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Force Reduction Factors for the Seismic Provisions of the National Building Code of Canada

by J.H. Rainer

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Force reduction factors for the seismic provisions of the National Building Code of Canada

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A derivation of force reduction factors for the seismic provisions of the National Building Code of Canada (NBCC), 1985, is presented. This includes the following: classification of seismic actions, applicable limit states, change in load factor, derivation of force reduction factors, and classification of structural configurations. Quantitative comparisons are made between the derived force reduction factors and the response modification factors of the Applied Technology Council and good agreement was found. It is suggested that seismic requirements should be considered as accidental actions with a load factor $\alpha_Q = 1.0$. These results can form the basis for possible modifications to the 1985 NBCC seismic provisions.

Key words: earthquake resistant structures, building code, loads, load factors.

Une dérivation des facteurs de réduction de la force pour les exigences du Code national du bâtiment (1985) relatives aux séismes est présentée. Ceci inclut : classification de l'activité sismique, états limites applicables, modification du coefficient de charge, dérivation des facteurs de réduction de la force et classification des configurations des structures. Des comparaisons quantitatives sont effectuées entre les facteurs de réduction de la force dérivés et les facteurs de modification de la réponse proposés par le Applied Technology Council, et on a constaté une bonne correspondance entre les deux. L'auteur suggère de considérer les exigences relatives aux séismes comme des événements accidentels avec un coefficient de $\alpha_Q = 1.0$. Ces résultats peuvent servir de base à des modifications possibles des exigences du Code national du bâtiment (1985) relatives aux séismes.

Mots clés : structures résistant aux tremblements de terre, code du bâtiment, charges, coefficient de charge.

[Traduit par la revue]

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1. Introduction

The objective of the seismic provisions of the National Building Code of Canada (NBCC 1985a) is to protect the occupants of buildings from the destructive effects of earthquakes. This is achieved by specifying design loads and detailing requirements so that the probability of building collapsing or injuring people is acceptably low when the structure is subjected to a certain level of ground motion. At the same time it is admitted that the building or portions of it may sustain minor or major damage (NBCC 1985b). Code requirements for achieving such seismic resistant characteristics of buildings have evolved in a semiempirical manner by simplified mathematical and experimental models and from experience of the behaviour of structures in earthquakes. A history of the development of the NBCC seismic provisions has been presented by Uzumeri *et al.* (1978). Because of the semiempirical development of the code, the basis for some of the loading requirements are sometimes obscured to the user. This tends to impede the rational application of the provisions to specific problems, particularly nonstandard ones, and slows the progress in improving code provisions in light of new research results.

Significant steps toward a rationalization of the NBCC seismic provisions were taken in the 1970 edition when a seismic risk map based on probabilistic principles was introduced. Also an explicit ground acceleration parameter A was incorporated into the base shear calculation. Further rationalization occurred in 1985 when two ground motion parameters, acceleration and velocity, were adopted, resulting in a seismic response factor S that varies with the applicable velocity and acceleration zones Z_v and Z_a (Heidebrecht *et al.* 1983; Heidebrecht and Tso 1985). In the following presentation,

further possible improvements in the seismic design requirements of the NBCC are suggested. These topics are currently under consideration by the Canadian Committee on Earthquake Engineering (CANCEE). Before commencing with detailed considerations of seismic resistant design requirements, however, it is desirable to examine how the seismic phenomenon is treated by the building code.

The natural phenomenon that is to be addressed in seismic design is the ground motion at a given location as a result of a seismic event at some distance. This ground motion is given in terms of displacement, velocity, or acceleration and is termed the "action" (CSA 1981; ISO 1984). For code purposes, the action corresponds to the design ground acceleration or velocity, or related coefficients, which are obtained from seismic risk maps that are derived from a statistical evaluation of historical and geological data corresponding to a small probability of exceedance per annum. This seismic action is independent of any structure to be considered.

The building to be built at a particular location has to be designed to resist this action. Since the seismic events implied by the specified seismic action occur infrequently, nonlinear material behaviour is accepted, nay encouraged, thereby reducing member sizes. A trade-off is thus made between ductile structural behaviour and accompanying damage and the initial cost of the structure.

