Determining Seismic Base Shear - A More Rational Approach

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Abstract

This paper first reviews the different approaches taken by codes of practice in their treatment of ductility by the use of force modification factors. The way in which structural overstrength can be accounted for and the factors influencing overstrength are discussed. Nonlinear analyses of reinforced concrete structures, designed by the Canadian codes, demonstrate the significance of structural overstrength. The manner in which structural overstrength can be accounted for in the design of reinforced concrete structures is presented. Concern is expressed about a separate requirement for avoiding yielding under service conditions and limiting interstorey drifts at service load levels.

Introduction

In order to have a more rational approach for determining seismic design base shears, it is first necessary to appreciate the roles of both structural ductility and structural overstrength. North American codes determine seismic design base shear by dividing the elastic base shear by a force modification factor. This paper first reviews the seismic base shear formulation used in Canada, the U.S. and Mexico. The significance of accounting for structural overstrength in determining the design base shear is demonstrated by comparing non-linear dynamic analyses of different structures. The study of the design philosophy used enables the determination of minimum values of structural overstrength factors for a particular code. A design equation is proposed, which accounts for structural ductility and structural overstrength. Additional requirements which ensure that no yielding takes place under service conditions and that limit the interstorey drifts at service load levels may be necessary.

Design Base Shears in North American Codes

The NEHRP Recommended Provisions (NEHRP, 1985) use seismic response modification coefficients, R_N, which are used to determine the minimum design base shear, V. The values of R_N were chosen to account for:

(i) the performance of different systems in past earthquakes,

(ii) the ability of different structural systems to absorb energy without serious degradation, and

(iii) the amount of damping in different structural systems in the non-linear range.

Essentially, the NEHRP Recommended Provisions determine the minimum design base shear, V, as:

$$V = V_e / R_N \qquad (1)$$

where V_e is the elastically-determined base shear. The value of R_N ranges from 1.25 for unreinforced masonry bearing walls to 8.0 for special moment frames. Table 1 gives values of R_N for different concrete structural systems. Also given in the NEHRP Recommended Provisions are the values of the deflection amplification factor, C_d, for these systems. The factor, C_d, accounts for the magnification of the computed elastic displacements due to inelastic action. This factor varies from 1.25 to 6.5 depending on the ductility of the structural system and the type of material.

The minimum seismic design base shear given in the National Building Code of Canada (NBCC, 1990) is:

$$V = (V_e U) / R \qquad (2)$$

where R is the force modification factor which reflects the ability of the structure to dissipate energy through inelastic action. This factor, R, ranges from 1.0 for unreinforced masonry to 4.0 for ductile moment-resisting concrete or steel frames. Table 1 gives the force modification factor, R, recommended by the National Building Code of Canada (1990) for concrete structures. The factor U was intended to be a code calibration factor attempting to maintain the same "level of protection" as in previous code editions. The value of U is set equal to 0.6 in the 1990 NBCC. In order to estimate realistic values of lateral displacements, the calculated elastic deflections corresponding to V are multiplied by R to account for inelastic response. Hence the factor, R, can be thought of as the "structural ductility factor". More information on the design procedures used in Canada is given by Mitchell and Paultre (1991) and Uzumeri (1993).

The code for the design of structures for the Federal District of Mexico City (Instituto de Ingeniería, 1987) determines the seismic design base shear for long-period buildings from the following equation:

$$V = V_e / Q \qquad (3)$$

where V_e is the elastic seismic coefficient for the three major soil zones within the city, and Q is the equivalent ductility factor. Before the 1985 Mexico earthquake, the Q factors varied from 2.0 to 6.0 for reinforced concrete structures. Following this major earthquake, the Q factors were adjusted as shown in Table 1, with the maximum value of Q being 4.0.

Idealized concept of Ductility for Single-Degree-of-Freedom Systems

Figure 1 illustrates the concept of ductility for a single-degree-of-freedom (SDOF) system, where μ is the structural ductility factor. For such simple systems, the design base shear is taken as $V = V_e/\mu$ Upon first yielding, the elastic-perfectly plastic SDOF systems exhibit a load-deflection response with a flat plateau.

Influence of Ductility and Overstrength on Response of Multi-Degree-of-Freedom Systems

In order to study the influence of ductility and overstrength, the response of two different structural systems, a moment-resisting frame system and a frame-wall system, will be investigated. Figure 2 shows the "push-over" response, determined using the

Table 1: Force modification factors for the U.S., Canada, and Mexico.

