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EXPLICIT INCORPORATION OF ELEMENT AND STRUCTURE OVERSTRENGTH IN THE DESIGN PROCESS

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ABSTRACT

Sources of overstrength of reinforced concrete structures subjected to seismic actions are considered. The most common reasons for overstrength are steel and concrete with strengths higher than specified, member sizes and quantities of steel reinforcement being larger than necessary, the use of strength reduction or material factors in design, other load combinations, and the participation of non-structural elements. The inclusion of the effects of overstrength in the seismic design process is discussed. Taking account of overstrength is a necessary part of the capacity design procedure used in New Zealand to ensure that the preferred mode of post-elastic deformation occurs during a severe earthquake and that the level of seismic design actions used is appropriate.

KEYWORDS

Brittle failure; capacity design; concrete confinement; design beam shear forces; design column bending moments; design column shear forces; design seismic forces; moment resisting frames; reinforced concrete; steel characteristics; structure overstrength.

SOURCES OF STRUCTURE OVERSTRENGTH

It has been recognised that most well constructed reinforced concrete structures as built have significant overstrength. That is, the actual lateral load strength of the structure is greater than the ultimate lateral seismic forces used in the design. The overstrength of the structure is due to many factors including:

- Steel and concrete strengths greater than specified.
- Use of a strength reduction factors or material factors in design.
- Section sizes larger than assumed; for example, due to the participation of slab reinforcement in beam flexural behaviour.
- Effects of member deformations at large displacements; for example, axial compression in beams due to lateral restraint and the effect on adjacent structural elements of the elongation of structural walls due to plastic hinge rotation.
- Additional reinforcement placed for construction purposes or to satisfy minimum reinforcement requirements or to satisfy available bar sizes, and unaccounted for in the design calculations.
- More critical loading cases for the design of some sections for gravity or wind loads.

- Moment redistribution after yielding greater than assumed in design.
- Participation of non-structural elements.
- Overestimation of structural stiffness leading to high design seismic forces.

Some of these contributions to overstrength of structures can be reasonably accurately predicted. For example, the magnitude of steel and concrete overstrength can be estimated. The effect of strength reduction factors or material factors can be calculated. For instance, if the strength reduction factor for flexure is $\phi = 0.85$, the nominal flexural strength is equal to the design flexural strength multiplied by $1/\phi = 1/0.85 = 1.18$. Slab participation in the flexural behaviour of beams is important in the negative moment regions where slab reinforcement can add significantly to the flexure strength. As a result of tests by Cheung et al, 1991 the New Zealand concrete design standard (Standards New Zealand, 1995) recommends that the slab width within which effectively anchored longitudinal slab reinforcement shall be considered to contribute to the negative moment flexural strength of the beam, in addition to those bars placed within the web width of the beam, for interior spans shall be defined as one-quarter of the span of the beam extending each side of the beam from the centre of the beam section but not greater than the width to the centre of the slab panels each side.

The other contributions to overstrength listed above are more difficult to quantify. However, it is apparent that the real lateral load strength of reinforced concrete structures could easily be 50 to 100% greater than the ultimate lateral seismic force used in the design.

ACCOUNT TAKEN OF STRUCTURE OVERSTRENGTH WHEN DETERMINING THE DESIGN SEISMIC FORCES

The equivalent static design seismic forces at the ultimate limit state are generally found by factoring down the accelerations found from elastic response spectra to account for the reduction in elastic response inertia forces possible due to structure ductility, and often also to account for structure overstrength. For example, the Uniform Building Code (International Conference of Building Officials, 1991) obtains the design seismic acceleration by dividing the elastic response acceleration by a modification factor R_w (for example, $R_w = 12$ for special moment resisting frames of steel or concrete) which includes allowance for both ductility and overstrength. The New Zealand loadings standard (Standards New Zealand, 1992) determines the design seismic acceleration from uniform elastic seismic hazard acceleration spectra using a structure displacement ductility factor μ (for example, $\mu = 6$ for ductile moment resisting concrete frames) and a structural performance factor S_p generally taken as $S_p = 0.67$. The structural performance factor is a numerical assessment of the ability of a building to survive cyclic displacements and includes the effect of overstrength. Thus the S_p factor assumes an overstrength which may approach 1.5 times the nominal strength of the building.

