and 10 which are located in near-source areas of UBC Seismic Zone 4 (loosely designated as Zone 5 in Table 7). The input design data and the output desired are detailed in Table 8.

The design base shears computed for Buildings #7-11 by UBC 1997 and IBC 2000 are listed in Table 9(a) and are graphically shown in Fig. 3(a). It can be seen that in the near-fault areas of Seismic Zone 4 (Zone 5) the design base shear decreases from UBC 1997 to IBC 2000. This indicates that at the particular location of Buildings #7 and #10 (Berkeley, CA), the consideration of near-source effects is overly conservative in the UBC, compared with the IBC. Of course, no generalization can be made from designs done for this one location. In downtown San Francisco (Zone 4), there is hardly any difference in base shear levels between UBC 1997 and IBC 2000.

In New York (UBC Seismic Zone 2), the design base shear for IBC 2000 is significantly lower than the corresponding value calculated by UBC 1997. This is primarily reflective of the fact that the IBC design spectral response acceleration S_{D1} for Type D soil is considerably lower than the corresponding value of C_V of UBC 1997 at that particular location.

The quantities of materials (volumes of concrete and steel) were computed for one typical moment frame bay (see Fig. 2c) in the long direction of the example structure (Buildings #7-9) for the designs by the 1997 UBC and the 2000 IBC. The quantities are given in Table 9(b). Materials quantities were also computed for one shearwall between grade and the first floor level in the short direction of the example structure (Buildings #10, 11) for the designs by the 1997 UBC and the 2000 IBC. The quantities are listed in Table 9(c). The steel quantities for Buildings #7-#10 are plotted in Fig. 3(b), and those for Buildings #10 and #11 are plotted in Fig. 3(c).

A comparison between Fig. 3(b) and Fig. 3(a) shows that for the Berkeley (Zone 5), San Francisco (Zone 4) and New York (Zone 2) designs, the material quantities follow the trends of the design base shears.

A rational shearwall design procedure for regions of high seismicity was adopted into the 1994 UBC and then further refined in the 1997 UBC. ACI 318-99, as adopted into IBC 2000, includes a design procedure for shearwalls in high seismic zones (or assigned to high seismic performance/design categories) that represents a further evolution of the 1997 UBC procedure. In downtown San Francisco (Zone 4), the steel quantity goes up slightly from the UBC 1997 design to the IBC 2000 design (Fig. 3c), because the boundary zone detailing requirement is slightly more conservative in ACI 318-99 (as adopted into the 2000 IBC) than in UBC 1997. In near-fault areas of Seismic Zone 4 (Zone 5), the steel quantity does not go up in spite of the conservatism of the shearwall design procedure of ACI 318-99, because the 2000 IBC design base shear is significantly lower than the 1997 UBC value, as noted earlier.

Five-Story Reinforced Concrete Residential Building

A typical plan and elevation of the structure considered (Buildings #12-#16) are shown in Fig. 4 - for Buildings #12 and #13 in Fig. 4(a) and for Buildings #14, #15 and #16 in Fig. 4(b).

The structure employs a Building Frame System (shearwalls resisting 100% of the design lateral forces and essentially complete moment frames resisting substantially all the gravity loads) in the transverse or short direction, and was designed only in that direction. The building in the source document (Neville 1984) uses one shearwall on each face of the building in the transverse direction, along Lines 1 and 6 (Fig. 4b). This investigation started with basically the same configuration, with barbell walls replacing channel-shaped walls. However two shearwalls had to be used along Lines 1 and 6 in Buildings #12 and #13 (Fig. 4a). Building #12 is in the near-fault areas of UBC Seismic Zone 4 (loosely designated as Zone 5), and is assigned to Seismic Design Category E of IBC 2000. Building #13 is in UBC Seismic Zone 4, and is assigned to Seismic Design Category D of IBC 2000. The input design data and the output desired are detailed in Table 10.

The design base shears computed for Buildings #12-#16 by UBC 1997 and IBC 2000 are listed in Table 11(a) and are graphically shown in Fig. 5(a). For Building #12 (near-fault areas of UBC Seismic Zone 4 and IBC Seismic Design Category E), the design base shear decreases from UBC 1997 to IBC 2000. This decrease is partly due to the R-values of 5.5 and 6, respectively, in these two documents for the Building Frame System consisting of specially detailed shearwalls. However, the substantive difference also indicates that at the particular location of Building #12 (Berkeley, CA), the consideration of near-source effects is overly conservative

Table 8. Input Design Data and Output Desired for Buildings #7-#11

COMPARATIVE SEISMIC DESIGN of OFFICE BUILDINGS

Building # 7-11 (Table 7)

12-story CIP Concrete Building utilizing the Moment Resisting Frame System for Buildings #7, #8, and #9 and utilizing the Dual System for Buildings #10 and #11

Input:

Materials Data: f_c' = 4000 psi

f_y = 60,000 psi (for main bars as well as stirrups)

Load Data:

Dead Load: Flat Plate 8.0 in. thick, equivalent of 97 psf

Typical Floor:

SDL: 30 psf (20 for partition + 10 for ceiling & misc.)

Live Load: 50 psf (per Table 16-A of UBC 1997)

Roof:

SDL: 10 psf + 200 kips for penthouse

Live Load: 20 psf

Buildings #7 and #10 are identical, except that the lateral-force-resisting system is considered to be dual system when lateral load is applied in N-S direction and is considered to be a MRF system when the load is applied in E-W direction. The same is the case with Buildings #8 and #11.

Output Required:

The buildings are to be designed by two codes (UBC 1997 and IBC 2000) and on different site conditions (as shown in Table 7).

