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CONCEPT OF OVERSTRENGTH IN SEISMIC DESIGN

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ABSTRACT

Observations during many earthquakes have shown that building structures are able to sustain without damage earthquake forces considerably larger than those they were designed for. This is explained by the presence in such structures of significant reserve strength not accounted for in design. Seismic codes now implicitly rely on such reserve strength. The possible sources of such strength are outlined and it is reasoned that a more rational basis for design would be to account for such sources in assessing the capacity rather than in reducing the design loads. As an exception, one possible source of reserve strength, the redistribution of internal forces, may be used in scaling down the design forces. This is because such scaling allows the determination of design forces through an elastic analysis rather than a limit analysis. In order to assess the extent of reserve strength attributable to redistribution, a series of moment-resisting steel building frames from 2 to 30 storeys is analyzed. A static nonlinear push-over analysis is used in which the gravity loads are held constant while the earthquake forces are gradually increased until a mechanism forms or the specified limit on inter-storey drift is exceeded.

KEYWORDS

Seismic design; overstrength factor; reserve strength owing to redistribution; moment-resisting steel frames; push-over analysis.

INTRODUCTION

The generally accepted objectives in the seismic design of a building are to ensure that (1) the life safety of the users and the general public is preserved in the event of the maximum credible earthquake that the building may experience, and (2) that the building suffers no structural damage under a less severe but more frequent earthquake. For special structures additional objectives may be defined. For the seismic design of normal buildings most codes, in fact, specify only a single design earthquake which the building and its components are required to sustain without collapse. The building is expected to undergo some structural and nonstructural distress during the design earthquake. It is assumed that the building designed in this manner will automatically meet the goal of no damage in a moderate earthquake.

In keeping with the foregoing principle, the codes require that the building be designed to resist a base shear obtained from an idealized response spectrum which is representative of the design earthquake and hence of the seismicity of the site. The elastic base shear is reduced to account for inelasticity in the structure. The amount of such reduction is dependent on the capacity of the structure to undergo inelastic deformation without collapse, often referred to as the ductility capacity. An additional reduction is permitted in recognition of the observed fact that structures designed according to the provisions of the code possess considerable reserve of strength beyond that required by the design forces.

The National Building Code of Canada (NBCC) (Canadian, 1995) defines the design earthquake as one that has a 10% likelihood of being exceeded during a period of 50 years. The corresponding elastic base shear V_e is obtained from

$$V_e = vSIFW \tag{1}$$

where v is the zonal velocity ratio representing the seismicity of the site, S is the seismic response factor, I is the importance factor, F is the foundation factor and W is the gravity load contributing to inertia forces. The seismic response factor S is a function of the fundamental natural period of the structure. Thus, vS represents the spectral acceleration in units of g. The elastic base shear is reduced as follows to obtain the design base shear.

$$V = V_e \frac{U}{R} \tag{2}$$

where the reduction factor R is a measure of the ductility capacity of the structure, and the calibration factor U accounts for the reserve strength or overstrength. In NBCC, $U = 0.6$ and R varies from 1 to 4.

In US the Uniform Building Code (UBC) (International, 1991) stipulates that the building be designed for a base shear given by

$$V = \frac{ZIC}{R_W} W \tag{3}$$

where Z is the seismic zone factor, I is the importance factor, C is the seismic response factor, W is the seismic weight, and R_W is a reduction factor. Factor ZC represents the elastic spectral acceleration in units of g. The magnitude of the reduction factor R_W varies from 4 to 12. In UBC the design earthquake forces are supposed to be used in association with the calculated strength at working stress level. For the purpose of comparison with NBCC, it is useful to convert the design forces to correspond to yield level. This can be done by applying a load factor of 1.5 to the forces or by changing the reduction factor to $R = R_W/1.5$. The maximum value of the modified reduction factor thus works out to 8. Such a large value can not be justified purely on the basis of the ductility of the structure. Evidently the reduction factor of UBC is a composite factor accounting for both ductility and overstrength.

It is apparent that both the US and the Canadian Code provisions rely on the presence of significant overstrength in the structure. The recognition of the overstrength is explicit in NBCC, with the provision of factor U, while it is implied in UBC with the use of composite reduction factor R representing both ductility and overstrength. In either case, the manner in which the overstrength is accounted for in design is not entirely rational. First, the reserve strength should, in principle, be used in assessing the capacity of the structure and not in scaling the design forces. Second, it is necessary that the sources of reserve strength be clearly recognized so as to judge whether the contribution to reserve strength from a particular source can, in fact, be relied upon.

