A STATIC FORCE-BASED PROCEDURE FOR THE SEISMIC ASSESSMENT OF EXISTING REINFORCED CONCRETE MOMENT RESISTING FRAMES

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This paper is based on a presentation made by the author at the NZNSEE Annual Conference in New Plymouth in 1996.

SUMMARY

A force-based seismic assessment procedure for existing reinforced concrete moment resisting frames is discussed. The assessment procedure is based on determining the probable strength and ductility of the critical mechanism of post-elastic deformation of the frame. Account is taken of the likely seismic behaviour of reinforced concrete beams, columns and beam-column joints with substandard reinforcement details typical of structures designed before the 1970s, as determined by the results of experimental testing and analytical studies. The assessment aims at determining the available lateral load strength and structural (displacement) ductility factor of the frames so that the designer can determine the likely seismic performance of the structure by referring to acceleration response spectra for design earthquake forces for various levels of structural ductility factor.

INTRODUCTION

The major advances made in the seismic design of reinforced concrete structures through the years have been the outcome of a better understanding of the nonlinear dynamic response of structures, the mechanisms of post-elastic deformation of structures, and the methods for detailing reinforcement in reinforced concrete structures so as to achieve the ductile behaviour necessary to survive severe earthquakes.

A significant step forward in the design of multi-storey reinforced concrete buildings for earthquake motions was the publication in the United States of the pioneering book of that title by Blume, Newmark and Corning in 1961 (1).

In New Zealand a major step forward has been the introduction of seismic design procedure referred to as capacity design. The basis of this procedure was first described in 1969(2) in a paper by Hollings and further developed in 1975 in a book by Park and Paulay (3). In the capacity design procedure the designer chooses the most appropriate mechanism for the structure to achieve adequate ductility during a major earthquake, normally by flexural yielding occurring at selected plastic hinge positions. The plastic hinge regions are designed for adequate flexural strength and ductility. All other regions of the structure are then made adequately strong in flexure, and the shear strengths of the whole structure made adequately large, to ensure that the post-elastic deformations occur only at the selected plastic hinge regions. Significant developments in ductile detailing have also occurred in New Zealand since the 1960s. The most recent seismic standards are the standard for general structural design and design loadings for buildings NZS 4203:1992(4) and the standard for concrete design NZS 3101:1995(5).

SEISMIC ASSESSMENT

These developments in seismic design have brought the realization that many reinforced concrete structures designed before the mid-1970s may be deficient according to the seismic requirements of current codes. As a result there has been increasing emphasis in recent years on the assessment and retrofit of buildings so as to improve their seismic performance.

The need for the assessment of old reinforced concrete building structures, and to retrofit if necessary, has been emphasized by the damage caused by major earthquakes. The most recent example was the Hyogo-ken Nanbu earthquake which badly damaged many buildings in Kobe, Japan on 17 January 1995(6). In that earthquake damage to reinforced concrete buildings was much more severe for buildings built before the current Japanese seismic code came into effect in 1981. Most buildings built after 1981 suffered only minor damage.

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The structural deficiencies of many existing reinforced concrete structures designed to early codes in New Zealand and other countries are generally not just a result of inadequate strength. For example, longitudinal reinforcement in many existing structures results in a lateral load strength which approaches or exceeds that required by current standards for ductile structures. The poor structural response during severe earthquakes is normally due to a lack of a capacity design approach to ensure the formation of an appropriate mechanism of deformation and/or to poor detailing of reinforcement, which means that the available ductility of the structure may be inadequate to withstand the earthquake without collapse.

There has been increased activity in many countries in the seismic assessment of buildings and the retrofitting where necessary to improve seismic performance. The decision to retrofit has normally been made by comparing the details of the as-built structure with the requirements of current seismic codes. The emphasis in these retrofit projects has been to bring structures up to near current code requirements by the provision of additional strength and/or ductility. However, the evidence of tests and analysis of existing structures, and of observed earthquake damage, is that not all structures designed before the current generation of codes will respond poorly to severe earthquakes, even when according to current standards the detailing of reinforcement in some regions is substandard.