The following parameters are to be compared:

1. Base Shear
2. Design of a Typical Shearwall for Buildings # 10 and #11
3. Quantity of Materials (Steel & Concrete) in a Typical Shearwall (B2-C2 between Grade and Level 2) for Buildings #10 and #11
4. Design of a Typical Bay (Beam, Columns and Joints) for Buildings # 7, #8 and #9
5. Quantity of Materials (Steel & Concrete) in a Typical Bay (Bay C4-C5 between Level 2 and Level 3) for Buildings #7, #8 and #9

Table 9(a). Comparison of Base Shear (kips) by Two Codes for Office Buildings (based on equivalent lateral force procedure)

Building	Seismicity/Soil	Structural System	UBC 1997	IBC 2000
7	5D	Special MRF	4216	3033
8	4D	Special MRF	1783	1732
9	2D	Intermediate MRF	1200	553
10	5D	DUAL	6323	4549
11	4D	DUAL	2674	2598
		V/W =	$C_v I/RT$	$S_{D1} I/RT$

Table 9(b). Comparison of Quantities of Concrete and Steel in a Typical Bay of Office Building by Two Codes

CODE	Building #	Volume of Concrete cu.ft.	Volume of Steel cu.ft.
UBC 1997	7	399	10.60
UBC 1997	8	303	7.50
UBC 1997	9	235	6.15
IBC 2000	7	346	9.54
IBC 2000	8	303	7.50
IBC 2000	9	216	5.59

Table 9(c). Comparison of Quantities of Concrete and Steel in a Typical Shearwall of Office Building by Two Codes

CODE	Building #	Volume of Concrete cu.ft.	Volume of Steel cu.ft.
UBC 1997	10	786	25.84
UBC 1997	11	564	17.75
IBC 2000	10	773	19.28
IBC 2000	11	564	18.85

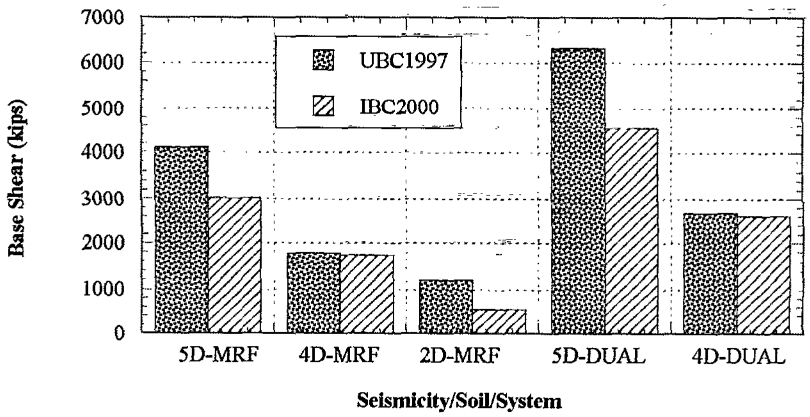


Fig. 3(a). Comparison of Base Shear by Two Codes for Office Buildings

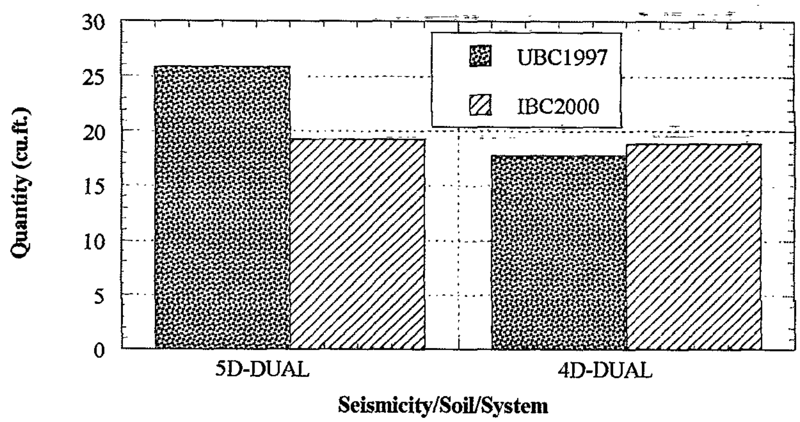


Fig. 3(b). Comparison of Quantity of Steel in One Shearwall of Office Building by Two Codes

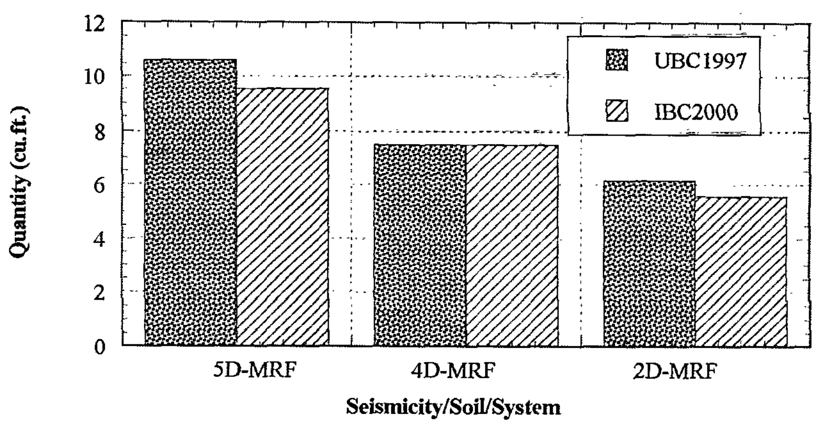


Fig. 3(c). Comparison of Quantity of Steel in a Typical Bay of Office Building by Two Codes

