A CALIBRATION OF THE LATERAL FORCE REQUIREMENTS OF THE UBC

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SUMMARY

This paper quantifies some of the more significant underlying assumptions and experiences introduced into the seismic provisions of the Uniform Building Code over many decades. The effective peak ground acceleration associated with the Zone coefficients, the ductility ratios associated with the K factors, the direct interpretation of the S factor and the implications of the importance factors, in terms of non-exceedence probabilities, have all been studied and reported.

INTRODUCTION

The seismic provisions of the UBC (Uniform Building Code) are fashioned after the Recommended Lateral Force Requirements of the Structural Engineers Association of California. Embodied in these recommendations are the collective experience of California engineers since the great San Francisco earthquake of 1906, even though it was not until the 1925 Santa Barbara earthquake that legislative actions were taken towards the development of seismic building codes. Since then, many West Coast earthquakes, in addition to other earthquakes worldwide, have come to impact on the provisions of the Code. Over the years, the behavior of structures during earthquakes has been studied in an attempt to identify both weaknesses and successful performance and to improve the Code provisions accordingly. Thus, the present Code (1982) has been arrived at by a process of successive changes over many decades. Before the Code is modified or completely replaced by more recent developments, it would be extremely useful to calibrate its lateral force requirements. The present paper is such an attempt. Due to space limitations, many of the details of the study are not included.

UNIFORM BUILDING CODE SEISMIC PROVISIONS

The UBC requirement for total base shear is given by

$$V = ZIKCSW \tag{1}$$

where Z = Zone Coefficient, with the following values

Zone	4	3	2	1	0
Coefficient	1	3/4	3/8	3/16	-

I = Importance Factor, used for different types of occupancy

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Type of Occupancy	Factor
Essential Facilities	1.50
Assembly Facilities	1.25
(More than 300 people in one room)	
All others	1.00

K = Building Coefficient, a measure of ductility. See Table 1.

C = Seismic Coefficient, not to exceed 0.12

$$C = \frac{1}{15\sqrt{T}} \quad \text{where } T = \text{building period} \tag{2}$$

S = Site-Structure resonance factor. $1.0 \leq S \leq 1.5$

$$S = 1 + \frac{T}{T_s} - 0.5\left(\frac{T}{T_s}\right)^2 \quad \text{for } \frac{T}{T_s} \leq 1 \tag{3a}$$

$$S = 1.2 + 0.6\left(\frac{T}{T_s}\right) - 0.3\left(\frac{T}{T_s}\right)^2 \quad \text{for } \frac{T}{T_s} < 1 \tag{3b}$$

where T_s = site period. The product CS shall not exceed 0.14.

W = Total dead load.

CALIBRATION SCHEME FOR RESPONSE FACTOR, KC

The product KC is simply an alternate definition of an inelastic response spectrum anchored to some effective peak ground acceleration in the most seismic regions of California (Z = 1.0). To calibrate the UBC code requires the determination of a simultaneous equivalency of responses based on the Code and an appropriate inelastic spectrum. The range of structural periods used in this study is from 0.3 sec (3.3 Hz) to 5.0 sec (0.2 Hz). The inelastic spectra derived by Riddle and Newmark (1979) are used. Given the empirical evolution of Eq. 1, the details of the underlying characteristics of the selected target inelastic spectrum are not that important. Because of the relative difficulty of establishing a simultaneous equivalency for many types of structures and for a large range of frequencies, a graphical solution is adopted. The outcome of the solution will be the implicit ductilities associated with the K factors and the effective peak ground acceleration for Zone 4 (and hence for all other zones).