The reader will no doubt ask "Why are the seismic design forces not actions?" These forces are not basic loading conditions but are derived from the ground motions to represent roughly the inertia forces induced in a structure by the postulated or specified seismic action. While the specified action is constant at a particular location, greatly different seismic design forces are obtained for the same structures depending on the degree of ductility assumed and provided for in the design. Thus it is the ground motions that are the basic seismic actions and not the forces derived from a dynamic analysis or those prescribed in a building code.

NOTE: Written discussion of this paper is welcomed and will be received by the Editor until November 30, 1987 (address inside front cover).

2. Classification of seismic actions

The traditional manner of achieving seismic resistant design of structures has been to determine an equivalent static lateral load which is deemed to represent the dynamic loading effects of the earthquake, and then to design the structure to resist these forces. In the design process, the resulting seismic forces have generally been treated in the same way as wind loads, and are together called "lateral loads." They have been assigned a common load factor $\alpha = 1.5$ (NBCC 1985a). There are a number of reasons, however, for differentiating between wind and seismic loads in Canada and in other parts of the world where similar hazards of these natural phenomena exist:

(1) The frequency of occurrence of high winds corresponding to the specified design loads is greater than that for earthquakes. This is reflected in the return periods of 30 years for the design wind speed, whereas it is 475 years for the establishment of the seismic risk maps (NBCC 1985b).

(2) The statistical behaviour of the two phenomena is quite different. In terms of a plot of amplitude versus probability of exceedance (or return period), the wind speeds start to level off around the 30-year return period whereas the seismic action keeps rising and begins to level off in the order of a few hundred years return period (Ferahian 1971).

(3) Because seismic design forces are large but relatively infrequent, the conscious decision is made — and is incorporated in the seismic code requirements — that large cyclic inelastic deformations are permitted so long as the structure does not collapse. Wind forces, on the other hand, are usually resisted elastically.

While the probability of occurrence of the specified seismic action is small during the life of the structure, it is nevertheless large enough so that it has to be considered as an environmental action. Therefore the structure is designed according to the ultimate limit state and, under these conditions, is considered to be on the verge of collapse. It can thus be argued that the seismic action pertaining to seismic resistant design of buildings should correspond to an "accidental action." As there is no further "factor of safety" required, the load factor associated with these seismic actions should therefore be 1.0.

For wind action the design load may be reached once or a few times during the life of the structure. It is therefore a "variable action" in the International Standard Organization (ISO) terminology and should have a load factor that reflects the relatively high probability that this load can be exceeded; consequently $\alpha = 1.5$ is used. Design conditions that could be compared to earthquakes are tornados. These could also be considered as accidental actions, for which the ultimate limit state and possibly a load factor of 1.0 would be indicated.

The distinction between wind and seismic actions presented above corresponds generally to the description of variable actions and accidental actions, respectively, contained in the Guidelines for Development of Limit States Design (CSA 1981) and in ISO/DIS 2394 (ISO 1984). However, in the CSA document, earthquake loads are listed together with wind under "live loads" or variable actions. This may be due to historical reasons. Also, the seismic zoning in effect in 1981 was based on a 100 year return period and could therefore be viewed more like a variable action. With the zoning map for 1985 NBCC based on a return period of 475 years, however, seismic action now corresponds more clearly to an accidental event.

It is instructive to examine the approach taken by the Japanese seismic code for buildings (Ishiyama 1985). The seismic

design loads are specified for two levels: (1) For moderate earthquakes a shear coefficient C_0 of 0.2 is given corresponding to ground motions that can be expected a few times during the life of the building. The design for this specified loading is to be carried out elastically and under such an earthquake the building would be expected to sustain little or no damage. (2) For a major earthquake, $C_0 = 1.0$ is prescribed, for which the calculation and verification of the ultimate lateral shear strength is required. Nonlinear ductile behaviour and increased damping values can be utilized. Thus the Japanese code in effect specifies both a frequent (variable) and an infrequent (accidental) action, whereas the NBCC only prescribes what amounts to an accidental one.