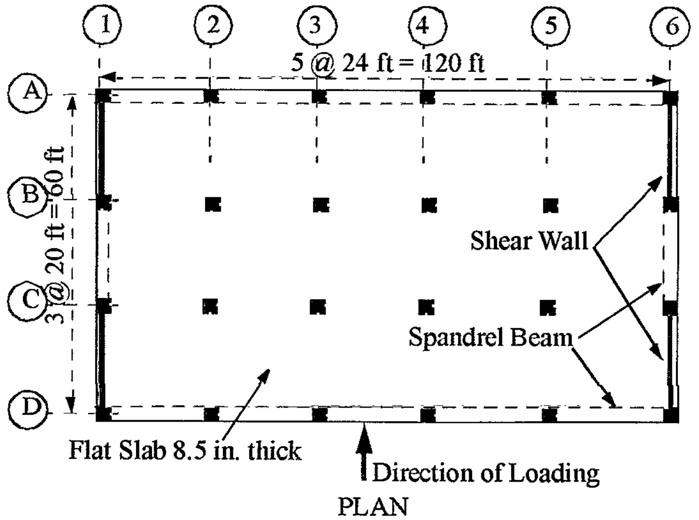


Fig. 4(a). Floor Plan of Residential Building Studied – Buildings #12 and #13

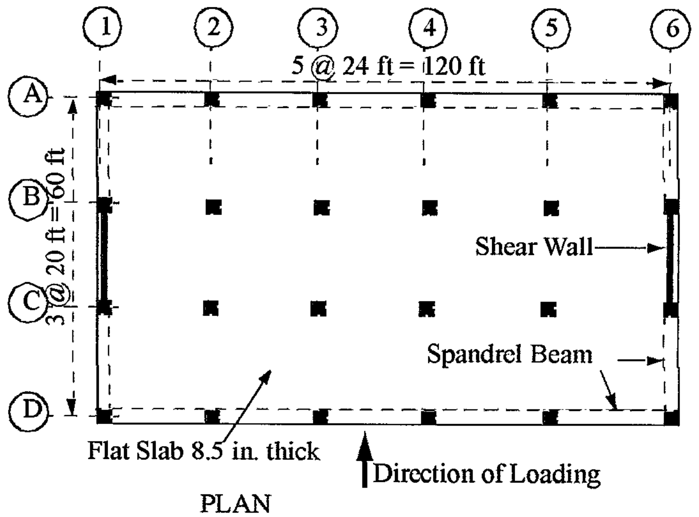


Fig. 4(b). Floor Plan of Residential Building Studied – Buildings #14, #15 and #16

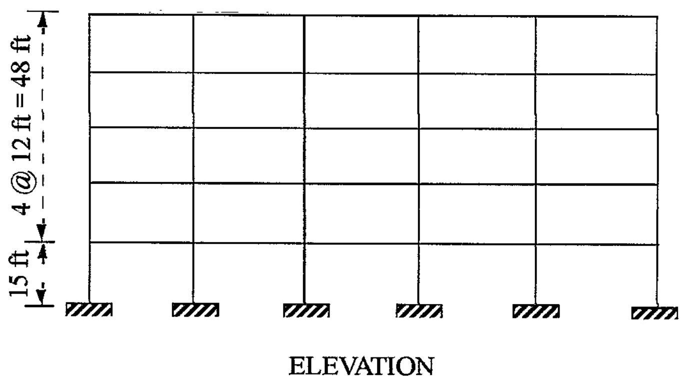


Fig. 4(c). Elevation of Residential Building Studied – All Buildings

Table 10. Input Design Data and Output Desired for Buildings #12-#16

COMPARATIVE SEISMIC DESIGN of RESIDENTIAL BUILDINGS

Buildings # 12-16 (Table 7)

5-story CIP Concrete Buildings utilizing the Building Frame System

INPUT:

Materials Data: $f_c' = 4000$ psi

$f_y = 60{,}000$ psi (for main bars as well as stirrups)

Load Data:

Dead Load:		Flat Plate 8.5 in. thick, equivalent of 106 psf
Typical Floor:		
	SDL:	30 psf (20 for partition + 10 for ceiling & misc.)
	Live Load:	50 psf
Roof:		
	SDL:	10 psf + 200 kips for penthouse
	Live Load:	20 psf

OUTPUT REQUIRED:

The buildings are to be designed by two codes (UBC 1997 and IBC 2000) and on different site conditions (as shown in Table 7).

The following parameters are to be compared:

1. Base Shear
2. Design of a Typical Shearwall
3. Quantity of Materials (Steel and Concrete) in the Shearwall

Table 11(a). Comparison of Base Shear (kips) by Two Codes for Residential Buildings (based on equivalent lateral force procedure)

Building	Seismicity/Soil	System	UBC 1997	IBC 2000
12	5D	Building Frame	1893	1402
13	4D	Building Frame	1207	1006
14	2D	Building Frame	581	374
15	1C	Building Frame	236	276
16	1E	Building Frame	502	544
		V/W =	**$2.5C_aI/R$**	**$S_{DS}I/R$**

Table 11(b). Comparison of Quantities of Concrete and Steel in a Typical Shearwall of Residential Buildings by Two Codes

CODE	Building #	Volume of Concrete cu.ft.	Volume of Steel cu.ft.
UBC 1997	12	345	7.43
UBC 1997	13	276	5.23
UBC 1997	14	276	4.61
UBC 1997	15	229	2.06
UBC 1997	16	276	3.59
IBC 2000	12	345	5.43
IBC 2000	13	276	4.02
IBC 2000	14	276	3.59
IBC 2000	15	229	2.06
IBC 2000	16	276	3.59