Riddle-Newmark Inelastic Spectra

The basis of the above comparison is the 5% damped mean response curves shown in Fig. 3. This damping value is considered appropriate for the response of structures at relatively high levels of "elastic" response. The elastic mean amplification values for the three segments of the response spectrum curve are:

Displacement — 1.47 x 36 = 52.9 in.
Velocity — 1.55 x 48 = 74.4 in/sec.
Acceleration — 2.28 x 1 = 2.28g

The deamplification factors, for elastoplastic systems, as a function of the ductility factor, are given by the following formulas:

$$\text{Displacement} - R = 0.951\mu^{-1.07} \tag{4a}$$
$$\text{Velocity} - R = (2.418\mu-1.418)^{-0.619} \tag{4b}$$
$$\text{Acceleration} - R = (2.851\mu-1.851)^{-0.422} \tag{4c}$$

K Factors

Although the K factors describe the relative ductility relationship among several types of structures, an absolute fix of the ductility of each one is missing. Thus, many possibilities exist wherein a ductility factor of unity (i.e. max. elastic response) can be assumed for a given base K value and the other relative ductilities then determined. Four such possibilities are listed in Table 2, where the deamplification ratio R is obtained from K/K_{base}. As implied above, K_{base} is not yet determined and its determination is one of the objectives of the present solution. Using Eq. 4 and the deamplification factors of Table 2, the associated ductility factors, μ, can be calculated for each of the spectral segments. It must be recognized that, given the same deamplification factor, the associated ductilities differ for each of the three segments of the spectrum. For purposes of illustration, Table 2 lists the associated ductilities for the velocity segment of the inelastic spectrum.

Graphical Solution And Discussion of Results

Since the inelastic response spectra are anchored to 1.0g effective peak ground acceleration, the ratio of the response according to the UBC to that of the Inelastic Response Spectrum, would determine the effective peak ground acceleration for Zone 4. A solution is obtained only if all ratios, for all of the periods considered and all K values, are equal. This is best visualized if the data is displayed graphically as in Fig. 1. A quick evaluation of these curves should lead one to the conclusion that, if any solution is to be selected, then Trial 4 is the one. The associated ductilities are listed in Table 3. No unique ductility can be ascribed to each K factor, since the deamplification factors in each of the response segments are different for any given ductility factor. As a first approximation, the ductilities listed under the Velocity column could be considered representative. The fact that a perfect solution is not obtained should not come as a surprise considering that several assumptions exist throughout both the UBC and the Inelastic Response Spectra procedures. One of the more important observations though is that the "solution" gives increasing response ratio with increasing structural period. More on this issue below. Adopting Trial 4 as the solution to the problem in hand, it would be useful, for discussion purposes, to replace it by a linear function. This is shown in Fig. 2 and the associated equation is given by

$$a = 0.05T + 0.085 \tag{5}$$

The underestimation of response by Eq. 5, in the range of 0.3 to

0.5 sec., would be approximately compensated for when the Code limitation, that the product of CS $\leq$ 0.14, is applied. If the Code requirements were derived directly from first principles, Eq. 5 should have been a constant (zero slope). The fact that it is an increasing function of the period, T, implies that not only the higher mode effects are taken into account, but that also high-rise structures have been implicitly provided with an increasing safety factor with height. Anchoring the acceleration of 0.1g at 0.3 sec. period (the shortest period considered in this study) and as-assuming that $T \approx 0.1N$, where N is the number of stories, Eq. 5 reduces to

$$a = 0.1\ (0.05N + 0.85) \qquad N \geq 3 \qquad (6)$$

The term in parenthesis in Eq. 6 then accounts for the implicit importance factor together with the higher mode effects. Preliminary calculations suggest that the higher mode effects could constitute from about 1/3 to all of this increase depending on the deformation shape of the structure.

The effective peak ground acceleration of 0.1g for Zone 4 obviously is a low value for California. There are certain considerations, such as special provisions for braced frames, working stress design methods (or load factors greater than unity) and the ratio of seismic to total stress, that would tend to raise this value to about 0.25g. These considerations, although an integral part of any calibration, are beyond the scope of this paper.