3. Limit states

The limit states applicable to seismic resistant design are the ultimate limit states concerning safety, and the serviceability limit states which concern restriction of normal use and occupancy and durability (CSA 1981; ISO 1984; Allen 1982). The overwhelming preoccupation of earthquake engineering in the past has been with the prevention of collapse of structures, i.e., the ultimate limit state. There are, however, situations where the serviceability limit state is also of interest. This occurs when continuation of the function or preservation of the contents of a building during and after an earthquake is important. Examples are hospitals, communications facilities and other "life-lines," and important military installations. There may also be economic reasons for trying to achieve a certain level of serviceability as, for example, in a factory, an office building, or buildings housing precious objects; it may be desired to reduce potential losses of function, investment, production, or irreplaceable artifacts.

Whereas the ultimate limit state is reasonably well defined in terms of prevention of collapse, the serviceability limit state needs to be specified in terms of the desired result that is to be achieved. Such serviceability limit states could be: no major cracking, i.e., no inelastic deformation; a limit on maximum interstorey deformation in order to reduce damage to architectural finishes or components; or a limit on the maximum tolerable acceleration in the building. To achieve these objectives different design decisions would have to be taken. For most cases of serviceability, however, a knowledge of what constitutes the load level corresponding to the elastic limit and the degree of nonlinear deformation implicit in the design forces needs to be clarified. The following treatment of force reduction factors will facilitate this determination of load levels that correspond to the elastic limit of structural behaviour.

4. Relationship between K -factors and force reduction factors

The design forces specified in building codes are considerably smaller than the levels corresponding to elastic structural response that results from the given seismic actions. In the NBCC (1985a, b) the force level is a function of K -factors, with an additional overall scaling factor contained in the expression for the seismic response factor S . The K -factors are merely a means of assigning relative merit to different structural systems on the basis of observed or expected behaviour in earthquakes; their numerical values have no absolute rational interpretation. The K -factors can be related, however, to the expected plastic behaviour of a structure and the correspondingly lower forces required during the dynamic response to

FIG. 1. Force-deformation relation for simple elastic-plastic systems.

FIG. 2. Seismic response factor S for the 1985 NBCC.

seismic ground motions. As such, these factors are an indicator of a "system ductility" — not to be confused with "member ductility." Although system ductility for multistorey structures cannot yet be defined simply, the notion is useful in qualitative terms and can be applied quantitatively to single-degree-of-freedom (SDOF) oscillators. Reference to the SDOF oscillator has to be made until such time that a simple quantitative definition for more complex structures becomes available.

The K -factors in the NBCC (1985a, b) are used as multipliers in the calculation of base shear V :

$$[1] \quad V = vSKIFW$$

where v = zonal velocity ratio; S = seismic response factor for the structure (see Fig. 2); I = importance factor; F = foundation factor; W = dead load plus some live or storage loads; and K = factor related to structural behaviour.

The base shear V thus calculated assumes a certain level of ductile behaviour in the structure and corresponds to the yield level required for the seismic response of the structure. This is shown diagrammatically in Fig. 1. If the structure were to remain elastic, a much higher elastic capacity V_e would be required. The difference between these two force levels needs to be provided by the ductile behaviour of the structure and its members.

While it is not usually practical to design a structure to remain elastic under severe seismic loading, it is nevertheless

useful to calculate the elastic load level V_e that corresponds to the specified seismic action, and then to apply a force reduction factor R :

$$[2] \quad V = V_e/R = v\bar{S}IFW/R$$

Here, $\bar{S}$ is an elastic response spectrum similar to but larger than S in [1]. The numerator represents the base shear of the elastic response and R in the denominator accounts for the force reduction on account of ductile behaviour of elastic-plastic systems and (or) increased damping. This is the format employed by the Applied Technology Council seismic code recommendations (ATC-3 1978). For calibration purposes, the case is chosen for which the velocity seismic zone is equal to the acceleration seismic zone, i.e., $Z_v = Z_a$. This represents an average condition throughout the country.