A realistic assessment procedure has been suggested by Priestley and Calvi(7), which gets away from the check-list type approach and considers the overall performance of the structure. The suggested procedure is based on determining the available static lateral load strength and ductility of the critical post-elastic mechanism of deformation of the structure. Once the available lateral load strength and ductility of the structure has been established, reference to the current code response spectra for earthquake forces for various levels of structural displacement ductility factor then enables the designer to assess the likely seismic performance of the structure.

This approach can be considered to be force-based since it assesses the ductility required for a given lateral load strength from inelastic acceleration spectra.

More recently Priestley(8) has extended this approach to a static displacement-based seismic assessment procedure. Priestley claims that the weakness of the force-based approach is the assumed relation between the elastic and the inelastic acceleration response spectra for various structural (displacement) ductility factor levels, and the emphasis on strength. In the static displacement-based approach the comparison of demand and capacity is made in terms of displacements found from displacement response spectra for different levels of equivalent viscous damping(8). There is no doubt that the displacement based approach is more logical.

However, since current seismic codes recommend seismic design in terms of design seismic forces and the associated ductility demand, most designers will wish to use a force-based approach for seismic assessment for the time being at least.
This paper outlines a static force-based procedure for assessing the likely seismic performance of existing reinforced concrete moment resisting frames using recent analytical and experimental evidence of the behaviour of elements and joints subjected to simulated seismic loading.

It is to be noted that time-history nonlinear dynamic analysis can also be used to estimate the ductility demand and the displacement response of an existing structure to major earthquake shaking. The force-deformation hysteresis loops used in the analysis need to realistically model any strength and stiffness degradation, and pinching, of the critical regions of the structure.

This paper considers only the more simple static force-based procedure.

PROBLEM AREAS OF MOMENT RESISTING FRAMES OF BUILDING STRUCTURES

Analyses of existing moment resisting frames typical of early reinforced concrete building structures, laboratory testing of elements and subassemblages containing typical early detailing, and observations of damage caused in recent earthquakes (for example, (6-12)), have indicated that the major problem areas are:

(a) Inadequate flexural strength of members, typically columns, due to insufficient longitudinal reinforcement.

(b) Inadequate anchorage of longitudinal reinforcement due to poor anchorage details.

(c) Inadequate ductility and shear strength of potential plastic hinge regions of beams and columns due to insufficient transverse reinforcement.

(d) Inadequate anchorage of transverse reinforcement due to poor anchorage details.

(e) Inadequate shear strength of beam-column joints due to insufficient transverse reinforcement.

(f) Inadequate strength of footings and/or piles and their connections.

(g) Uncertain behaviour of the structure as a result of the presence of non-structural elements (typically curtain walls, infill walls and partitions) which can significantly alter the structural behaviour of the frame.

The static force-based procedure described in the following sections considers problem areas (a) to (e) above.

A STATIC FORCE-BASED ASSESSMENT PROCEDURE

The suggested steps of a static force-based seismic assessment procedure of an existing moment resisting frame are:

Step 1: Establish the probable strengths of the concrete and reinforcing steel.

Step 2: Estimate the probable flexural and shear strengths of the critical sections of the beams and columns and of the beam-column joints assuming that no degradation of strength occurs due to cyclic lateral loading in the post-elastic range.

Step 3: Determine the post-elastic mechanism of deformation of the frame that is likely to occur during lateral seismic loading and the associated probable lateral seismic force capacity of the frame V_u at the ultimate limit state.

Step 4: Estimate the probable seismic coefficient $C_h(T,\mu)$ corresponding to the probable lateral force capacity of the frame V_u found in Step 3 from:

$$C_h(T,\mu) = \frac{V_u}{W_t} \quad (1)$$

where W_t = weight of the structure due to the dead load and the live load considered to be present during the earthquake.

Step 5: Estimate the fundamental period of vibration of the structure responding in the elastic range, T_1.