It is evident that the effect of structure overstrength in the determination of the design seismic forces has been included in a very subjective way in design standards for loading. The degree of precision used to take advantage of the likely structural overstrength by reducing the design seismic forces does not match the more precise calculations conducted for strength of members and connections to resist those design forces. For example, in reality the actual overstrength will vary from structure to structure even within groups of particular types of structure.

POSSIBLE ADVERSE EFFECTS OF OVERSTRENGTH

The effects of overstrength are not always beneficial to structural behaviour. For instance, flexural overstrength of the beams of moment resisting frames may result in brittle column sidesway mechanisms (soft storey failure). Also, flexural overstrength of members leads to increased shear forces when plastic hinges form which may result in brittle shear failure. Non-structural elements, for example brick infills, could lead to shear failure due to short column effects or to soft storey failures. The overstrength of reinforcing bars may lead to bond deterioration. Hence the effects of possible overstrength need to be carefully considered in design to ensure that undesirable failure mechanisms do not occur as a result of a change of failure hierarchy.

THE CAPACITY DESIGN PROCEDURE

In New Zealand since the 1970s a procedure of seismic design known as capacity design has been used to ensure that the most appropriate mechanism of post-elastic deformation occurs in the event of a major earthquake. In the capacity design of structures, appropriate regions of the primary lateral earthquake force resisting system are chosen and suitably designed and detailed for adequate strength and ductility. All other regions of the structural system, and other possible failure modes, are then provided with sufficient strength to ensure that the chosen means for achieving ductility can be maintained throughout the post-elastic deformations that may occur during a severe earthquake.

For example, for moment resisting frames of buildings structure ductility is best achieved by the formation of beam sidesway mechanisms (see Fig. 1c) since with proper design the plastic hinges in the beams and at the column beams can be designed to be adequately ductile. Thus strong column-weak beam design is advocated. When designing the plastic hinge regions for adequate shear strength, and other regions of the structure for adequate flexural and shear strength, the maximum likely actions likely to be imposed on those regions should be calculated including the effects of flexural overstrength in the chosen yielding regions. Hence soft storey failures involving excessive plastic rotation demand in the columns of one storey (see Fig. 1b) can be avoided.

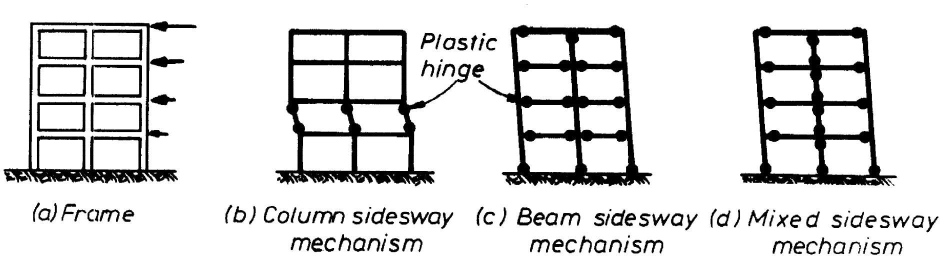


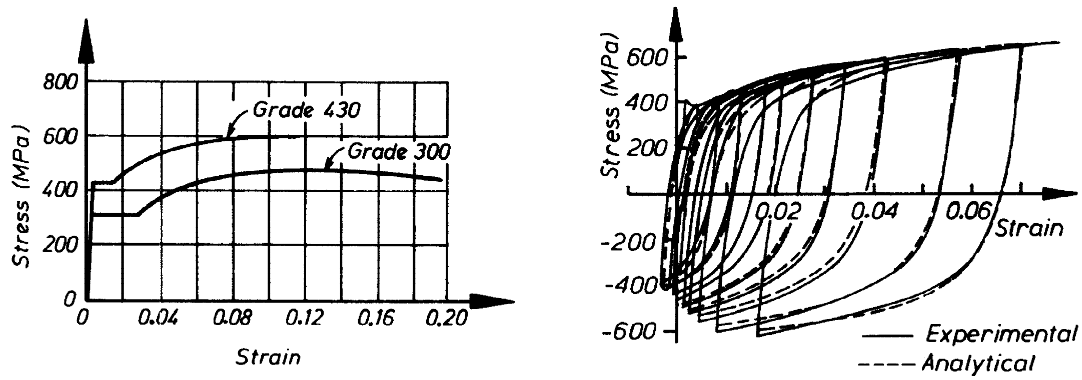
Fig. 1 Some mechanisms of post-elastic deformation of moment resisting frames during severe seismic loading