To find the relationship between the currently used K -factors and the corresponding force reduction factors R , [1] and [2] are equated, so that

$$[3] \quad SK = \bar{S}/R$$

As $\bar{S}$ is taken to represent an elastic response spectrum for a single-degree-of-freedom oscillator, it will need to have an upper bound of 3.0 in the low-period range corresponding to the amplification factor 3.0 applicable to the acceleration portion of response spectra for 5% critical damping (Newark *et al.* 1973). In the 1985 NBCC, $S = 0.22/T^{1/2} \leq 0.44$ (Fig. 2); this expression has to be scaled so that $\bar{S} = 1.5/T^{1/2} \leq 3.0$, in order that the bounding value 3.0 be equal to the elastic amplification factor in the short-period region. Thus [3] becomes

$$[4] \quad 0.22K = 1.5/R$$

$$[5] \quad R = 6.8/K$$

This results in the following correspondence between K for the 1985 NBCC and the force reduction factor R of [2]:

K -factor (1985 NBCC)	Derived force reduction factor R
0.7	9.71
0.8	8.50
1.0	6.80
1.3	5.23
2.0	3.40
3.0	2.26

The above result shows that in order to achieve elastic behaviour against the ground motions that are implied by the building code, the specified code forces would have to be multiplied by R corresponding to the applicable K -factors listed above.

The correspondence between K and R has been established assuming that identical load factors apply to both calculations. This load factor is $\alpha_0 = 1.5$ specified in the 1985 NBCC for service loads and other environmental loads such as wind and earthquakes. As was pointed out, however, there are strong arguments for treating the specified seismic effects in the building code as accidental actions for which the load factor α_0 should be 1.0. For the ultimate seismic loads to remain constant and changing α_0 from 1.5 to 1.0 requires dividing the R -values derived above by 1.5 to give:

FIG. 3. Normalized average response spectrum and design seismic response factors.

K -factor ($\alpha_Q = 1.5$)	Force reduction factor R ($\alpha_Q = 1.0$)
0.7	6.47
0.8	5.66
1.0	4.53
1.3	3.49
2.0	2.27
3.0	1.51

The above derivation of R is merely a change in format from [1] to [2], and does not involve a change in seismic design forces or seismic resistance of buildings. The advantages of this new format will be discussed later.

Just as the K -factor in the 1985 NBCC is a measure of the ductility of the structure, so is the proposed R -factor but in the inverse ratio. Increasing values of R generally denote greater structural ductilities, but a simple quantitative relationship for multi-degree-of-freedom structures is currently not available.

5. Comparison of K and R of NBCC and ATC-3

With α_Q equal to 1.0 there is now a consistent basis for comparing the "response modification factors" $\bar{R}$,¹ suggested by the Applied Technology Council (ATC-3 1978), since it also uses $\alpha = 1.0$ as a load factor. For comparable conditions, however, ATC-3 employs the relationship $1.2/T^{2/3} \leq 2.5$ rather than the scaled $\bar{S} = 1.5/T^{1/2} \leq 3.0$. This is shown in Fig. 3, and in the short- and medium-period range amounts to a difference of about 20%. This difference will not be utilized

in the subsequent comparison, but could of course be applied to the final result if this were desired.

The comparison between K , R , and $\bar{R}$ is shown in Table 1, where the structural types in NBCC and ATC-3 have been matched in the most suitable manner. The derived R 's for the NBCC and the $\bar{R}$'s of ATC-3 show good agreement in general. The largest discrepancies occur for case 1, the most ductile category, for which ATC-3 gives higher values, and category 7, unreinforced masonry or other structural categories, for which ATC-3 gives a lower rating than 1985 NBCC.

6. Classification of structural systems and K -factors

In the seismic provisions of the National Building Code of Canada (and similarly of the Structural Engineers Association of California (SEAOC 1980) and the Uniform Building Code (UBC 1985)), various structural systems are assigned relative values of K , where K is defined in the NBCC 1985 as "numerical coefficient that reflects the material and type of construction, damping, ductility and (or) energy-absorptive capacity of the structure as given in Sentence . . ." Although the table for K in the 1985 NBCC appears at first sight without any inherent rationale, the following grouping can be discerned: (1) primary structural forms; (2) combined systems; and (3) special or sensitive structural forms.

(1) *Primary structural forms*: These consist of the following structural types with the associated K -factors: (a) Ductile frame, $K = 0.7$; (b) Ductile flexural wall (ductile reinforced concrete, K -braced or tension-compression braced frames), $K = 1.0$; (c) Regular shear wall (infill frame, reinforced masonry, regular reinforced concrete), $K = 1.3$; and (d) Cross-braced frames (tension-tension system), $K = 1.3$.

(2) *Combined systems*: These are structural forms that consist of two or more different structural types that are primary

¹Although the symbol R is used in ATC-3, $\bar{R}$ will be used here for clarity.