CODE	Type of lateral load-resisting system	Force modification factors
NEHRP 1988		R
	Reinforced concrete shear walls (bearing walls)	4.5
	Reinforced concrete shear walls	5.5
	Special moment frames	8.0
	Ordinary moment frames	2.0
	Intermediate moment frames	4.0
	Reinforced concrete shear walls with special moment frame	8.0
	Reinforced concrete shear walls with intermediate moment frame	6.0
NBCC 1990		R
	Ductile moment-resisting space frame	4.0
	Ductile flexural wall	3.5
	Moment-resisting space frame with nominal ductility	2.0
	Wall with nominal ductility	2.0
	Other concrete lateral force-resisting systems not described above	1.5
MEXICO 1987		Q
	Ductile moment-resisting frames of concrete or steel meeting special design and detailing requirements	4.0
	Dual systems of moment-resisting frames together with either braced frames or concrete shear walls meeting special design and detailing requirements	3.0
	Frames (braced or unbraced) with beams and columns not satisfying the two cases above and masonry walls meeting special confinement requirements	2.0
	Hollow block masonry walls with or without frame members	1.5
	Other systems not described above	1.0

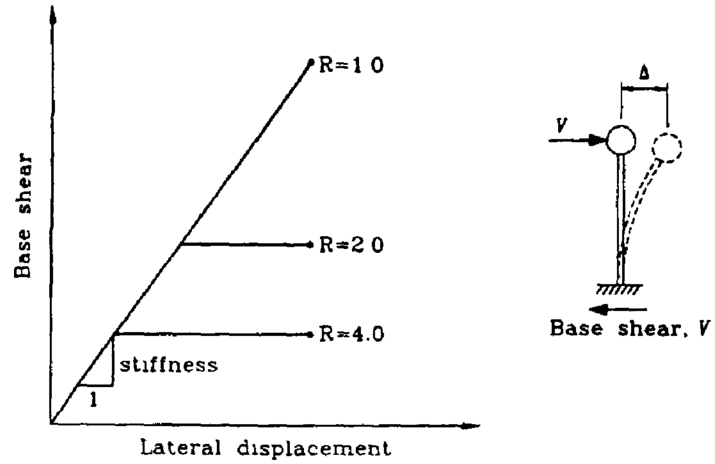

Figure 1: Effect of inelastic action and energy dissipation on maximum base shear

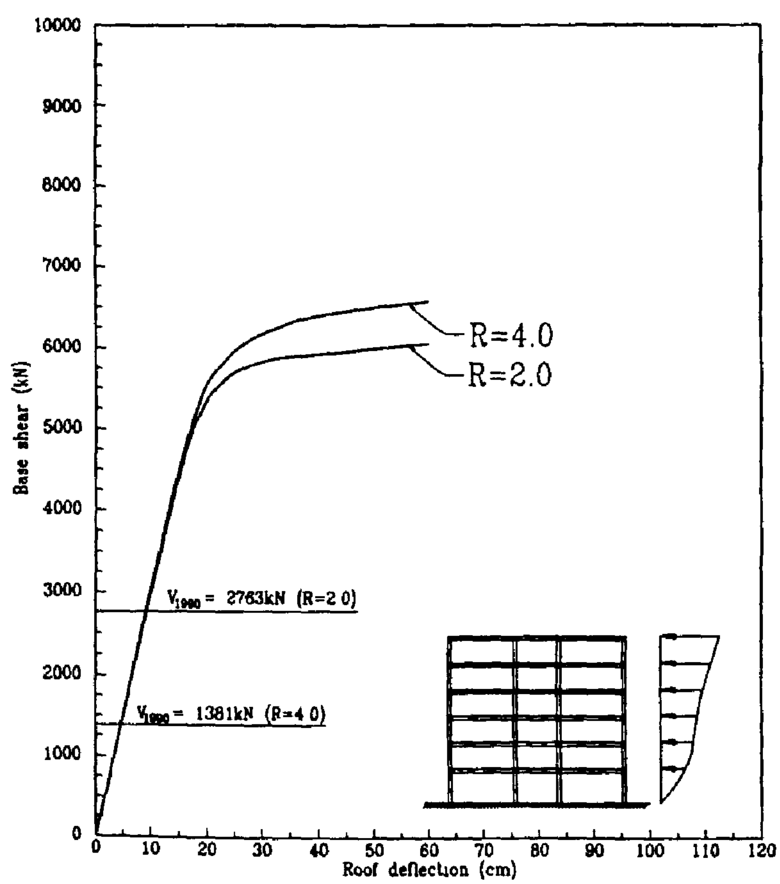

Figure 2: Comparison of predicted responses for 6-storey frame structures designed for Montreal with $R = 2.0$ and $R = 4.0$.