Step 6: Estimate the required structural (displacement) ductility factor μ for the estimated $C_h(T,\mu)$ and T_1 using the appropriate inelastic seismic acceleration spectra.

Step 7: Estimate whether the plastic hinges have the available ductility to match the required structural (displacement) ductility factor μ. The frame will require retrofitting if the rotation capacity of the plastic hinges is inadequate.

Step 8: Estimate the degradation in the shear and bond strength of the members and the beam-column joints during cyclic deformations to the imposed ϕ_u/ϕ_y in the plastic hinge regions, where ϕ_u = imposed maximum curvature at the ultimate limit state and ϕ_y = curvature at first yield in the plastic hinge region. Check whether any degradation in shear and bond strength will cause failure of the frame. If it does not, then the assessment apart from Step 9 is complete. If it does, the frame will require retrofitting.

Step 9: Estimate the interstorey drift and decide whether it is tolerable.

The procedure for carrying out these nine steps is discussed in more detail in the following sections.

STEP 1 : PROBABLE STRENGTH OF CONCRETE AND STEEL

In the assessment of an existing structure, realistic values for the material strengths should be used to obtain the best estimate of the probable strength of the members. It follows that the use of the nominal material strengths, that were specified in the original design is inappropriate. Ideally, samples of concrete and steel should be taken from the structure to measure typical strengths.

Inevitably some variation of strength throughout the structure will be found. It is suggested that the 5 percentile values of the measured strengths should be used for the probable concrete compressive cylinder strength and the steel yield strength (see Figure 1), and they will normally be somewhat higher than the strengths originally specified.

The actual compressive cylinder strengths of old concrete are likely to considerably exceed the nominal value as a result of conservative mix design, age and the less finely ground cement particles. Recent tests on the concrete of 30 year old bridges in California consistently showed compressive strengths approximately twice the nominal strength (8). Concrete from the columns of the Thorndon overbridge in Wellington has a measured compressive strength about 30 years after construction of about 2.3 times the specified value of 27.5 MPa. Similarly, concrete from collapsed columns of the elevated Hanshin Expressway in Kobe, Japan after the January 1995 earthquake has a measured compressive strength almost 30 years after construction of about 1.8 times the specified value of 27.5 MPa.

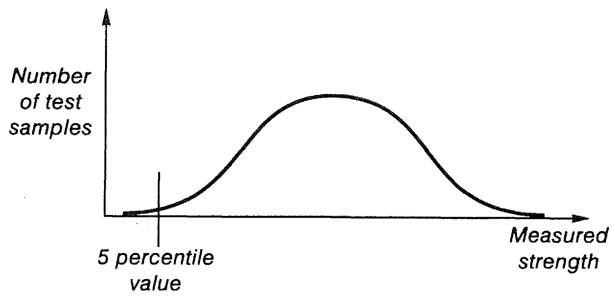


FIGURE 1 ***5 Percentile Value of Strength of Test Samples of Reinforcing Steel or Concrete***

A value of 1.5 times the specified compressive cylinder strength could be used to estimate the probable compressive strength of "old" concrete in assessment. Wherever practicable, cores should be taken from the structure to more accurately assess typical strengths. The quality of the concrete should also be inspected since if compaction in the critical regions was poor, a lower concrete compressive strength may need to be assumed.

The specified yield strength of reinforcing steel used in New Zealand has varied through the years. Many existing reinforced concrete structures were constructed using structural grade reinforcing steel with a minimum yield strength of about 227 MPa (33,000 psi); for example, as specified in NZS 1693:1962(13). Subsequently the minimum yield strength was increased to 275 MPa in the 1968 amendment of NZS 1693 and in NZS 3402P:1973(14), to more accurately reflect the minimum yield strength. A high yield steel with minimum yield strength of 414 MPa was also available in 1964 (15) and subsequent years. Chapman (16) reports that it has been found by site sampling and testing that in structures built in New Zealand during the 1930 to 1970 period the structural grade reinforcement is likely to possess a lower characteristic (5 percentile) yield strength which is 15 to 20% greater than the specified value. Reinforcing steel from the pile caps of the Thorndon overbridge in Wellington constructed in the 1960s has a measured lower characteristic yield strength of 286 MPa.