The New Zealand concrete design standard has two exceptions to the requirement of strong column-weak beam design of moment resisting frames. The first exception is that for ductile frames of one or two storey buildings, or in the top storey of a multistorey building, column sidesway mechanisms are permitted (that is, a strong beam-weak column approach), since the curvature ductility demand at the plastic hinges in the columns in such cases of low frames is not high and can be provided by proper detailing. The second exception is for buildings in areas of low seismicity and/or where beams have long spans where the gravity load considerations may govern and make strong column-weak beam design impracticable. In such case, interior columns of ductile frames three storeys or higher may be designed to develop plastic hinges in any storey simultaneously at the top and bottom ends, providing that the exterior columns remain in the elastic range (see the mixed sidesway mechanism in Fig. 1d). Such frames are required to be designed for a higher design seismic force than for ductile frames with beam sidesway mechanisms if the total shear strength of the columns remaining in the elastic range in a storey is less than one-half of the total shear strength of all columns in that storey.

OVERSTRENGTH DUE TO THE ACTUAL STRESS-STRAIN CHARACTERISTICS OF THE MATERIALS

Reinforcing Steel

The actual yield strength and stress-strain characteristics of the reinforcing steel in reinforced concrete structures can have a significant effect on the post-elastic behaviour of reinforced concrete structures during severe earthquakes. For example, Fig. 2a shows stress-strain curves measured for typical New Zealand manufactured steel reinforcing bars under monotonic loading. In practice the actual yield strength of the steel will normally exceed the yield strength f_y used in design. Also, in the plastic hinge regions of ductile reinforced concrete structures during a major earthquake the longitudinal reinforcement may reach strains in the order of 20 or more times the strain at first yield and a further increase in steel stress due to strain hardening may occur. The maximum likely flexural strength at the plastic hinges is referred to as the flexural overstrength.



(a) With Monotonic Loading

(b) With Cyclic Loading Mainly in the Tensile Range of Strain

Fig. 2 Typical stress-strain curves for reinforcing steel

It is evident that the properties of the reinforcing steel to be used in seismic design should be based on thorough statistical analyses of the stress-strain properties, to determine the lower and upper bounds of the flexural strength of reinforced concrete elements. As an example, Andriono and Park, 1986 have conducted statistical studies of samples of 5 years of production of the grades of reinforcing steel manufactured in New Zealand to establish the 5 percentile value of the yield strength (below which not more than 5% of test results fall), the 95% percentile value of the yield strength (above which not more than 5% of test results fall), the ultimate strength, elastic modulus, strain hardening modulus, strain at commencement of strain hardening, and ultimate strain. The statistical results of the stress-strain properties of the steel reinforcement were used in moment-curvature analyses of reinforced concrete beam sections, incorporating the Monte Carlo simulation technique, to assess the beam flexural overstrength factors for use in seismic design taking into account the likely variation in the steel and concrete properties.

The 5 percentile value of the yield strength, referred to in New Zealand as the lower characteristic yield strength, is considered to be the appropriate value for use for determining the required areas of reinforcement in the strength design of members. The New Zealand standard for reinforcing steel (Standards Association of New Zealand, 1989) also requires that the 95 percentile value of yield strength, referred to as the upper characteristic yield strength, does not exceed a specified value, in order to ensure that the overstrength of the steel is not too great. In addition, the absolute minimum and maximum values for the yield strength are also be specified and the yield strengths are not permitted to lie outside the range of those minimum and maximum values. The specified values for the yield strengths for the two grades of reinforcing steel manufactured in New Zealand are:

Yield Strengths:	Minimum MPa	Lower Characteristic MPa	Upper Characteristic MPa	Maximum MPa
Grade 300	275	300	355	380
Grade 430	410	430	500	520

Note: ratio of upper to lower characteristic yield strengths = 1.17 on average.