Additional Implicit Margins

An important source of margin comes about by ignoring some or all of the potential ductility of structures. For example, in the design of nuclear power facilities, no advantage is taken of the inherent ductility of structures. Assuming that a concrete shear wall structure in a nuclear facility can potentially mobilize a ductility of 3, an increase of the design spectrum curve by the following factors is implicitly introduced.

$$D: \frac{1}{.294} = 3.40, \quad V: \frac{1}{.336} = 2.98 \quad \text{and} \quad A: \frac{1}{.448} = 2.23$$

The denominators in the above fractions are the deamplification factors for a ductility factor of 3. This concept is generalized in Fig. 4. The figure is split for clarity. The abscissa represents the Potential Ductility that a structure can mobilize. The numbers in circles next to the curves are the ductility factors specified for design. Fig. 4, used in conjunction with Table 3, can provide information regarding additional margins. For example, assuming that a structure with K = 0.67 can potentially have a ductility of 6 instead of the 4 implicitly specified in the Code, then the corresponding increase in design forces would be 1.54, 1.33 and 1.22 for the Displacement (D), Velocity (V), and Acceleration (A) segments of the frequency spectrum.

SITE-STRUCTURE RESONANCE FACTOR

Eq. 3 gives the site-structure resonance factor S, wherein a site period concept is advanced. The concept of a site period is difficult to visualize except maybe for some confined alluvial deposits. A large expanse of overburden cannot get into a "resonance" mode much like a building or a dam does. A more generally acceptable concern is the phenomenon of soil-structure interaction. In addition to the building dynamic characteristics, foundation size and site shear wave velocity profile contribute significantly to this problem. To study the impact of soil-structure interaction on base shear and overturning moment, a three-degree-of-freedom model of a structure, with a 50' x 50' base dimensions, was selected. To represent a large spectrum of structures, the following fundamental frequencies were assigned to the model: 1/16, 1/4, 1, 4, 16, and 64 Hz. The shear wave velocity of the foundation material was also varied to cover a wide spectrum of potential sites: V_s = 750, 1500, 2000, 3000 and 10,000 ft/sec. Thus a total of 30 structure-soil systems were analyzed for shear and bending moment throughout the structure. The ratio of the system response to its fixed base counterpart value was obtained and plotted against the dimensionless period, $T_o = V_s/\omega R$, where V_s is the equivalent shear wave velocity of the site; ω, the fixed base structure fundamental frequency, in radians/sec.; and $R = \sqrt{BL/\pi}$, the equivalent radius of the foundation, with BL being the area of the foundation. Because of the definition of the dimensionless period, there is no need to also vary the size of the foundation. The base shear and base overturning moment ratios are plotted in Fig. 5. As expected the ratios are both above and below unity, indicating that soil-structure interaction can both amplify and reduce response of structures. In a Code environment it would be difficult to exploit the above reduction, even though it mainly occurs for the soft site (V_s = 750 ft/sec). The data on the amplification side of the abscissa can be enveloped by the following empirical relationship

$$S = 1 + \frac{5}{10 + T_o^{1.3}} \tag{7}$$

Eq. 7 is simpler to use then Eq. 3, and it addresses the issue of soil-structure interaction directly. Furthermore, the shear wave profile of a site can be obtained more reliably than the site "period." It is interesting to note though that the maximum value of S from both Eqs. 3 and 7 is 1.5.

IMPORTANCE FACTOR

As described earlier, the Importance Factor, I, takes on three distinct values of 1, 1.25, and 1.5. To calibrate these factors the statistical data from Riddle and Newmark (1979) will be used. For the elastoplastic system, the 5% damped curve has the following coefficients of variation (COV): 0.43, 0.39, and 0.22 for the D, V, and A segments of the response spectrum respectively. These values change slightly as a function of the ductility factor. For ductility values being considered herein, more representative values of COV would be 0.45, 0.35, and 0.17 for the D, V, and A segments of the curve respectively. Based on these values the following levels of non-exceedence probabilities may be estimated assuming a normal distribution.