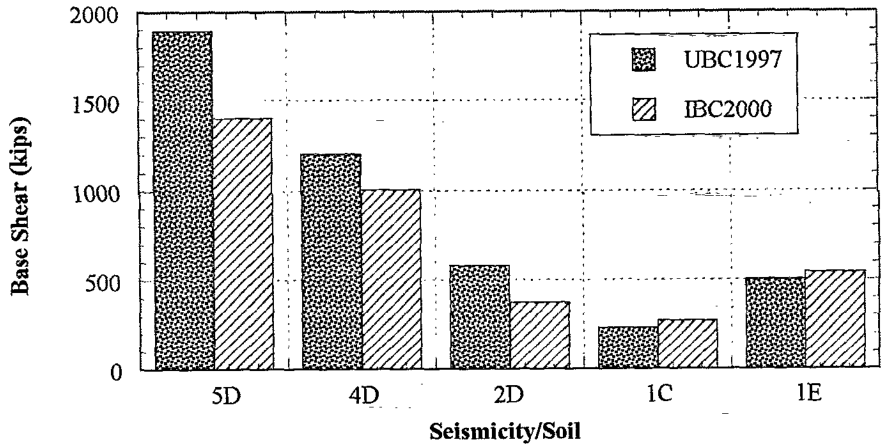


Fig. 5(a). Comparison of Base Shear by Two Codes for Residential Buildings

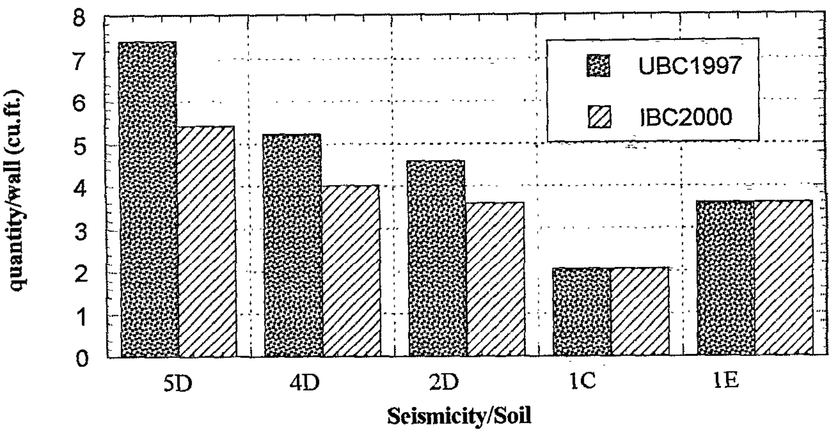


Fig. 5(b). Comparison of Quantities of Steel in One Shearwall by Two Codes for Residential Buildings

compared with the IBC. Of course, one cannot and should not generalize from designs done for this one location.

For Building #13 (UBC Seismic Zone 4 and IBC Seismic Design Category D), the design base shear decreases a little from UBC 1997 to IBC 2000. This decrease reflects the R-values of 5.5 and 6 respectively, in these two documents, for the Building Frame System consisting of specially detailed shearwalls.

For Building #14 (UBC Seismic Zone 2 and IBC Seismic Design Category C), the design base shear decreases from UBC 1997 to IBC 2000. The R-values for the Building Frame System with ordinarily detailed shearwalls are 5.5 and 5 in the two documents, respectively. The design ground motion parameters $2.5C_a$ (UBC 1997) and S_{DS} (IBC 2000) are 0.55 and 0.39, respectively. These, together with the R-values above, explain the design base shear variations by the two documents.

For Building #15 (UBC Seismic Zone 2 and IBC Seismic Design Category B) the design base shear goes up slightly from the 1997 UBC to the 2000 IBC. For the location of Building #15, Atlanta, on Soil Profile Type C, the values of $2.5C_a$ (UBC 1997) and S_{DS} (IBC 2000) are 0.225 and 0.24, respectively. These values, along with the R-values of 5.5 and 5 for Building Frame Systems consisting of ordinarily detailed shearwalls in the 1997 UBC and the 2000 IBC, respectively, explains the design base shears shown in Fig. 5(a).

For Building #16 (UBC Seismic Zone 1 and IBC Seismic Design Category C), the design base shear goes up slightly from the 1997 UBC to the 2000 IBC. The design ground motion parameters $2.5\ C_a$ (UBC 1997) and S_{DS} (IBC 2000) are 0.475 and 0.47, respectively. The slight increase in the design force from the 1997 UBC to the 2000 IBC is strictly due to the R-values of 5.5 vs. 5 for the Building Frame System consisting of ordinarily detailed reinforced concrete shearwalls in the 1997 UBC and the 2000 IBC, respectively.

The quantities of materials (volumes of concrete and steel) for one shearwall spanning in the short direction between grade and first floor have been computed and are listed in Table 11(b). The steel quantities are graphically shown in Fig. 5(b). The steel quantities generally follow the same trends as the design base shears because the shearwall design procedures of the two codes are now comparable (with the IBC procedure being slightly more conservative), as mentioned in the previous section.

Three-Story Reinforced Concrete School Building

A typical plan and elevation of the structure considered (Buildings #25-28) are shown in Fig.6.

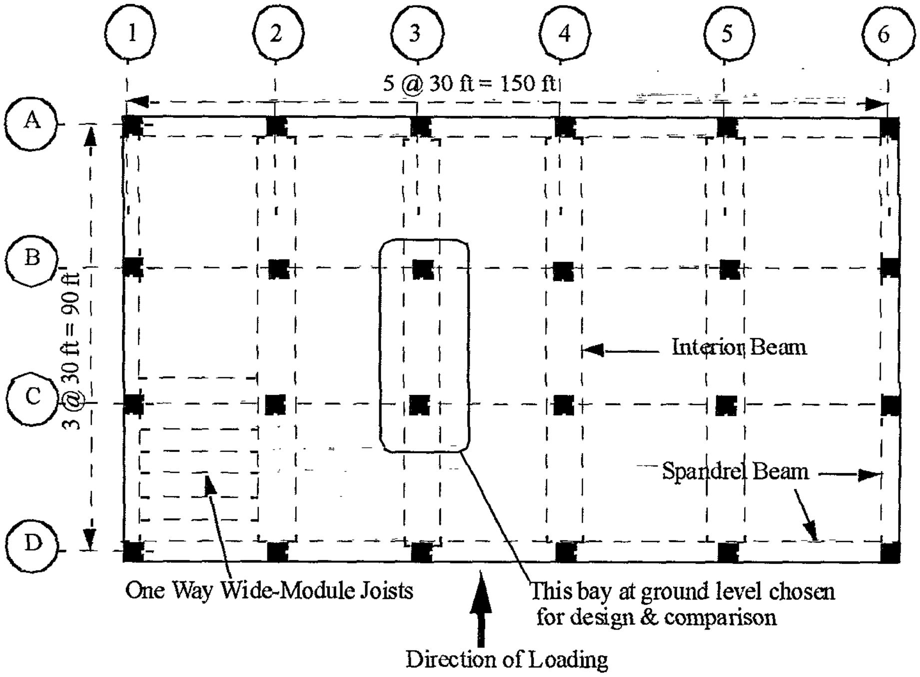
The structure employs a Moment-Resisting Frame System to resist lateral forces along each principal plan axis. It was designed only in the short or transverse direction. The input design data and the output desired are detailed in Table 12.