In the absence of other information, a probable yield strength of about 280 MPa can be used in the assessment of structures reinforced by structural grade reinforcement of the 1930-1970 period. Whenever practicable, samples of steel from the structure should be tested to obtain a better estimation of the probable yield strength of the reinforcement.

A further consideration is whether the longitudinal reinforcement is from deformed or plain round bars. Plain round bars were used in earlier years (before the mid-1960s in New Zealand). The development length needed for plain round bars is at least twice that for deformed bars (5). Also, during cyclic loading, the bond degradation for plain round bars is much more significant than for deformed bars. Hence old structures reinforced by plain round longitudinal bars will show a greater reduction in stiffness during cyclic loading.

STEP 2 : PROBABLE FLEXURAL AND SHEAR STRENGTHS

The probable flexural strength of members should be calculated using the probable material strengths and standard theory for flexural strength. The axial forces present in columns at the ultimate limit state need to be estimated in order to calculate the probable flexural strengths of the columns. A strength reduction factor $\phi = 1.0$ may be assumed for the flexural strength calculations for the beams and columns since the probable properties of the members as built are used.

The probable shear strength of members and beam-column joints should be calculated using the probable material strengths and theory for the shear strength of members and beam-column joints not undergoing cyclic deformations in the post-elastic range. The effect of the degradation of shear strength due to post-elastic cyclic deformations is considered in Step 8. A strength reduction factor $\phi = 0.85$ could be used for these shear strength calculations since, although the probable properties of the members and joints as built are used, the theory is less exact and shear failures can be catastrophic.

Shear Strength of Beams

The probable shear strength of beams with rectangular stirrups or hoops according to the non-seismic provisions of NZS 3101:1995(5) is given by:

$$V_p = v_c b_w d + A_v f_{yt}\, d/s$$

$$= k\sqrt{f_c'}\, b_w d + A_v f_{yt} d / s \qquad (2)$$

where v_c = nominal shear stress carried by the concrete mechanisms, f_c' = probable concrete compressive cylinder strength, b_w = width of beam web, d = effective depth of beam, A_v = area of transverse shear reinforcement at spacing s, and f_{yt} = probable yield strength of the shear reinforcement. This equation assumes that the critical diagonal tension crack is inclined at 45° to the longitudinal axis of the beam.

In the non-seismic provisions of NZS 3101:1995, k for beams is given as $(0.07 + 10\, p_w)$ when f_c' is in MPa units, where $p_w = A_s/b_w d$ and A_s = area of tension reinforcement. NZS 3101:1995 requires that k so determined be not more than 0.2, nor need it be less than 0.08. On the basis of test results, both Hakuto et al (12,17) and Priestley (8) suggest that these code recommendations are conservative estimates and that for regions of beams without plastic hinging k = 0.2 could be assumed.

Shear Strength of Columns

Priestley et al (18) on the basis of extensive test results also suggests that the shear strength of columns given by NZS 3101:1995(5) are also conservative estimates and that the probable shear strength of columns without plastic hinging can be taken as:

$$V_p = V_c + V_s + V_n \qquad (3)$$

In Equation 3, V_c is the shear resisted by the concrete mechanisms and is given by:

$$V_c = v_c\, 0.8\, A_g$$

$$= k\sqrt{f_c'}\, 0.8\, A_g \qquad (4)$$

where k = 0.29 when f_c' is in MPa units, v_c = nominal shear stress carried by the concrete mechanisms, A_g = gross area of the column and f_c' = probable compressive cylinder strength of the concrete.