Factor	Std. Deviation D	V	A	Non-Exceedence Probability D	V	A
1.25	0.56σ	0.71σ	1.47σ	71%	76%	93%
1.50	1.11σ	1.43σ	2.94σ	87%	92%	99.8%

The following observations may be made from the above data. The Importance Factor as presently recommended by the Code provides different safety margins for the same type of occupancy depending on the structural frequencies. For structures in the 3-8 Hz range, a 1.5 factor provides an inordinately high relative safety margin.

CONCLUSIONS

The basic objective of this study was to explore the implicit assumptions and decisions embodied in the seismic provisions of the UBC and in the process to salvage the experience gained through many decades of successive improvements. The quantification of this experience should prove useful in future changes of the Code.

REFERENCES

International Conference of Building Officials, Uniform Building Code, 1982, Whittier, California

Riddle, R. and Newmark, N. M., Statistical Analysis of the Responses of Nonlinear Systems Subjected to Earthquakes, UILU 79-2016, Dept. of Civil Engineering, University of Illinois, August 1979

Table 1. Building Coefficient, K

Structural System	K
Building with box system: No complete vertical load-carrying space frame; lateral forces resisted by shear walls.	1.33
Building with dual bracing system consisting of ductile moment-resisting space frame and shear walls, designed so that: (1) Frames and shear walls resist total lateral force in accordance with their relative rigidities, considering the interaction of shear walls and frames. (2) Shear walls acting independently of space frame resist total required lateral force. (3) Ductile moment-resisting space frame has capacity to resist at least 25% of required lateral force.	0.80
Building with ductile moment-resisting space frame designed to resist total required lateral force.	0.67
Other building framing systems.	1.00
Elevated tanks, plus full contents, on four or more cross-braced legs and not supported by a building.	2.50
Structures other than buildings.	2.00

Table 2. Possible Sets of Ductilities Associated with K Values (Ductilities are computed from Eq. 4b as an example)

	Trial 1		Trial 2		Trial 3		Trial 4	
K	R	μ	R	μ	R	μ	R	μ
0.67	0.134	11.3	0.167	8.1	0.223	5.3	0.267	4.1
0.80	0.160	8.6	0.200	6.2	0.267	4.1	0.320	3.2
1.00	0.200	6.2	0.250	4.5	0.333	3.0	0.400	2.4
1.33	0.266	4.1	0.333	3.0	0.444	2.1	0.533	1.7
2.00	0.400	2.4	0.500	1.9	0.667	1.4	0.800	1.2
2.50	0.500	1.9	0.625	1.5	0.833	1.1	1.000	1.0
3.0	0.600	1.5	0.750	1.2	1.000	1.0	-	-
4.0	0.800	1.2	1.000	1.0	-	-	-	-
5.0	1.000	1.0	-	-	-	-	-	-

Table 3. Ductility Factors Associated with Coefficient K

K	Displacement	Lower Freq. (Hz)	Velocity	Upper Freq. (Hz)	Acceleration
0.67	3.3	(0.28)	4.1	(2.7)	8.7
0.80	2.8	(0.26)	3.2	(2.6)	5.9
1.00	2.2	(0.24)	2.4	(2.4)	3.7
1.33	1.7	(0.23)	1.7	(2.2)	2.2
2.00	1.2	(0.23)	1.2	(2.0)	1.2
2.50	1.0	(0.23)	1.0	(1.9)	1.0