TABLE 1. Comparison of K of 1985 NBCC, derived R , $\bar{R}$ of ATC-3

Case	1985 NBCC				ATC-3 (1978)	
	K^*	R^*	$R^\dagger$	Abbreviated description	$\bar{R}^\dagger$	Abbreviated description
1	0.7	9.71	6.47	Ductile moment resistant space frame	8 7	Special moment frames — steel — reinforces concrete
2	0.7	9.71	6.47	Dual structural system consisting of 25% moment resistant space frame and ductile flexural walls	6 6 1/2 8	Dual system of 25% special moment frame and shear walls of — braced frames — reinforced masonry — reinforced concrete, all loads shared in proportion to their relative rigidities
3	0.8	8.50	5.66	Dual structural system 25% moment resistant space frame and 100% capacity shear walls	5 1/2 4 1/2 5	Dual system of space frame and 100% shear walls of — reinforced concrete — reinforced masonry — braced frames
4	1.0	6.80	4.53	Ductile flexural walls	4 1/2	Reinforced concrete shear walls (does not correspond well)
5	1.3	5.23	3.49	100% capacity infilled shear wall and 25% ductile moment resistant frame		No corresponding classification
6	1.3	5.23	3.49	Ordinary reinforced concrete, braced steel or reinforced masonry	4 1/2 2 4	Ordinary moment frames — steel — reinforced concrete — braced frames
7	2.0	3.40	2.27	Unreinforced masonry and others	1 1/2	Unreinforced or partially reinforced masonry shear walls
8	3.0	2.26	1.51	Braced elevated water tanks on 4 or more legs	2 1/2 1 1/2	Inverted pendulum structures — special moment steel, reinforced concrete — ordinary steel frame

* $\alpha_Q = 1.1$. $^\dagger\alpha_Q = 1.0$.

system by themselves. Their combined action under major seismic motion is considered to be beneficial on account of increased redundancy, and also because of the notion of a backup system in case the major load-resisting element fails. These combined systems include the following, with the applicable K -factors and reference to above primary systems given in parentheses: (a) flexural wall (1b), together with an interactive ductile frame (1a), $K = 0.7$; (b) regular shear wall (1c) or braced frame (1d), together with ductile frame (1a) (100% V taken by wall, 25% by ductile frame), $K = 0.8$.

(3) *Special or sensitive structural forms*: This includes structural systems not included above: (a) all others not included above (e.g., unreinforced masonry, single element structural forms), $K = 2.0$; (b) cross-braced elevated water towers, $K = 3.0$.

For the primary systems the K -factors increase inversely as the actual or perceived increase in ductility of the systems, resulting in higher design forces for the less ductile ones. For the combined systems, the K -factors are reduced as compared

with those that correspond to the principal element. This grouping is illustrated in Fig. 4.

7. Suggested modifications of NBCC seismic requirements

The above considerations and comparisons indicated three main areas in which the seismic requirements of the NBCC could be modified: (a) load factors for earthquake loads; (b) assignment of force reduction factors; and (c) classification of structures. Following are further reasons in support of such possible changes.

(a) *Load factor for earthquake loads*: The arguments for changing the load factor for seismic loads from 1.5 to 1.0 have already been presented. In addition, the suggested change is desirable in connection with a possible introduction of force reduction factors so that these will result in reasonable values, rather than the somewhat excessive quantities that would be needed if the present load factor of 1.5 were retained.

FIG. 4. Classification of structural systems.

(b) *Assignment of force reduction factors R* : The change in format for the base shear calculation from [1] to [2] is advocated as a desirable development for the following reasons:

(1) The force reduction factor R becomes an explicit parameter that is applied to the elastic-calculated base shear, thus clarifying the basis for the seismic resistant design calculations.

(2) In such a format it is relatively simple to compare the performance of different structural configurations in a qualitative sense, and even quantitatively if some simplifications are adopted.