The design base shears computed for Buildings #25-28 by UBC 1997 and IBC 2000 are listed in Table 13(a) and are graphically shown in Fig. 7(a). For Building #25 (UBC Seismic Zone 4 and IBC Seismic Design Category D), the design base shear decreases slightly from UBC 1997 to IBC 2000. The governing design ground motion parameters of 2.5Ca (UBC 1997) and S_{DS} (IBC 2000) are 1.1 and 1.0, respectively. The R-values are comparable in the two documents at 8.5 and 8 for the 1997 UBC and the 2000 IBC, respectively. The UBC and IBC designs incorporate an importance factor I of 1.25. All of this explains why the design base shears are basically the same for the 1997 UBC and the 2000 IBC.

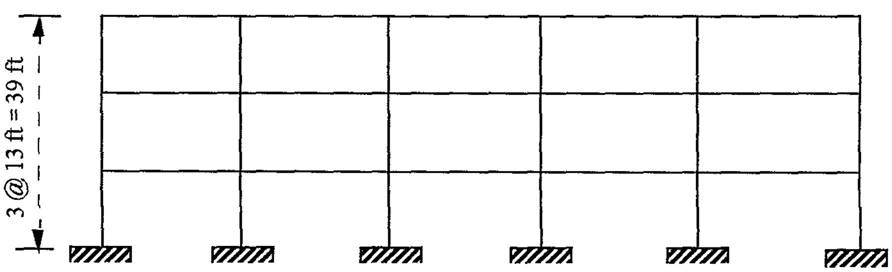
For Building #26 (UBC Seismic Zone 2 and IBC Seismic Design Category C), the design base shear decreases substantially from the 1997 UBC to the 2000 IBC. The reason for the substantive drop is that the governing ground motion parameters of $2.5C_a$ (UBC 1997) and S_{DS} (IBC 2000) drop substantially from 0.55 to 0.39. The R-values are comparable at 5.5 in the 1997 UBC and 5 in the 2000 IBC. The I-values are identical at 1.5.

For Building #27 (UBC Seismic Zone 1 and IBC Seismic Design Category B), the design base shear increases from the 1997 UBC to the 2000 IBC. From UBC 1997 to IBC 2000, the governing design ground motion parameter increases from 2.5Ca = 0.225 to S_{DS} = 0.24, the R-value decreases slightly from 3.5 to 3, and the I-value remains constant at 1.25. The increase in the ground motion parameter and the decrease in th R-value combine to cause the increase in the design base shear.

For Building #28 (UBC seismic Zone 1 and IBC Seismic Design Category C), the design base shear increases from UBC 1997 to IBC 2000. From the 1997 UBC to the 2000 IBC, the governing design ground motion parameter remains essentially unchanged from $2.5\ C_a = 0.475$ to $S_{DS} = 0.47$; however, the R-value decreases from 3.5 to 3,



PLAN



ELEVATION

Fig. 6. Plan and Elevation of School Building Studied – Buildings #25- #28

Table 12. Input Design Data and Output Desired for Buildings #25-#28

COMPARATIVE SEISMIC DESIGN of SCHOOL BUILDINGS

Buildings # 25-28 (Table 7)

3-story CIP Concrete building utilizing the Moment Resisting Frame System

INPUT:

Materials Data: f_c' = 4000 psi

f_y = 60,000 psi (for main bars as well as stirrups)

Load Data:

Dead Load: Use one-way wide-module multiple-span joists (from CRSI handbook)

Joist size = 14 + 4.5x6 + 66 which gives

total depth of 18.5 in. and dead load of 74 psf

Typical Floor:

SDL: 45 psf (20 for partition + 25 for ceiling & misc.)

Live Load: 60 psf

Roof:

SDL: 10 psf + 200 kips for penthouse

Live Load: 20 psf

OUTPUT REQUIRED:

The buildings are to be designed by two codes (UBC 1997 and IBC 2000) and on different site conditions (as shown in Table 7).

The following parameters are to be compared:

1. Base Shear
2. Design of Beam and Connected Columns in a Typical Bay (Bay B3-C3 between ground and Level 2)
3. Quantities of Materials (Steel and Concrete) in the Typical Bay

Table 13(a). Comparison of Base Shear (kips) by Two Codes for School Buildings (based on equivalent lateral force procedure)

Building	Seismicity/Soil	System	UBC 1997	IBC 2000
25	4D	MRF	983	950
26	2D	MRF	698	429
27	1C	MRF	420	523
28	1E	MRF	947	1089
		V/W =	$2.5C_aI/R$	$S_{DS}I/R$

Table 13(b). Comparison of Quantities of Concrete and Steel in a Typical Bay of School Buildings by Two Codes

CODE	Building #	Volume of Concrete cu.ft.	Volume of Steel cu.ft.
UBC 1997	25	360	9.03
UBC 1997	26	213	5.88
UBC 1997	27	167	5.13
UBC 1997	28	227	6.47
IBC 2000	25	360	9.03
IBC 2000	26	198	4.77
IBC 2000	27	175	5.17
IBC 2000	28	360	6.51