nonlinear analysis program developed by Mondkar and Powell (1975), of two six-storey frame structures designed for a moderate seismic zone (Montreal). One structure was designed as a ductile moment-resisting frame with R = 4.0, and the other structure was designed for nominal ductility, with R = 2.0. Contrary to the responses of the SDOF systems, the six-storey structure with R = 4.0 has a strength greater than the strength of the structure with R = 2.0! This at first seems surprising since the R = 4.0 structure had been designed for half the base shear used to design the R = 2.0 structure. Figure 2 demonstrates that the SDOF analogy to account for ductility is not directly applicable to multi-degree-of-freedom systems. The redundancy, hierarchy of yielding and member overstrength all contribute to structural overstrength. The effects of redundancy and hierarchy of yielding are not present in SDOF systems.

Figure 3 shows the predicted responses of two 12-storey frame-wall structures designed with R = 3.5 and R = 2.0 for Montreal. Once again the R = 3.5 structure is stronger than the R = 2.0 structure due to structural overstrength.

Dependable Levels of overstrength

In formulating the design procedure which directly accounts for minimum or dependable values of structural overstrength, it is necessary to examine the code design process for different levels of force modification factors.

The Canadian codes distinguish between structures exhibiting "nominal ductility" and those considered to possess higher levels of ductility. In the 1984 CSA Standard, the factored moment resistance, M_r, is calculated using a factored concrete compressive strength of $\phi_c f'_c$ and a factored steel resistance, of $\phi_s f_y$, where $\phi_c = 0.6$ and $\phi_s = 0.85$. The probable moment resistance, calculated with $\phi_c = \phi_s = 1.0$ and an equivalent yield stress of $1.25f_y$, is equal to about $1.47M_r$, for pure flexure, and about $1.57M_r$ for flexure and axial load. The nominal flexural resistance, M_n, calculated using $\phi_c = \phi_s = 1.0$ is equal to about $1.2M_r$ for pure flexure. The probable resistance corresponds to the expected strength for "ideal" construction with the steel exhibiting stresses above the specified yield stress. The nominal resistance corresponds to "ideal construction and materials having their specified values. The factored resistance represents construction within the code tolerances and material resistances somewhat less than ideal. The variation between the factored and probable resistances can be thought of as likely ranges of capacities due to quality of construction, quality of materials and the ability to develop large strains. The CSA Standard uses these definitions of resistances in ensuring a hierarchy of failure for ductile moment-resisting frames (R = 4.0). For example, the factored shear resistance of the beams must exceed the shear corresponding to the development of probable moment resistances at the ends of the beams. In order to ensure that the columns are stronger than the beams, the CSA Standard requires that the sum of the factored flexural resistance of the columns at a joint exceeds 1.1 times the sum of the nominal resistances (i.e., $1.1 \times 1.2M_r = 1.32M_r$) of the beams framing into the joint. All of the joints in a structure designed with R=4.0 are designed and detailed such that the joints can transmit a shear corresponding to the development of probable moment resistances in the beams framing into the joint. Additional design and detailing requirements provide minimum confinement reinforcement in columns and joints, adequate anchorage of longitudinal reinforcement, sufficient hoops in beams to prevent buckling of longitudinal beam bars, etc.

Fajfar and Fischinger (1990) and Nassar and Krawinkler (1991) have described the many factors that contribute to overstrength. Paulay and Priestley (1992) have defined the overstrength factor, for any level in a ductile structural system, as the sum of the probable strength (accounting for all possible fctors that contribute to strength) divided by the required seismic design resistance at that level. This is a very general definition of the system overstrength factor, S, which can be determined once the design has been completed. This definition can also be used in the seismic evaluation of existing structures.