FACTORS THAT AFFECT RESERVE STRENGTH

Observations of structural performance under many past earthquakes have led to the conclusion that code designed buildings must possess significant overstrength in order for them to have survived without damage earthquake forces considerably larger than those considered in design. Many researchers have attempted to identify the factors that may have contributed to the observed overstrength (Mitchell and Paultre, 1994; Nassar and Krawinkler, 1991). These attempts have been useful in understanding the phenomenon of overstrength but may have also led to the belief that the identified sources of overstrength can be counted upon in designing new building structures. This belief is not justified in all cases and yet researchers are attempting to quantify the overstrength and are developing recommendations for its use in reducing the design seismic forces (Uang, 1992; Mitchell and Paultre, 1994). A critical examination of the factors that contribute to the reserve strength is necessary to understand when and if a particular source of overstrength can be relied upon. For this purpose, it is useful to divide the contributing factors into several categories as outlined below.

Factors that involve uncertainty

The factors included in this category may have been, at least in part, responsible for the observed overstrength and satisfactory performance during past earthquakes of buildings designed to meet the requirements of current earthquake codes. However, the amount of contribution, if any, from these factors is not certain and can not be relied upon in the design of new building structures. The following is a partial list of factors that belong to this category

(a) The difference between the actual strength of the material used in construction and the strength used in calculating the capacity.

Most material standards specify an acceptance criterion to ensure that the probability of the material strength falling below its nominal value used in design is reasonably small. The mean strength is in general higher than the nominal strength. Furthermore, in estimating the capacity of the structural member, codes using a limit states approach to design stipulate that a resistance factor less than 1 be applied to the specified nominal strength. The magnitude of this factor depends on the variability of the material and the quality of control. The underlying philosophy is to minimize the risk of the member capacity falling below its estimated value. Evidently, in a majority of cases the actual capacity will be larger than its estimated value. This may be cited as one reason why structures have been able to sustain seismic loads significantly larger than those used in design. However, the excess capacity can not be used in design, for that will increase the risk of the capacity falling below that estimated and will be contrary to the philosophy of limit states design. If it is argued that a greater risk is acceptable in designing for earthquake forces than for say wind forces, the right thing to do would be to make the specified nominal value of strength closer to the mean and/or to use a resistance factor nearer to 1.

(b) Effect of using discrete member sizes, for example, selection of members from a discrete list of available sections, and the use of limited bar sizes and arrangement in concrete structures.

It is true that the use of discrete member sizes will lead to a capacity that is higher than required. However, the amount of overstrength is quite uncertain and can not be relied upon in design.

Factors that can not be accounted for because of lack of knowledge

There are a number of factors that are known to contribute to strength but which are difficult to quantify because of the complexity of the behaviour and/or lack of knowledge. The following can be cited as examples.

(a) Use of conservative models for predicting member capacities.

(b) Effect of nonstructural elements, such as for example, infill walls.

(c) Effect of structural elements that are not included in the prediction of lateral load capacity, for example, contribution of reinforced concrete slabs, contribution of columns in flat plate structures with shear walls, increased resistance due to concrete confinement, and reduced stiffness due to concrete cracking.

Factors that can be but are not commonly accounted for in calculating the capacity.

In many cases the contribution from an identified source of additional strength can be estimated at the time of design. The correct approach in such a case would be to design for realistic loads instead of the reduced loads and at the same time to account for the contributing factors in estimating the capacity. The alternative procedure to ignore the contribution from the source of strength and to reduce the design load by an arbitrary factor which may have no relation to the magnitude of the strength contribution is not rational. The following are examples of factors that belong to this category.

(a) Effect of minimum requirements prescribed by the code. It is not necessary to reduce the design loads and then to implicitly rely on the extra capacity resulting from the use of member sizes, reinforcement etc. dictated by the minimum requirements of the code. The use of realistic loads should not result in any change in member design as long as the capacity of the member is calculated on the basis of actual size and reinforcement provided.

(b) Architectural consideration that dictate provision of extra or larger structural members, for example, shear walls. Again, use of realistic loads instead of the reduced load will not alter the design as long as all structural members are included in calculating the capacity.