In Equation 3, V_s is the shear resisted by the transverse reinforcement assuming that the critical diagonal tension crack is inclined at 30° to the longitudinal axis of the column. For rectangular hoops:

$$V_s = \frac{A_v f_{yt}(d'' - c)}{s} \cot 30° \qquad (5)$$

and for spirals or circular hoops

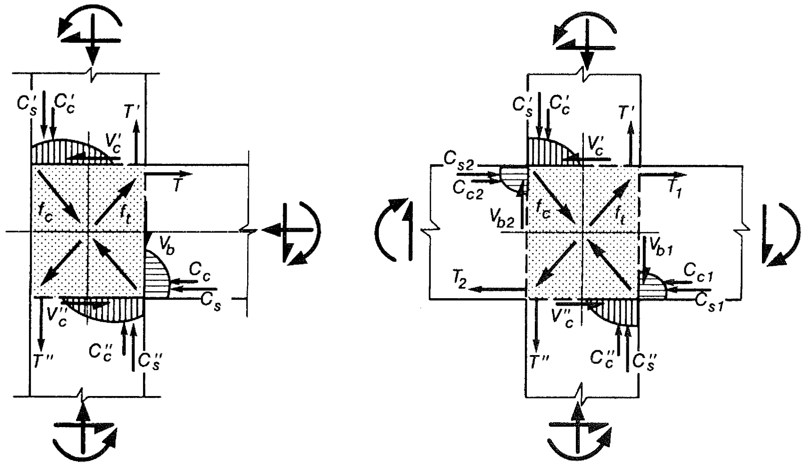
$$V_s = \frac{\pi}{2}\, \frac{A_{sp} f_{yt}(d'' - c)}{s} \cot 30° \qquad (6)$$

where A_v = total area of hoops and cross ties in the direction of the shear force at spacing s, A_{sp} = area of spiral or circular hoop bar, f_{yt} = probable yield strength of the transverse reinforcement, d'' = depth of the concrete core of the column measured in the direction of the shear force for rectangular hoops and the diameter of the concrete core for spirals or circular hoops, and c = distance from the neutral axis to the extreme compression fibre of the section. The introduction of $d'' - c$ rather than d'' into Equations 5 and 6 is a modification by Kowalsky et al (19) to the original truss model which is based on the assumption that only the portion of the transverse reinforcement on the tensile side of the neutral axis cross the potential shear failure plane.

In Equation 3, V_n is the shear resisted as a result of the axial compressive load N^* acting on the column and is given by:

$$V_n = N^* \tan\alpha \qquad (7)$$

where for a cantilever column α is the angle between the longitudinal axis of the column and the straight line between the centroid of the column section at the top and the centroid of the concrete compression force of the column section at the base,



$V_{jh} = T - V'_c$ $\qquad$ $V_{jh} = T_1 + C_{c2} + C_{s2} - V'_c = T_1 + T_2 - V'_c$

(a) Exterior Joint $\qquad$ (b) Interior Joint

FIGURE 2 Forces In Beam-Column Joints During Seismic Loading and Calculation of Horizontal Shear Force Acting on Joint Core.

and for a column in double curvature α is the angle between the longitudinal axis of the column and the straight line between the centroids of the concrete compressive forces of the column section at the top and bottom of the column.

Shear Strength of Beam-Column Joints

NZS 3101:1995 (5) requires the presence of substantial quantities of transverse reinforcement in beam-column joint cores. Hence if there is no, or insignificant, transverse reinforcement in the joint cores NZS 3101:1995 implies that the shear strength of the joint core is negligible.

However, Hakuto et al (12,17) as a result of tests and the analysis of the test results of other researchers, and Priestley (8), suggest that beam-column joints without any, or insignificant, transverse reinforcement in the joint core do have some shear strength, particularly if the joint core is uncracked or if plastic hinges undergoing cyclic deformations in the post-elastic range do not occur adjacent to the joint core. Hakuto et al (12,17) suggest that for beam-column joints without shear reinforcement the maximum probable horizontal joint shear force that can be resisted is:

$$V_{pjh} = v_{ch} b_j h$$
$$= k\sqrt{f'_c}\sqrt{1 + \frac{N^*}{A_g k \sqrt{f'_c}}}\, b_j h \le 1.5\sqrt{f'_c}\, b_j h \qquad (8)$$

where v_{ch} = nominal horizontal joint shear stress carried by a diagonal compressive strut crossing the joint, b_j = effective width of the joint being normally the column width (6), and h = depth of column. It is proposed that when f'_c is in MPa units the following values for k be used: for interior joints, k = 1.0; for exterior joints with beam longitudinal bars anchored by bending the hooks into the joint core, k = 0.4; and for exterior joints with beam longitudinal bars anchored by bending the hooks away from the joint core (into the columns above and below), k = 0.25.