The permitted range for the ratio of ultimate strength to yield strength is 1.15 to either 1.4 or 1.5.

It is also evident that the steel strain at the ultimate strength of the steel (that is, at the peak stress of the stress-strain curve) should be at least about 10%, to ensure high available curvature ductility factors in beams. It is of concern that reinforcing steels with much lower strains at the ultimate strength of the steel have been used in some countries in seismic regions.

Figure 2b shows stress-strain curves measured for reinforcing steel under cyclic loading. The "rounding" of the stress-strain curve during loading reversals in the post-elastic range is due to the Bauschinger effect. It is of interest that in beams the plastic elongation occurs mainly in the tension direction, since plastic elongation in compression is reduced by the concrete in compression when cracks there close, and the monotonic stress-strain curve gives a good envelop for the cyclic stress-strain curves.

It was confirmed in the study by Andriono and Park, 1986 that the flexural overstrength at plastic hinges in concrete beams due to overstrength caused by New Zealand manufactured reinforcing steel should be taken as $1.25M_n$, where M_n is the nominal flexural strength of the section calculated using the lower characteristic yield strength of the steel. This 25% increase in M_n takes into account the probability of the actual steel yield strength being greater than the lower characteristic value (approximately a 17% allowance is assumed, as given by the ratio of the upper to lower characteristic yield strengths of 1.17) and the steel strength increase above the yield strength due to strain hardening at high strains (approximately an 8% allowance).

It is evident that it is very important for statistical information on the stress-strain properties of reinforcing steel used in seismic regions to be available. A proper capacity design cannot be undertaken without knowledge of the likely variations of the steel properties to obtain overstrength factors, and adequate ductility of plastic hinges of members cannot be ensured if the steel is brittle.

Concrete Confinement

When detailing reinforced concrete columns for ductility the compressed concrete should be confined by appropriate arrangements of transverse and vertical reinforcement in order to increase the available ultimate compressive concrete strain, and hence to improve the ductility of the column. A consequence of the confinement is also an increase in the compressive strength of the concrete to above that for unconfined concrete, resulting in an increase in the column flexural strength.

The enhancement in flexural strength of reinforced concrete columns due to the increase in the concrete compressive strength as a result of confinement can be high. For example Fig. 3 shows the theoretical enhancement in flexural strength found by cyclic moment-curvature analysis of a circular reinforced concrete column with $p_t m = 0.1$, $f_y = 275$ MPa, f'_c anywhere in the range 20 to 40 MPa, and various lateral confining pressures, where $p_t = A_{st}/A_g$, A_{st} = total area of longitudinal reinforcement, A_g = gross area of column section, $m = f_y/0.85f'_c$, f_y = yield strength of the longitudinal reinforcement and f'_c = compressive cylinder strength (unconfined) of the concrete. In Fig. 3, N^* = axial compressive load on the column, f_ℓ = effective lateral confining stress acting on the concrete core due to the transverse reinforcement, M_{max} = maximum calculated flexural strength taking into account the increase in concrete strength and ductility due to confinement using the model of Mander et al, 1988 but neglecting the effect of steel overstrength, and M_{code} = flexural strength calculated using the conventional approach of the concrete design codes assuming a rectangular concrete compressive stress block with a mean stress of $0.85 f'_c$, an extreme fibre concrete compressive strain of 0.003, the measured material strengths f'_c and f_y, and a strength reduction factor $\phi = 1$. For the calculation of the maximum flexural strength M_{max} including the effect of both concrete confinement and steel overstrength an additional factor would need to be applied to the M_{max} in Fig. 3. That additional factor for steel overstrength is more or less independent of the axial load ratio and can be taken as approximately 1.25. The enhancement of the flexural strength of confined columns has also been demonstrated experimentally. In fact laboratory tests have shown greater enhancements than in Fig. 3 due to the additional confining effect of the beam or foundation adjacent to the critical section of the column (Watson and Park, 1994).