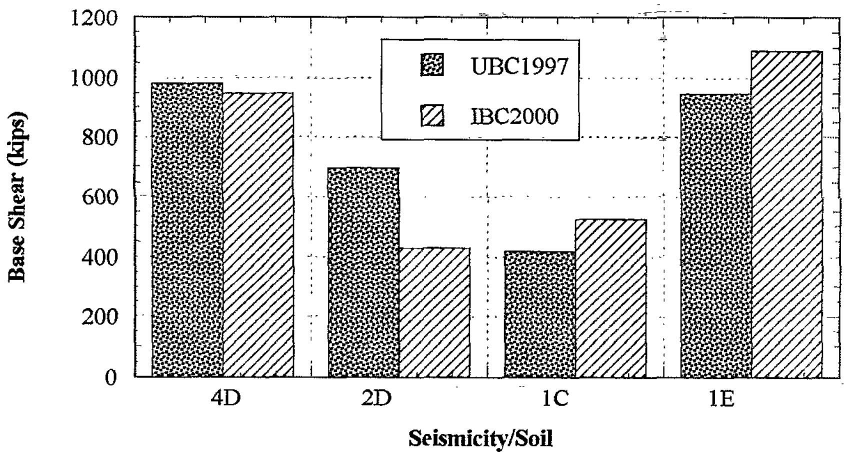


Fig. 7(a). Comparison of Base Shear by Two Codes for School Buildings

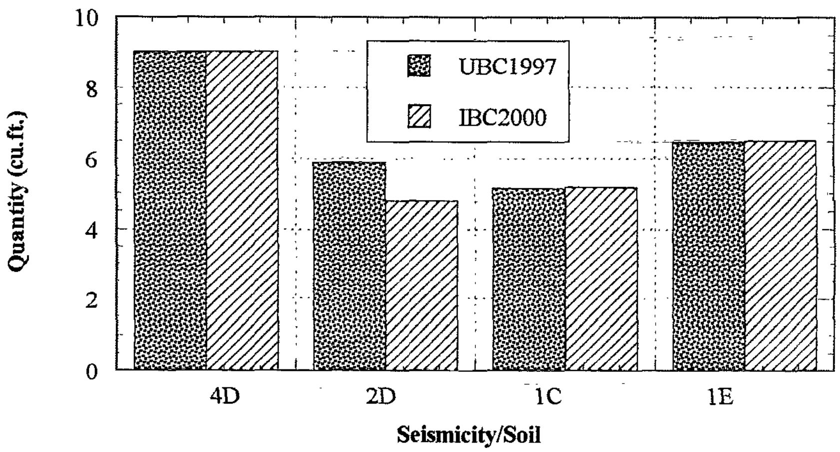


Fig. 7(b). Comparison of Quantities of Steel in One Bay (Bay B3-C3 between Ground and Level 2) by Two Codes for School Buildings

while the I-value remains the same at 1.25. the change in R-value causes the observed increase in the design base shear.

The quantities of materials (volumes of concrete and steel) for one typical bay spanning in the short direction of the building between grade and the first floor level has been computed and are listed in Table 13(b). The steel quantities are graphically shown in Fig. 7(b). The steel quantities generally follow the same trends as the design base shears, as is to be expected.

Four-Story Precast Concrete Parking Structure

The parking structure considered is of precast concrete, except that cast-in-place shearwalls are used in Building #33. The plan and elevation of Building #33 are shown in Figs. 8(a) and (b), respectively. In accordance with current California practice, the building uses 8-ft double-tees and to accommodate it, a 32 ft bay size in the longitudinal direction. Buildings #34-#37 use 10-ft double-tees as are prevalent outside of California, and consequently use 30-ft bay sizes in the longitudinal direction. Twelve walls are used in Buildings #34 and #36, whereas ten walls are used in Buildings #35 and #37. While the example structures are hypothetical, they are also practical in every respect.

The structures employ Building Frame Systems (shearwalls resisting 100% of the design lateral forces and essentially complete moment frames resisting substantially all the gravity loads) along both principal plan axes, and were designed only in the longitudinal direction. The input design data and the output desired are detailed in Table 14.

The design base shears computed for Buildings #33-37 by UBC 1997 and IBC 2000 are listed in Table 15(a) and are graphically shown in Fig. 9(a). For Building #33 (UBC Seismic Zone 4 and IBC Seismic Design Category D), the design base shear decreases from the 1997 UBC to the 2000 IBC. This is largely a reflection of the R-values of 5.5 and 6, used by UBC 1997 and IBC 2000, respectively, for a building frame system consisting of specially detailed reinforced concrete shearwalls. The governing design ground motion parameters are comparable in both cases: $2.5C_a = 1.1$ (UBC 1997) and $S_{DS} = 1.0$ (IBC 2000).

For Building #34 (UBC Seismic Zone 2 and IBC Seismic Design Category C), the design base shear decreases from UBC 1997 to IBC 2000. The reason is that the governing design ground motion parameters in the two cases are: 0.55 (UBC 1997) and 0.39 (IBC 2000). The R-values for a building frame system with ordinarily detailed shearwalls are 5.5 and 5 in UBC 1997 and IBC 2000, respectively.

For Building #35 (UBC Seismic Zone 1 and IBC Seismic Design Category B), the design base shears are not very different by the two documents considered. The design force increases a little from the 1997 UBC to the 2000 IBC. This is because the design ground motion parameters are fairly comparable in the two cases: $2.5C_a = 0.225$ (UBC 1997) and $S_{DS} = 0.24$ (IBC 2000). The R-values, once again, are 5.5 in UBC 1997 and 5 in IBC 2000.

For Building #36 (UBC Seismic Zone 1 and IBC Seismic Design Category C), the design base shears from UBC 1997 and IBC 2000 are comparable, although it is slightly larger for IBC 2000. This is because the design ground motion parameters are very comparable in the two cases: $2.5C_a = 0.475$ (UBC 1997) and $S_{DS} = 0.47$ (IBC 2000). The R-values are 5.5 and 5 in designs per UBC 1997 and IBC 2000, respectively.

The quantities of materials (volume of concrete and steel) for one shearwall spanning in the long direction between grade and the first floor (the 20-ft long segment spanning in the long direction of cruciform wall #5 of Figs. 8a, c and d) have been computed and are listed in Table 15(b). The steel quantities are graphically shown in Fig. 9(b). It should be noted that Building #33 uses cast-in-place shearwalls, while the other buildings use precast shearwalls. The shearwall-to-footing connection is obviously quite different for precast versus cast-in-place walls. Thus, while Fig. 9(b) depicts accurate code-to-code comparisons, a comparison among buildings is probably not valid, if Building #33 is to be included in the mix. Also, all precast walls were designed using mild reinforcement only. Precast walls are often prestressed to facilitate handling. To be considered as a "prestressed" wall under Chapter 18 of ACI 318, the minimum level of prestress, after losses, must be 225 psi. For 13% losses, ten ½ in. diameter low-relaxation strands, each pulled at 31, 000 lbs, would be required per 10-ft long 10-in. thick wall segment. If this level of prestress is not used, then the minimum steel requirements of Chapter 16 apply.

The steel quantities of Fig. 9(b) follow the pattern of the base shears presented in Fig. 9(a). There really are no significant deviations to remark on.