(c) Control of design by other loading cases, for example, wind. In this case too, the use of realistic earthquake forces will not change design.

Factors related to simplification in design procedure

Often, simplifying assumption are made in design that lead to overestimation of strength demand or underestimation of capacity. This approach may be necessary for routine design where the extra effort required in obtaining more accurate estimates of demand and capacity is not justified. The following are some examples.

(a) Use of single degree-of-freedom spectra along with assumed load distribution to estimate the demand on multi-degree-of-freedom systems. While this may be a source of overstrength in some cases, in others it may lead to underestimation of strength demand. Additional research is needed to address this issue.

(b) Redistribution of internal forces in the inelastic range. In seismic design the capacity of the structure is usually determined at the first yield. This allows the use of analysis procedures that are applicable to elastic structures. Redistribution allows the structure to resist forces that are significantly higher than those causing first yield. Thus the lateral force at which a mechanism will form in a frame structure is often considerably higher than that at which the first plastic hinge will form. In a similar manner, the redistribution of lateral force from a compression brace to a tension brace allows a braced structure to carry significantly higher lateral force than at compression brace buckling.

ACCOUNTING FOR OVERSTRENGTH IN DESIGN

The discussion in the previous section indicates that in a rational method of design only the reliable sources of extra strength should be taken into account and that the contribution to strength from such sources should be used in estimating the capacity rather in reducing the expected demand. An argument for reducing the design load may perhaps be justified in accounting for the reserve strength attributable to redistribution of internal forces. Even there a more logical procedure would be to use the concept of limit design in which the load corresponding to the development of a mechanism or the attainment of a specified limit on drift is explicitly calculated. Limit design, however, involves considerable complexity and may not be a practical procedure for the design of normal structures. Assessment of the difference between limit strength and the strength at first yield is thus of considerable interest.

As a simple example of the reserve strength attributable to redistribution of internal forces, consider the single storey frame shown in Fig. 1a. The frame is designed so as to remain elastic under the application of a lateral load P_y and a gravity load w per unit length on the beam. The columns are designed to have a strength that is higher than that of the beam, so that plastic hinges form only in the beam. It is also assumed that the moment-rotation relationship for a hinge is perfectly elasto-plastic. Let the frame be subjected to a lateral load P and let this load increase from zero as the gravity load remains constant at w. The relationship between the lateral force and the lateral displacement is plotted in Fig. 1b. For P less than P_y, the structure is elastic. When P reaches P_y, a plastic hinge is formed at the right hand end of the beam. The slope of the force-displacement relation becomes flatter. When P reaches P_u, a second plastic hinge is formed in the beam. Depending on the ratio of lateral to gravity load, this hinge may form either at the left hand end of the beam or along the span. Here we will assume that the hinge forms at the left hand end. The frame now becomes a mechanism and can not take any additional lateral load. The ratio $R_d = P_u/P_y$ represents the reserve strength due to redistribution.

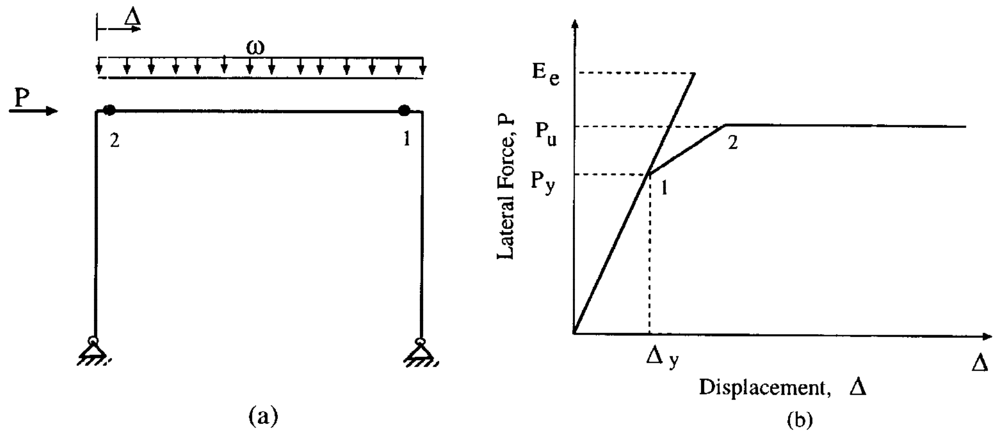

Figure 1: Load-displacement relationship for a single storey frame

Assume now that if the frame of Fig. 1 were to remain elastic, the design earthquake will induce in it a base shear E_e. Relying on the ductility of the frame, it is to be designed to have a base shear strength $E_u = E_e/R$. Thus the design requirement is