(3) For new structural systems or materials the assignment of appropriate force reduction factors is simplified when compared to the rather empirical manner of the current K -factors. Since the elastic base shears are readily calculated by [2] or by more sophisticated means such as response spectrum of time history analysis, the force reduction factor can be established by comparing the results with nonlinear dynamic analyses. This introduces the possibility of direct quantitative determination or validation of these code coefficients by analysis or tests. Such an approach could also be employed for classifying new

structural forms or innovative ways of reducing earthquake effects on buildings, such as rubber-steel sandwiches (Tarics *et al.* 1984), roller isolators (Stiemer and Barwig 1985), friction slip devices (Pall 1983), and possibly others. If a certain value of R can be shown to be provided by the device as demonstrated by special analyses or tests, then these greater force reductions (up to a reasonable upper limit) could be recognized to satisfy the minimum requirements of building codes.

(4) This proposed new formulation may provide the basis for and an incentive towards the development of a simple method of formulating expressions for system ductility. This would provide a useful tool for the simplified seismic analysis of structures. For single-degree-of-freedom structures the system ductility can be related to the force reduction factor (Newmark 1970), but for multistorey buildings the relationship is more complex.

(5) Since the code calculations involve the determination of an elastic base shear, a direct link can be made with results obtained from elastic time history and response spectrum analysis.

TABLE 2. Suggested force reduction factors R and corresponding K of 1985 NBCC

Category	Type	Abbreviated description	Suggested R (See note)	K (1985 NBCC)
I.	Primary systems	(a) ductile moment resistant space frame	6.5	0.7
		(b) ductile flexural walls	4.5	1.0
		(c) reinforced concrete shear walls	3.5	1.3
		(d) cross-braced frames	3.5	1.3
II.	Combined systems	(a) combinations of fully ductile systems of type Ia and Ib	6.5	0.7
		(b) 25% capacity of Type Ia with 100% of type Ic or type Id	4.5	1.0
III.	Special structures	(a) all other structural systems not covered by category I or II	2.0	2.0
		(b) structures of nonductile materials with some residual ductility and damping (e.g., nonreinforced masonry, plain concrete)	2.0	
		(c) structures of brittle materials with little or no residual ductility (e.g., glass)	1.0	
		(d) inverted pendulum structures		
		(i) with ductile behaviour	2.0	
		(ii) with tension-compression cross bracing	1.5	3.0

NOTE: Assuming load factor $\alpha_Q = 1.0$ and $S = 1.5/T^{1/2} \leq 3.0$.

(c) *Classification of structures*: It is suggested that buildings can be classified into three categories: (1) primary systems, (2) combined systems, and (3) special structures. Subdivision of each category would then consist of the description of the various structure types and the corresponding value of K for the 1985 NBCC and the suggested R . This is indicated in Table 2. The calculated values of R in Table 1 have been rounded to the nearest 0.5 value.

Under combined systems, the principle of load sharing with a ductile secondary element of 25% nominal capacity is given favourable rating by decreasing K (or increasing R) by one level as compared with the bare primary element structure. Coupled shear walls with ductile connecting beams could also be included in category 2a along with other combined ductile types of construction.

Special structures would include those not included above and constitute mainly those types that are considered nonearthquake resistant or those that have fared poorly in previous earthquakes; this includes unreinforced masonry or other structural forms with low ductility, nonredundant systems, or the elevated cross-braced water tower.

8. Summary and conclusions

Some basic considerations that underlie the structural seismic design requirements of the 1985 NBCC have been examined. These include load classification of seismic action, load factors, limit states to be considered, classification of structural systems, and correspondence of K -factors with derived force reduction factors R . The following conclusions are reached:

1. Seismic actions should be treated as accidental actions rather than as variable actions as is currently done in the 1985 NBCC.

2. The load factors for the ultimate limit state correspond-

ing to the accidental seismic action should be changed to 1.0 from the current value of 1.5. This would introduce a desirable distinction between seismic loads and wind to account for their significantly different characteristics.

3. The correspondence between K -factors and a force reduction factor R has been demonstrated, where the latter refers to a constant in the denominator of the base shear formula and indicates the force reduction relative to the expected elastic response of the structure.

4. A new classification of structures and corresponding K -factors and R -factors has been presented. Structures can be grouped into primary systems, combined systems, and special structures that include those with little or no ductility.

5. When a load factor of 1.0 is used for earthquake design loads, the derived values of force reduction factor R correspond well with the response modification factors proposed by the Applied Technology Council (ATC-3 1978).

6. The above results can provide a basis for possible future changes of the NBCC seismic provisions.

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