The above recommended values for k are based on the estimated maximum nominal horizontal joint core shear stress, resisted by beam-column joints in tests without joint shear reinforcement and without axial load. The term indicating the influence of axial load, $\sqrt{(1 + N^* / A_g k \sqrt{f'_c})}$, was obtained by assuming that the diagonal tensile strength of the concrete was $k\sqrt{f'_c}$ and calculating using Mohr's circle for stress the horizontal shear stress required to induce this diagonal (principal) tensile stress when the vertical compressive stress is N^*/A_g. The above recommended values for k are based on very limited experimental evidence (12,17,8). Further tests are badly needed to improve the accuracy of the assessment of beam-column joints without, or with little, shear reinforcement.

STEP 3 : THE POST-ELASTIC MECHANISMS OF THE FRAME AND THE PROBABLE LATERAL SEISMIC FORCE CAPACITY

Having determined the probable flexural and shear strengths of the beams, columns and joints of the frame, the next step in the assessment procedure is to identify the probable location of post-elastic deformations due to severe earthquake forces and hence to determine the critical mechanism of post-elastic deformation.

This will involve determining whether flexural plastic hinges occur in the beams or the columns at each beam-column joint and/or whether shear failure occurs in the members or joints. The imposed shear forces on members should be those associated with the plastic hinge (flexural) mechanism. The imposed horizontal shear forces on beam-column joint cores should be those associated with the adjacent plastic hinges. The method of calculating the horizontal joint shear force V^*_{jh} is shown in Figure 2. Comparisons of these calculated imposed shear forces and the probable shear strengths found in Step 2 will determine whether shear failures occur before the flexural strengths are reached or not.

The lateral seismic force capacity associated with the critical mechanism of post-elastic deformation can then be calculated.

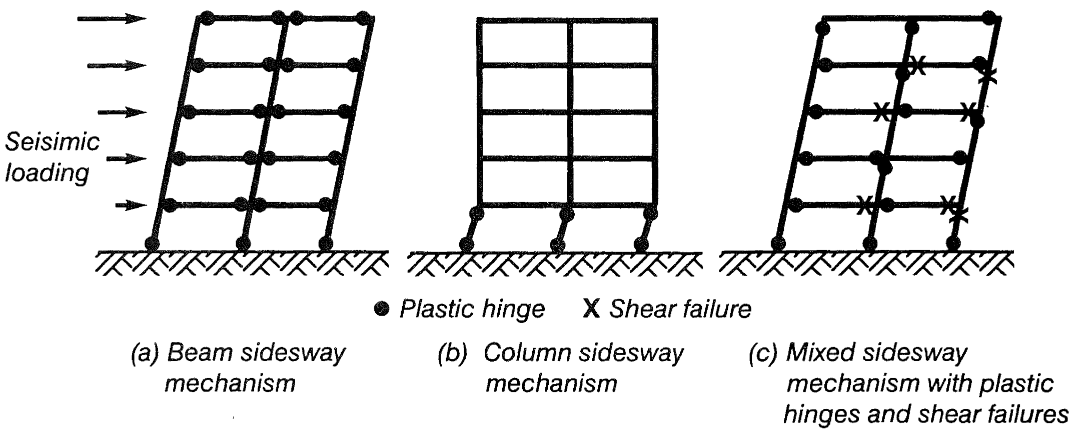


FIGURE 3 *Mechanisms of Post-Elastic Deformation of Seismically Loaded Moment Resisting Frames*