$$P_u \geq E_u \tag{4}$$

If R_d is known or can be determined, the design requirement can be stated in the alternative form

$$P_y \geq \frac{E_u}{R_d} = \frac{E_e}{RR_d} \tag{5}$$

The use of Eq. 5 offers the advantage that only an elastic analysis need to be carried out to determine the design forces. On the other hand, use of Eq. 4 requires a limit analysis.

ANALYTICAL STUDIES

In order to assess the range of reserve strength values associated with redistribution, a series of buildings is analyzed for a combination of gravity and earthquake forces. Figure 2 shows a plan view of the building. Buildings with a height of 2, 5, 7, 10, 15, 20, and 30 storeys are studied. The lateral load resistance is provided by moment-resisting frames of steel. Design for gravity and earthquake forces in the North-South direction is carried out according to the provisions of NBCC. The value of zonal velocity ratio, v, is assumed to be 0.2. The following load combinations are used

$$1.25D + 1.50L \tag{6a}$$

$$1.0D + 0.5L + 1.0E \tag{6b}$$

where D represents dead load, L is the live load, and E is the earthquake load.

The dead load is assumed to be 3.40 kN/m^2 and the live load is taken as 2.4 kN/m^2. A uniform reduction factor of 0.691 is applied to the live load for the design of both the beams and the columns.

In the design of low-rise buildings, if every frame in the N-S direction is assumed to be moment-resisting, the earthquake forces will be small and design will be governed by the combination of dead and live load. In such a situation it is not necessary to design every frame to be moment-resisting and some of the frames could be of simple construction. The number of moment-resisting frames is chosen so that the combination of gravity and earthquake forces starts becoming critical. Thus, for a two storey frame only 1 in every 3 frames is designed to be moment-resisting; in a five storey frame 2 out of 3 frames are moment-resisting.

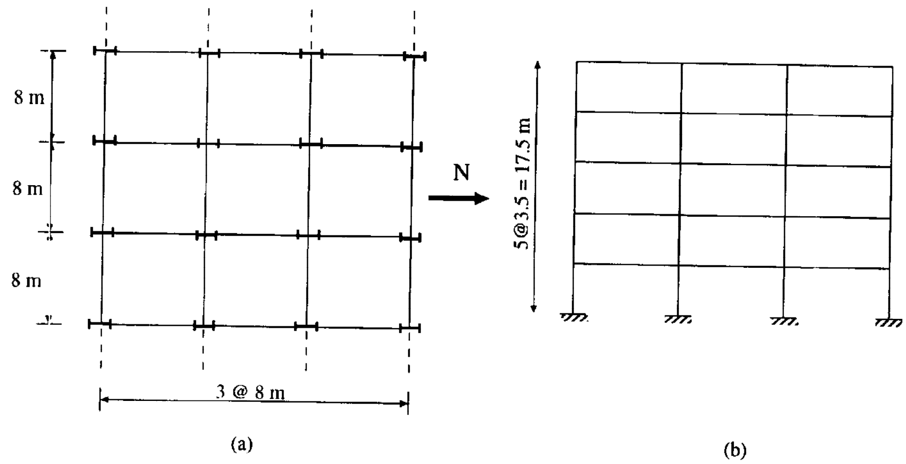

Figure 2: (a) Typical plan view of the building structure
(b) Elevation of a five storey frame

In designing the building frames a strong column weak beam principle is used so that in a nonlinear analysis plastic hinges do not form in the columns, except at the base. Also a drift limit of 0.005 the storey height under the specified earthquake forces is enforced. The column and beam sections required to resist the calculated design moments shears and axial forces are selected from the list of available sections given in a steel handbook. In order to discount the extra strength obtained by selection from a list of discrete member sizes, the values of the section strengths used in the subsequent analysis are assumed to be equal to the design moments, rather than the actual resisting moment of the section selected.