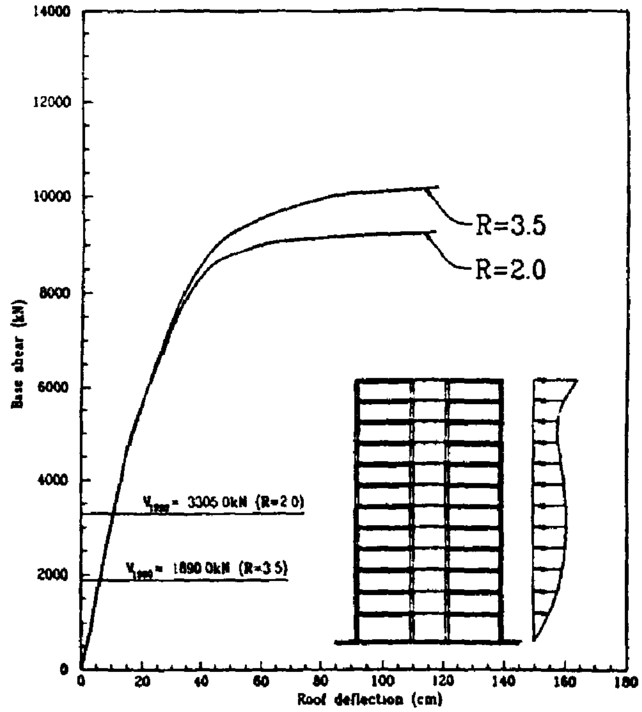

Figure 3: Comparison of predicted responses for 12-storey frame-wall structures designed for Montreal with $R = 2.0$ and $R = 3.5$.

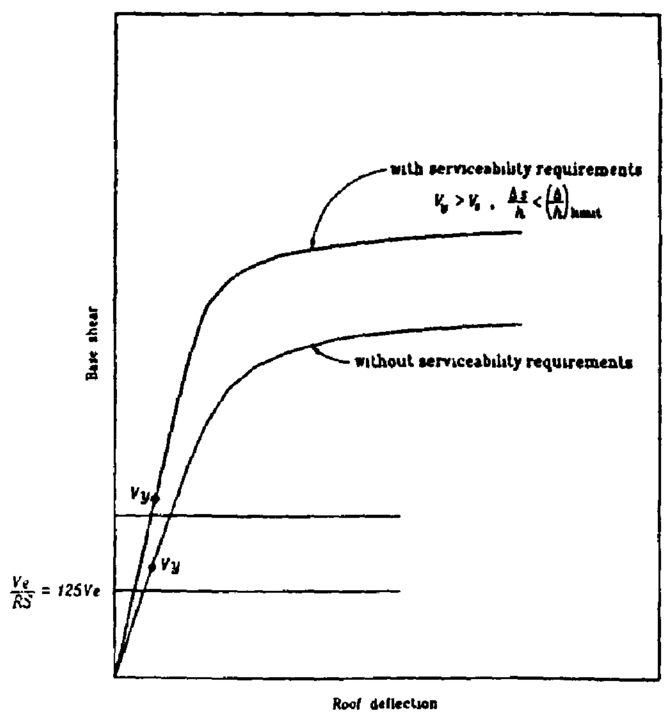

Figure 4: Satisfying serviceability conditions

A rational design approach would be to first design a structure by assuming a system overstrength factor, carrying out the design and detailing of all the members and then checking
the system overstrength factor by the method described above. A simpler, more conservative procedure would be to determine a minimum or dependable overstrength factor as:

$$S = S_H S_L S_O \tag{4}$$

where S_H is the overstrength factor arising from the design requirements for hierarchy of yielding, S_L is the overstrength factor due to other governing loading cases such as wind, and S_O is the overstrength factor coming from other sources, such as the strength enhancement due to the provision of minimum or confinement reinforcement. The factor, S_O, can only be determined once the design and detailing have been completed. The factor, S_L, can be determined once the structural analyses for different loading cases have been completed. If, for example, wind controls the design at a particular level, S_L can then be estimated from:

$$S_L = F_W/F_E \tag{5}$$

where F_W and F_E are the design forces due to wind and earthquake, respectively.

In order to estimate the factor, S_H, simple structural systems will be examined. For a single cantilever wall system having as an $R = 3.5$, the probable moment resistance at the base of the wall varies between $1.47M_r$ and $1.57M_r$ depending on the level of axial load acting on the wall. Hence, a minimum value of about $1.5M_r$ is appropriate, giving $S_H = 1.5$. For other types of single walls ($R \leq 2.0$) no hierarchy of failure is required by the code, and hence, the dependable overstrength factor is approximately $1/\phi_s = 1.2$. For a frame-wall system, redundancy plays an important role, giving rise to even larger values of S_H. As can be seen from the predicted response of frame-wall structures (Figure 3), the factors S_L and S_O can significantly increase the system overstrength, particularly for ductile structures located in moderate seismic zones.

For a simple ductile-frame structure ($R = 4.0$) the overstrength factor, S_H, is that arising from the hierarchy of failure required by the code. Consideration of sequence of events with hinging in the beams followed by column hinging gives an $S_H = 1.1 \times 1.2 \times 1.5 = 1.98$, or about 2.0. All other frame structures ($R \leq 2.0$) would have an overstrength factor, S_H, of about 1.2. In the design of structures with $R = 2.0$ if the columns are made stronger than the beams, it may be possible to develop higher values of S_H (Paultre et al., 1989). As with the frame-wall structures, larger system overstrength can be expected due to S_L and S_O (Figure 2), particularly for ductile structures located in moderate seismic zones. If moment redistribution is accounted for in the design of the beams, this would reduce the value of S_H by the amount of the redistribution.