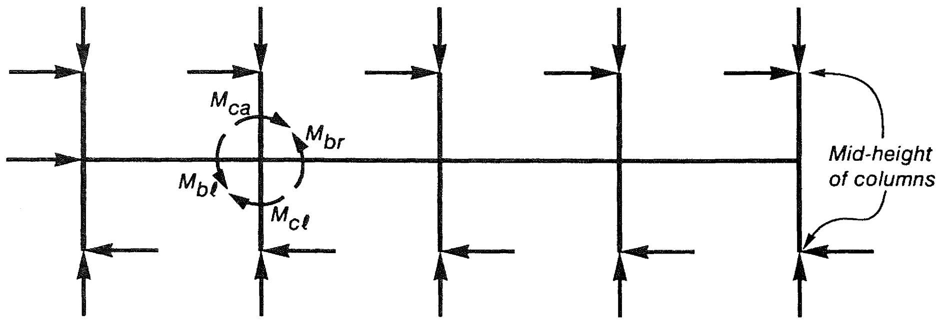


FIGURE 4 *Bending Moments Acting at a Beam-Column Joint at a Floor Level*

Often for a building frame the critical mechanism is not simply a beam sidesway mechanism (see Figure 3a) or a column sidesway (soft storey) mechanism (see Figure 3b), but is a mixed mechanism involving flexural plastic hinges at some locations combined with shear failures of members and/or joints at other locations (for example, see Figure 3c). Note that the plastic hinges could form in any storey of the column sidesway mechanism but generally form in the bottom storey as illustrated in Figure 3b.

To investigate whether the plastic hinges form in the beams or columns at a particular joint the sum of the probable flexural strengths of the beams and the columns at the joint centroid can be compared (see Figure 4). The flexural strength ratio at the joint may be defined as:

$$S_r = \frac{M_{bl} + M_{br}}{M_{ca} + M_{cb}} \qquad (9)$$

where M_{bl} and M_{br} = beam probable flexural strengths at the left and right of the joint, respectively, at the joint centroid, and M_{ca} and M_{cb} = column flexural strengths above and below the joint, respectively, at the centroid of the joint.

When $S_r > 1$ plastic hinges in the columns can be expected.

The sway potential index S_i can be defined as the sum of all the S_r values for the beam-column joints at a floor level. Thus at a floor level:

$$S_i = \Sigma S_r \qquad (10)$$

If the values of the flexural strength ratio S_r for all the beam-column joints at the floors above and below a storey are all greater than 1.0 a column sidesway mechanism can be assumed to occur in that storey since plastic hinges can form at the top and bottom of all columns in that storey.

Also, if the sway potential index S_i for the beam-column joints of the floors above and below a storey are both greater than 1.0 it is possible that a column sidesway mechanism will occur. However, the presence of some joints with a flexural strength ratio $S_r < 1.0$ will prevent a column sidesway mechanism even if $S_i > 1.0$. As an example, a common case for older frames may be the mechanism of post-elastic deformation shown in Figure 5. This mechanism has plastic hinges in the beams forming only at the faces of the exterior columns ($S_r < 1.0$ there). Elsewhere the plastic hinges form at the top and bottom of the interior columns ($S_r > 1.0$ there) and at the column bases, as a result of the flexural strengths of the beams being relatively high and the flexural strengths of the interior columns being relatively low. This mechanism is common for gravity load dominated frames with beams of long span.

Higher modes of vibration can cause an increase in column moments. Also, in a two-way frame seismic loading acting in a general direction can cause a significant increase in column moments due to the simultaneous input of beam moments from two directions (3). Hence it may be wise to assume that column plastic hinges form if $S_r > 0.8$.