A typical frame of each building is now subjected to an incremental static push-over analysis for the gravity and earthquake forces tributary to it. The gravity loads are held constant at their full value implied in load combination given by Eq. 6b. The lateral earthquake forces are assumed to be distributed along the height according to the provisions of NBCC. The lateral forces are now increased in suitable increments until a mechanism forms, or the interstorey displacements are well past the design limit of 0.02 of the storey height. A nonlinear analysis strategy is used in which the plastic hinges are assumed to form at only the ends of the members. The moment-rotation relationship for a potential hinge is taken to be bilinear or elasto-plastic. The computer program used in the study is DRAIN-2DX.

In the push-over analysis the first set of plastic hinges forms at the design earthquake forces. If the frame under consideration is not subject to any gravity loads, these plastic hinges would be enough to create a mechanism, and there would be no reserve strength beyond the first yield. However, because the frames have been designed for a combination of gravity and earthquake forces and the beam and column sections are uniform along the member lengths, the first set of hinges is not enough to form a mechanism. The frame thus continues to resist additional lateral forces.

The complete base shear versus top floor lateral deflection for a 10 storey frame is shown in Fig. 3b. The base shear value at the point where any one of the inter-storey drifts exceeds 0.02 of the storey height is also shown. This may be taken as the ultimate strength of the frame. The ratio of the ultimate base shear to the shear at first yield is 1.75 in this case. The order in which the various plastic hinges are formed is indicated in Fig 3a. It should be noted that hinges form also at the bases of the first storey columns. This is to be expected. A few hinges form also at other locations in the columns, even though the sum of the strengths of columns meeting at a joint are designed to be larger than the sum of the strengths of beams meeting the same joint. For the ten storey frame shown in Fig. 3a, the column hinges other than those at the base form after the ultimate shear has been reached and hence do not influence the reserve strength value calculated. Figures 3a and 3b clearly show the reserve strength beyond first yield that can be attributed to redistribution.

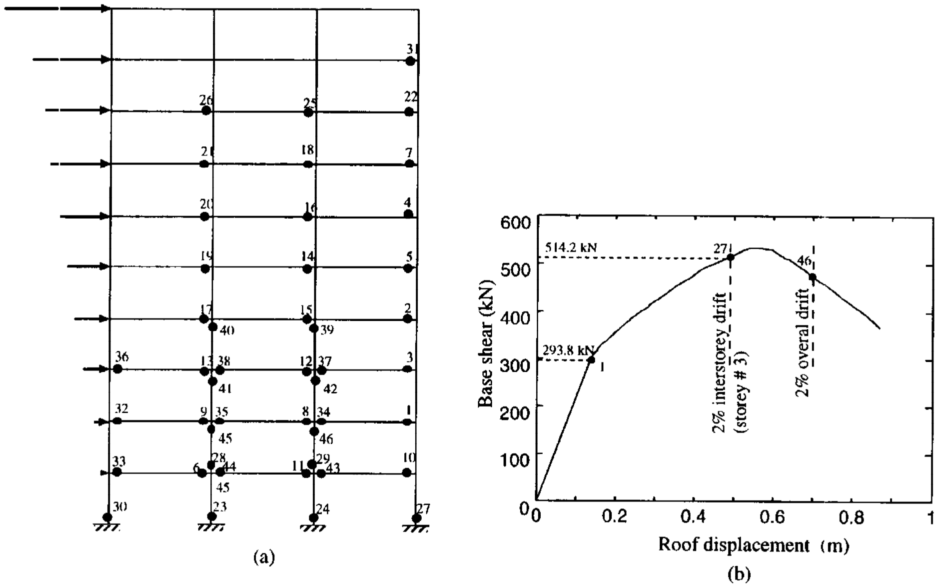

Figure 3: Results of push-over analysis for a 10 storey frame, P- Δ included, strain hardening modulus = 0; (a) sequence of formation of hinges; (b) base shear versus roof displacement

The calculated reserve strengths for all of the frames studied are shown in Fig. 4. The horizontal axis in the figure shows the number of storeys, or equivalently the period, taken to be equal to $0.1N$ as per the NBCC. The true calculated period is, in all cases, larger than this value. The ordinate of the graphs in Fig. 4 gives the ratio of the ultimate base shear strength P_u to the base shear at first yield P_y.