It is interesting to compare the value of $S_H \times R$ with the force-modification factors, R_N, used in the NEHRP (1988) provisions. For ductile moment-resisting frames, $S_H \times R = 2.0 \times 4.0 = 8.0$, which is the same value as R_N. For ordinary moment frames, $S_H \times R = 1.2 \times 1.5 = 1.8$, which is slightly less than the corresponding value of 2.0 for R_N.

Serviceability Considerations

Figure 4 shows possible responses for a structure designed with $R = 4.0$. If no serviceability conditions are imposed on the design, then the response would be shown by the lower curve in Figure 4. As can be seen, the first yield occurs at load level, V_y less than the "service" level base shear, V_s, and in addition, the maximum interstorey drift ratio of the structure at service load, Δ_s/h, is greater than the limiting interstorey drift ratio,

$(\Delta/h)_{limit}$. The upper curve in Figure 4 illustrates the influence of increasing the stiffness of the structure and increasing the strength of the members to satisfy serviceability conditions (i.e., $V_y \geq V_s$ and $\Delta_S \leq \Delta_{limit}$). Bertero and Bertero (1993) formulated a conceptual seismic code approach which relies on limiting calculated damage indices in order to satisfy serviceability conditions.

Conclusions

Nonlinear response predictions of six-storey frame structures and 12-storey frame-wall structures have enabled an assessment of the significance of structural overstrength for structures designed with the Canadian Code approach. It was demonstrated that structures designed with higher levels of ductility (R = 4.0 and R = 3.5), and hence lower design force levels, can in fact exhibit higher capacities than structures designed for lower levels of ductility. Hence ductile multi-degree-of-freedom systems are strongly influenced by overstrength.

A design approach accounting for overstrength in reinforced concrete structures is presented. Structural overstrength is divided into three components: a factor accounting for hierarchy of yielding, a factor accounting for design forces controlled by other load cases (e.g. wind), and a factor accounting for other causes such as provision of minimum and confinement reinforcement. Factors influencing structural overstrength are described and methods for their determinations are given. Minimum or dependable overstrength factors based on the hierarchy of yielding are derived for reinforced concrete structures. Overstrength factors for steel, timber and masonry structures can be derived in a similar fashion.

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Seismic Risk Reduction: The need for and Importance of Earthquake-Resistant Design and Construction of Structures.

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Abstract

General remarks about the nature of the earthquake (EQ) problem, the conditions that determine the occurrence of an EQ disaster, and the seismic risks in our urban and rural areas are followed by discussion of the role and importance of EQ Engineering in the overall problem of controlling these risks, with particular focus on EQ-Resistant Design (EQ-RD) and EQ-Resistant Construction (EQ-RC) and upgrading of existing seismically hazardous facilities. The need for an EQ preparedness programme is emphasized. Analysis of recent EQs [the 1985 Chile and Mexico EQs, the 1988 Armenia EQ, the 1989 Loma Prieta EQ, the 1990 Iran and Philippines EQs and the 1992 Erzincan (Turkey) EQ] shows that seismic risks are increasing rather than decreasing. Evaluation of the lessons learned from these EQs and from research results provides the reasons for this increase, pointing out that the lessons learned are either quickly forgotten or not taken seriously, and that there is an urgent need for EQ preparedness to control seismic risks. It is emphasized that, because EQs are inevitable, control of seismic risks requires control of the vulnerability of the built environment, which is a complex problem requiring the integration of knowledge and collaboration of experts from many different disciplines. The state of the knowledge and particularly the state of the practice in EQ-RD of new structures and seismic upgrading of existing facilities are discussed, and major issues and pressing problems requiring short-term solutions are identified. After a brief discussion of the importance of proper EQ-RC and monitoring of the function/use and maintenance of the facilities, the main issues in the formulation and application of seismic code procedures in EQ-RD are identified, with emphasis on the establishment of reliable design EQs. Solutions are suggested based on an energy approach. A conceptual methodology for the numerical design part of EQ-RD is proposed. This methodology is based on well-established fundamental principles of structural dynamics, mechanical behaviour of entire facility systems and comprehensive design, and is in compliance with the worldwide-accepted philosophy of EQ-RD. The main advantages of this conceptual methodology are discussed, and the successful application of this methodology to the preliminary design of a 30-storey RC building is illustrated. Recommendations for improving EQ risk reduction are offered.