The flexural strength enhancements shown in Fig. 3 are strongly dependent on the axial load ratio. This follows because a high axial compressive load means a large neutral axis depth, which in turn means that the flexural strength of the column is more dependent on the contribution of the concrete compressive stress distribution. According to the New Zealand concrete design standard (Standards New Zealand, 1995) the quantity of confining reinforcement required to achieve a particular curvature ductility factor increases with the level of axial load. Hence for a high axial load the effect of the increase of the strength of the concrete on the flexural strength due to confinement is greater.

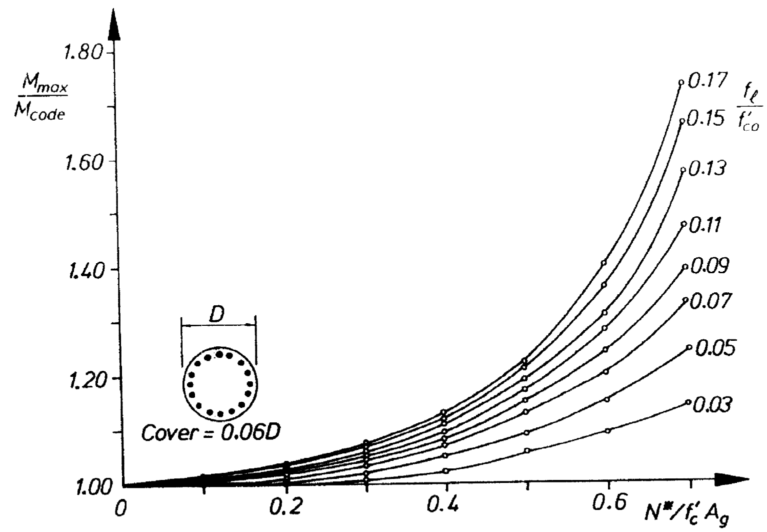


Fig. 3 Theoretical enhancement in the flexural strength of a circular reinforced concrete column due to concrete confinement (Zahn et al, 1986).

Economies would result from design based on the real flexural strength of confined columns, since a reduction in the quantity of longitudinal reinforcement necessary in columns would result.

CAPACITY DESIGN ACTIONS USED IN NEW ZEALAND FOR MOMENT RESISTING FRAMES WHEN WEAK COLUMN-STRONG BEAM ACTION IS SOUGHT

Design Bending Moments and Axial Loads in Columns of Ductile Frames

The capacity design procedure for ductile moment resisting frames is used to prevent, as far as possible, column sidesway mechanisms occurring during a severe earthquake. To achieve this aim the column bending moments obtained from analysis for equivalent static loading are amplified to take into account the effects of the flexural overstrength of the plastic hinges in beams, higher modes of vibration of the structure, and seismic forces acting along both principal axes of the building simultaneously. The New Zealand concrete design standard (Standards New Zealand, 1995) recommends that for ductile frames the design uniaxial bending moments for the critical sections of a column, considered separately in each of the two principal directions of the structure, be taken as:

$$M^*_{col} = \phi_o \omega M^*_e - 0.3 h_b V^*_{col} \tag{1}$$

where M^*_e = column moment at the centre of the beam-column joint derived by elastic structural analysis for the equivalent static design seismic forces at the ultimate limit state; ϕ_o = ratio of overstrength flexural capacity of the beams as detailed to the dependable flexural strength required by the standard $\geq 1.25/0.85 = 1.47$, where 1.25 = overstrength applied to the nominal flexural strength and 0.85 = strength reduction factor; ω = factor allowing for higher mode and bidirectional seismic load effects, given for one-way frames as $\omega = 0.6T + 0.85$ but not less than 1.3 or more than 1.8, and for two-way frames as $\omega = 0.5T + 1.1$ but not less than 1.5 or more than 1.9, where T = fundamental period of vibration of the structure; h_b = beam depth; and V^*_{col} = column shear force. In Eq. 1 the critical column section is assumed to be at the top or bottom of the beams and accordingly the centreline column moment $\phi_o \omega M^*_e$ is reduced by $0.3h_b V^*_{col}$, which is based on an estimated gradient of the column moment diagram. The recommended amplification of column moments by this procedure can be significant, the combined factor $\phi_o \omega$ being at least 1.91. Note that for two-way frames the columns are designed for uniaxial bending only, since ω includes some moment amplification for the effect of biaxial bending. The values of ω are based on dynamic analyses and judgement (Paulay, 1977).