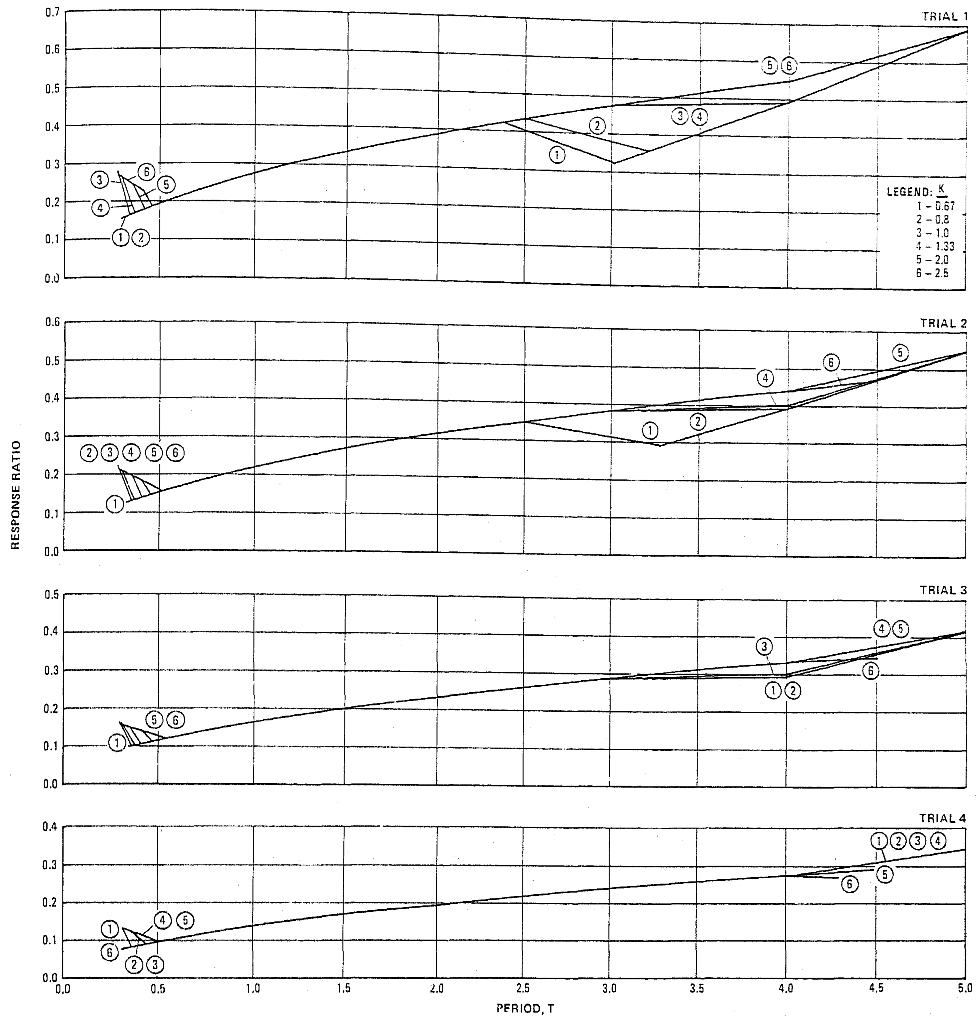

FIG. 1. PLOTS OF THE RATIO OF THE UBC RESPONSE TO THAT FROM THE INELASTIC RESPONSE SPECTRA

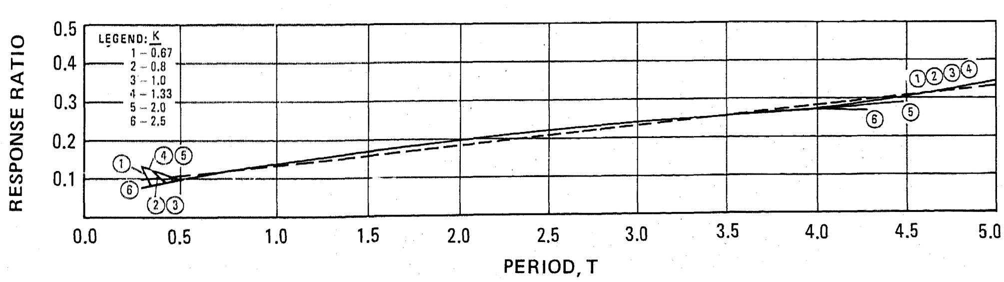

FIG. 2. LINEARIZATION OF THE RESPONSE RATIO FOR TRIAL 4

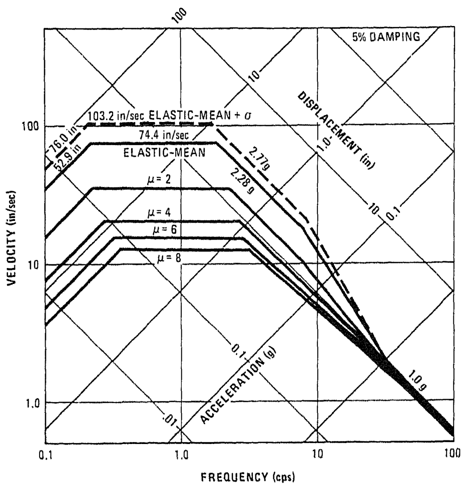

FIG. 3. MEAN DESIGN SPECTRA SCALED TO 1g GROUND ACCELERATION (RIDDLE AND NEWMARK, 1979)