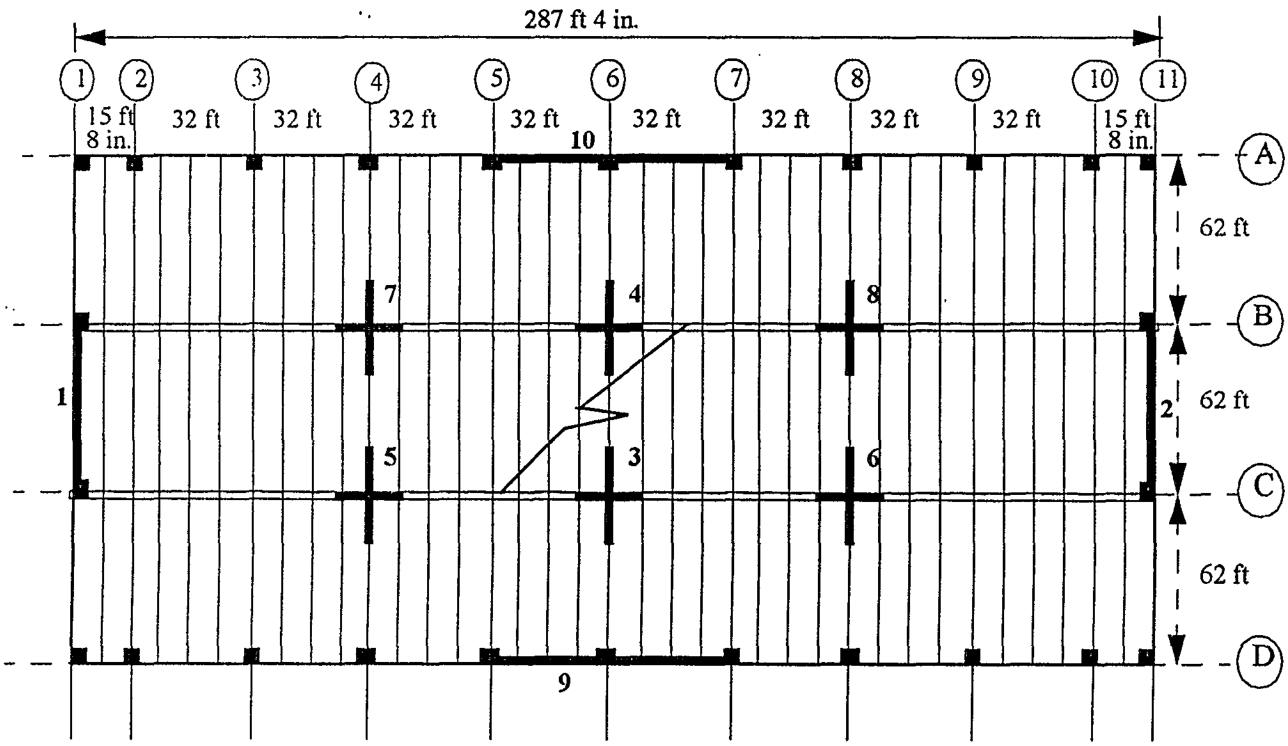


Fig. 8(a). Plan of Building #33

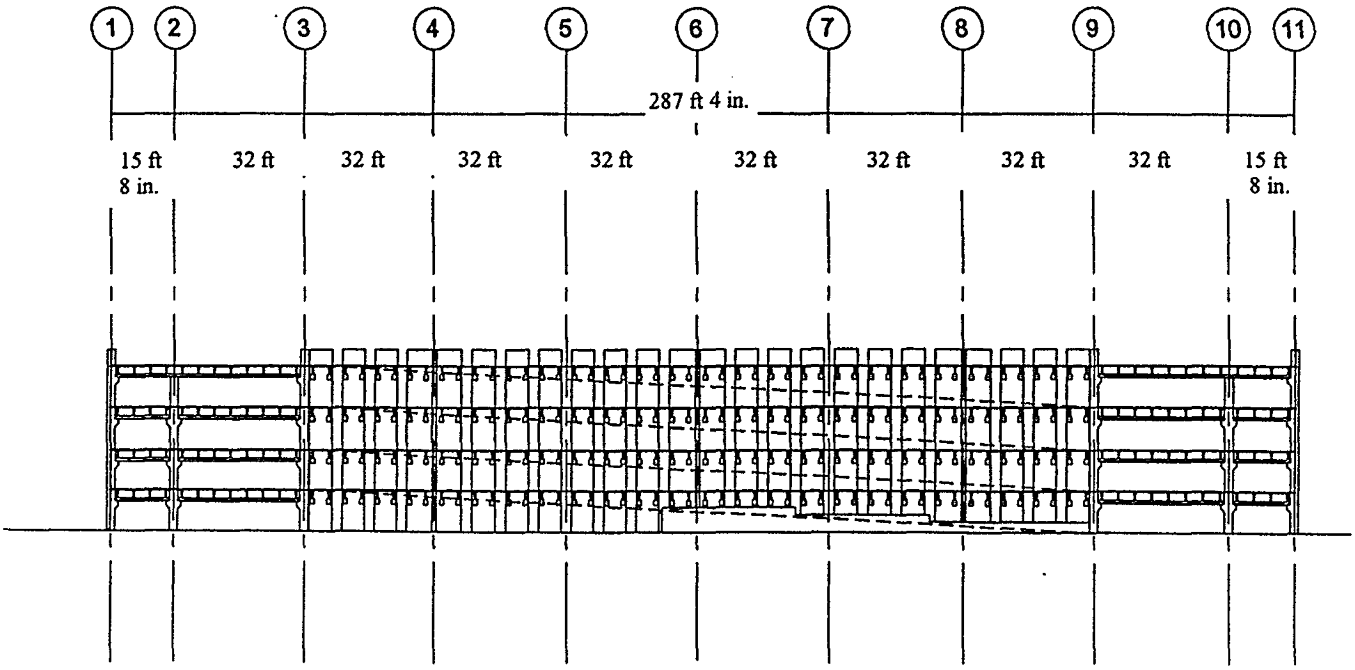


Fig. 8(b). Elevation of Building #33

Table 14. Input Design Data and Output Desired for Buildings #33-#37

COMPARATIVE SEISMIC DESIGN of PARKING STRUCTURES

Buildings # 33-37 (Table 7)
4-story Concrete Parking Structure utilizing the Building Frame System
(CIP Shearwalls in Zone 4, Site Class D and Precast in other Zones)

Input:

Materials Data: f_c' = 5000 psi
f_y = 60,000 psi (for main bars as well as stirrups)

Load Data:

Dead Load:	80 psf (90 psf for Building #33 in Zone 4, Site Class D)
Live Load:	50 psf

Output Required:

The buildings are to be designed by two codes (UBC 1997 and IBC 2000) and on different site conditions (as shown in Table 7).