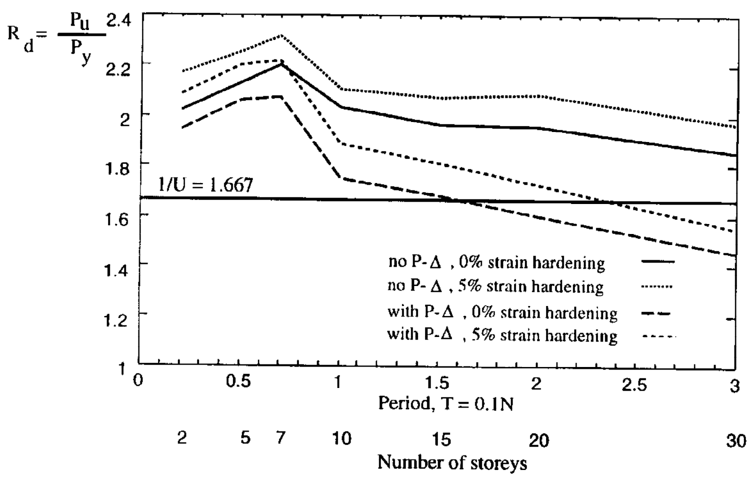

Figure 4: Reserve strength values for buildings of different heights

Four different curves are presented in Fig. 4, with and without $P-\Delta$ effect, and with and without strain hardening. The reserve strength becomes smaller when the $P-\Delta$ effect is taken into account. Similarly the strength is reduced when the effect of strain hardening is ignored and the moment-rotation relationship for plastic hinges is considered to be elasto-plastic.

In general, the reserve strength is higher for low rise buildings. However, the increase is not as significant as reported in the literature. This is so first because in the present study the reserve strength due only to redistribution is accounted for, and second because not all of the frames are designed to be effective in resisting the lateral forces.

For purpose of comparison, the reserve strength factor used in NBCC, $1/U = 1.67$ is also shown in Fig. 4. For the type of frames studied, this factor may be considered to provide a reasonable estimate of the reserve strength attributable to redistribution, except in the case of very tall buildings without any strain-hardening in the moment-rotation relationship.

As stated earlier, the magnitude of reserve strength depends on the relative values of the gravity and earthquake loads. The reserve strength decreases with an increase in the ratio of the earthquake base shear to the total gravity load. In the limit, if the lateral force resisting frame carries no gravity loads, the reserve strength is zero. The variation of reserve strength of a ten-storey frame with the ratio of earthquake base shear V to the total gravity load W used in the design load combination, Eq. 6b, is shown in Fig. 5. As would be expected, the reserve strength decreases with an increase in the ratio V/W.

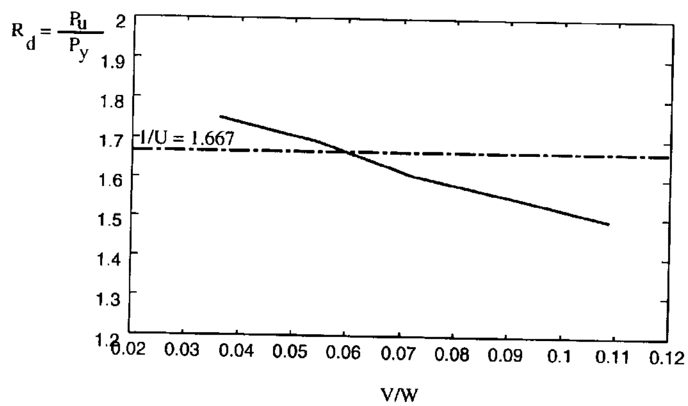

Figure 5: Effect of ratio of earthqauke force to gravity force on the reserve strength of a 10 storey building, P-Δ effect included, 0% strain hardening

SUMMARY AND CONCLUSIONS

Experience of satisfactory performance during past earthquakes shows that buildings designed according to current seismic codes must possess considerable reserve strength. Some of the sources of such overstrength are uncertain, others, although reliable, are best taken into account in design in assessing the capacity of the structure rather than in scaling down the design forces. One exception is the reserve strength owing to the redistribution of internal forces. Scaling down of design forces to account for this source of reserve strength simplifies the analysis and is therefore useful. For buildings with moment-resisting steel frames, reserve strength attributable to redistribution is of the order of 67%, but may vary considerably depending on the methodology used in design and on the ratio of earthquake to gravity loads affecting design. Additional studies are needed to assess the reserve strength in other structural systems.

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