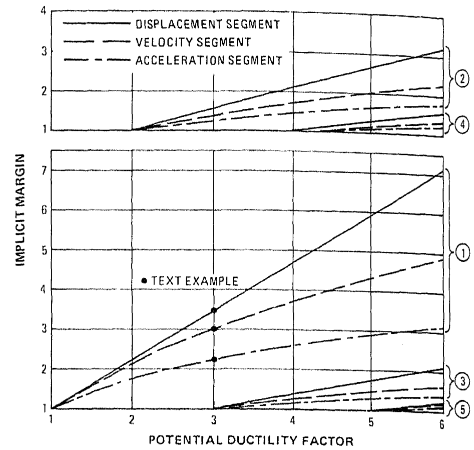

FIG. 4. IMPLICIT MARGINS FOR ELASTO-PLASTIC SYSTEMS WITH 5% DAMPING AS A FUNCTION OF POTENTIAL DUCTILE CAPABILITIES

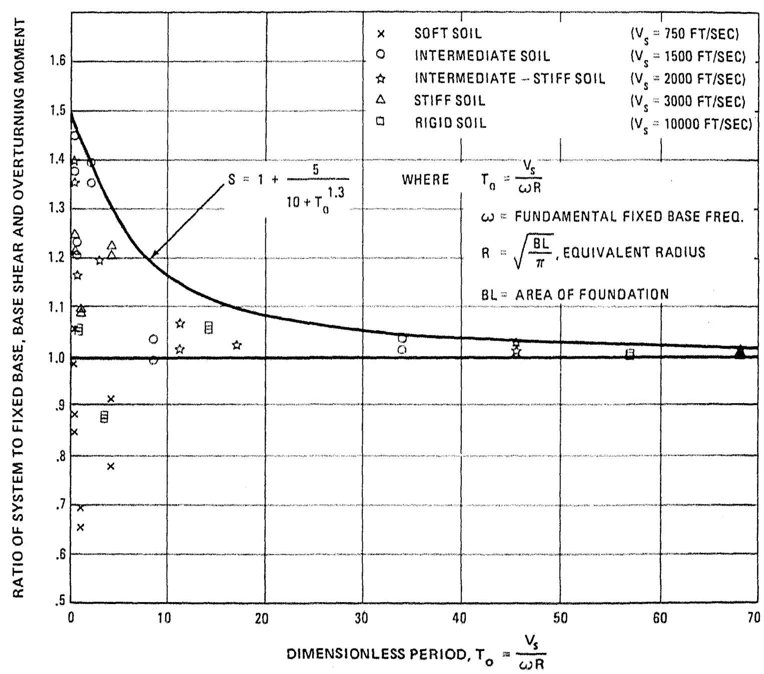

FIG. 5. IMPACT OF SOIL-STRUCTURE INTERACTION ON BASE SHEAR AND OVERTURNING MOMENT