It is to be noted that the value of ω given above varies between 1.3 and 1.9, whereas, Pinto, Colangelo and Giannini, 1995, as a result of recent dynamic analyses of one-way 4 and 8 storey moment resisting symmetrical frames, have concluded that $\omega = 1.35$ may be high enough.

The design axial loads in columns N^*_{col} to be used with M^*_{col} given by Eq. 1 for the design of the column sections should be, according to the New Zealand concrete design standard, derived from the shear forces applied at the column faces by the gravity loads from the beams and the moment induced shears from the beam plastic hinge moments in both directions acting at flexural overstrength. A reduction in the moment induced shears is allowed, to take into account the probability that not all beam plastic hinges have reached their overstrength simultaneously up the height of the frame. The maximum value for N^*_{col} is not permitted to exceed 70% of the concentric load strength of the column.

Design Shear Force in Beams of Ductile Frames

The design shear forces in beams of ductile moment resisting frames can be determined by the capacity design procedure for when the flexural overstrength is reached at the plastic hinge locations within the span and the gravity load is present.

Design Shear Forces in Columns

The New Zealand concrete design standard (Standards New Zealand, 1995) recommends that for ductile frames the design shear forces in columns up the height of the building, acting separately in each of the two principal directions of the structure, as determined by the capacity design procedure, be taken for the columns of a one-way frame as $V^*_{col} = 1.3\phi_o V_e$ and for columns of a two-way frame as $V^*_{col} = 1.6\phi_o V_e$, where ϕ_o = beam overstrength factor defined as for Eq. 1 and V^*_e = column shear force derived for the equivalent static design seismic forces at the ultimate limit state. These forces were estimated from probable critical moment gradients along columns (Paulay, 1977). The larger value for two-way frames is to include the effect of concurrent seismic loading acting along both principal axes of the building simultaneously. For the column of the first storey the design shear forces are taken as $V^*_{col} = (M_{o,\,col,\,bottom} + M_{o,\,col,\,top})/\ell_n$, where $M_{o,\,col,\,bottom}$ and $M_{o,\,col,\,top}$ are the flexural overstrength capacities of the bottom and top critical plastic hinge sections of the columns, respectively, and ℓ_n = clear length of column between beams. The flexural overstrength column capacities should include the effects of both the steel and the concrete overstrengths, resulting in multiplying the nominal flexural strength by a factor in the range 1.25 to 2.0 depending on the axial load ratio $N^*/f'_c A_g$ and the actual reinforcement contents.

Design Shear Forces in Beam-Column Joints

The design shear forces in beam-column joints should be calculated using the overstrength steel forces and the design shear forces for the members at overstrength. The calculation of the design horizontal shear force involves the determination of the net horizontal force above or below a horizontal plane passing through the joint core. Similarly, the design vertical shear force can be calculated from the net vertical force to one side or other of a vertical plane passing through the joint core.

Extent of Potential Plastic Hinge Regions of Columns

It should also be noted that the enhanced flexural strength of confined sections of columns means that flexural failure could occur in less heavily confined regions away from the section of maximum bending moment. Hence as demonstrated by Watson and Park, 1994, reinforced concrete columns which carry relatively large axial compressive loads need to have a longer fully confined length than columns which carry relatively light axial compressive loads. The fully confined potential plastic hinge region specified by the New Zealand concrete design standard is generally in the range of one to three times the larger column cross section dimension, increasing with the axial load ratio $N^*/f'_c A_g$.

CONCLUSIONS

There are many sources of overstrength of reinforced concrete structures, some of which are difficult to quantify. Some design standards recommend design seismic forces which are reduced assuming that structures will actually be stronger than designed. Special note should be taken of the effects of member overstrength when determining the relative strengths of members to be used in design to ensure the formation of the preferred plastic mechanism during a severe earthquake.

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