The following parameters are to be compared:

1. Base Shear
2. Design of a Typical Shearwall (Wall #5 in X-Direction)
3. Quantities of Materials (Steel and Concrete) in the Shearwall

Table 15(a). Comparison of Base Shear (kips) by Two Codes for Parking Structures (based on equivalent lateral force procedure)

Building	Seismicity/Soil	Structural System	UBC 1997	IBC 2000
33	4D	Building Frame	4778	3982
34	2D	Building Frame	2297	1813
35	1C	Building Frame	940	1102
36	1E	Building Frame	1984	2150
37	1D	Building Frame	1253	1433
		V/W =	**$2.5C_aI/R$**	**$S_{DS}I/R$**

Table 15(b). Comparison of Quantities of Concrete and Steel in a Typical Shearwall of Parking Structures by Two Codes

CODE	Building #	Volume of Concrete cu.ft.	Volume of Steel cu.ft.
UBC 1997	33	167	1.45
UBC 1997	34	167	2.07
UBC 1997	35	167	0.83
UBC 1997	36	167	1.72
UBC 1997	37	167	1.72
IBC 2000	33	167	1.23
IBC 2000	34	167	1.33
IBC 2000	35	167	1.11
IBC 2000	36	167	1.72
IBC 2000	37	167	1.72

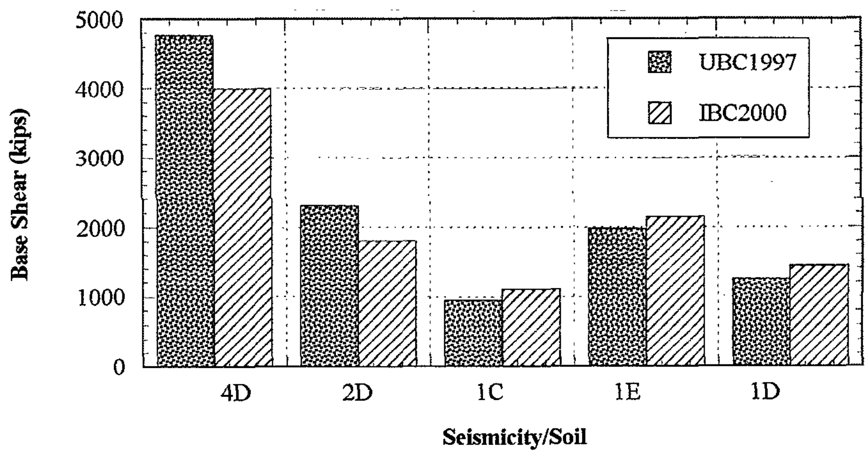


Fig. 9(a). Comparison of Base Shear by Two Codes for Parking Structures

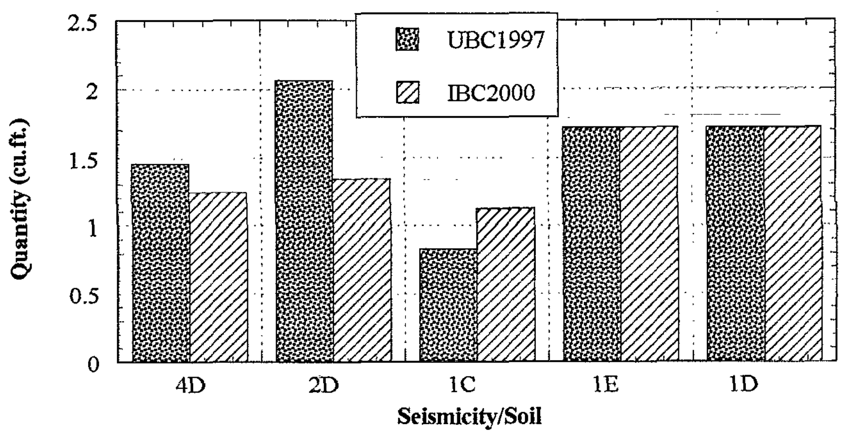


Fig. 9(b). Comparison of Quantities of Steel in One Shearwall by Two Codes for Parking Structures

Conclusions

In the 2000 IBC, permissible structural systems including the level of detailing required for structural members and joints, limitations on height of structure and structural irregularity, the type of lateral load analysis that must be performed as the basis of design, as well as nonstructural component requirements are all dependent upon the soil characteristics at the site of a structure. In the 1997 UBC, the above are determined solely by the seismic zone in which a structure is located. Generalized comparisons show that in five out of six Zones 3, 4 locations examined, site-specific geotechnical investigation will be required by the 2000 IBC on Site Class E, which is currently not required by the 1997 UBC. In Sacramento, on firmer soils, only the equivalent of Zone 2 detailing will be required under the 2000 IBC, because the seismicity there has been judged to be lower by the IBC. In two of the three Zones 0, 1 locations examined, the equivalent of Zone 2 detailing will be required on Site Class E.

The consideration of near-source effects appeared to be overly conservative in the 1997 UBC, based on the comparative designs of two different buildings (three different design cases). However, no generalization can be made from this because both buildings were at one single location. Generally speaking, all observed differences between comparative designs by the 2000 IBC and the 1997 UBC could be explained in terms of differences in governing ground motion parameters (S_{DS} of the I_{BC} vs. $2.5C_a$ of the UBC) and R-values between the two codes.

Acknowledgements

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