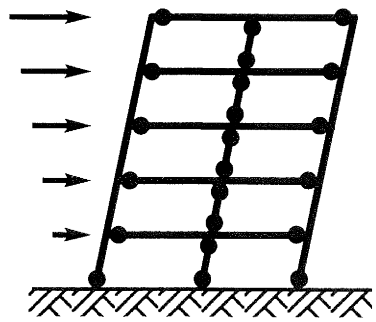


FIGURE 5 *Mixed Sidesway Mechanism of a Gravity Load Dominated Frame*

The probable lateral seismic force capacity of the frame in the general case, when dependent on the flexural strength of members, can be found by any one of the following three methods:

Method 1

Linear elastic structural analysis may be used to determine the distribution of bending moments due to lateral seismic forces and gravity loading. In this analysis, allowance should be made for the effect of concrete cracking on the effective second moments of area I_e of beams and columns. For example, it could be assumed that:

For rectangular beams $I_e = 0.4I_g$, and for T and L beams $I_e = 0.35I_g$,

For columns $I_e = 0.6I_g$ for $N^*/f'_cA_g = 0.2$ and $I_e = 0.8I_g$ for $N^*/f'_cA_g \geq 0.5$ with linear interpolation between,

where I_g = gross second moment of area, N^* = axial compressive load on column, f'_c = probable compressive cylinder strength of the concrete and A_g = gross area of column.

In the analysis the equivalent static earthquake forces are increased from zero until the first plastic hinge forms. The lateral seismic force corresponding to the development of the first plastic hinge gives a lower bound to the probable lateral force capacity of the frame. This lower bound estimate based on the bending moment diagram will always be equal to or less than the actual lateral force capacity. This method may give an extremely conservative estimate of the lateral force capacity. In reality, moment redistribution will permit higher lateral seismic forces to be resisted while further plastic hinges form until a mechanism develops.

Method 2

If the mechanism of post-elastic deformation is obvious from the onset, the lateral seismic force corresponding to the mechanism condition can be calculated directly. For example, a column sidesway mechanism will occur in the bottom storey of the frame if the flexural strength ratio S_r given by Equation 9 is greater than 1.0 for each of the beam-column joints of the floor above. In that case the probable lateral force capacity of the frame is given by the sum of the shear forces in the columns of that bottom storey, found from the sum of the probable flexural strengths of the plastic hinges at the top and bottom of the columns of that storey divided by the storey height. This estimate gives an upper bound to the probable lateral force capacity of the frame and will be always equal to or greater than the actual lateral force capacity. The danger of calculating the probable lateral force capacity by the upper bound approach is that the correct mechanism may be missed and the lateral force capacity overestimated as a result. The mechanism giving the least lateral force capacity is the correct one and must be sought.

Method 3

The most complete approach is to use nonlinear static push-over structural analysis. That is, the lateral seismic forces acting on the frame are gradually increased until a mechanism forms. The behaviour of the frame is in the elastic range until the first plastic hinge forms and then the post-elastic deformations at the plastic hinges need to be taken into account. The number of plastic hinges forming increases with increase in lateral force until a mechanism develops, giving the actual probable lateral force capacity (see Figure 6). Several computer programmes are available which are capable of nonlinear static push-over analysis.

It is to be noted in Figure 6 that V_l is the lower bound value for the probable lateral force capacity, as given by Method 1. V_u in Figure 6 is the correct value as given by Method 3 and corresponds to the upper bound value given by Method 2 for the correct mechanism.

A difficulty with the above methods is the estimation of the distribution of seismic loading to be used in the static analyses. Figure 6 shows a distribution of equivalent static lateral seismic force V which is triangular up the height of the frame, which is close to that assumed by many seismic loading codes. In important cases the designer could check the sensitivity of the seismic force capacity of the frame to this assumption by conducting the analysis for other shapes of seismic load distribution as well; for example, uniform up the height of the frame or point load at the top and less load distributed triangularly.

STEP 4 : DETERMINATION OF THE PROBABLE SEISMIC COEFFICIENT

The probable seismic coefficient available from the structure $C_h(T,\mu)$ can then be determined using Equation 1 knowing the probable lateral force capacity of the structure V_u and the weight of the structure due to the dead load and live load considered to be present during the earthquake W_t.

STEP 5: DETERMINATION OF THE FUNDAMENTAL PERIOD OF VIBRATION OF THE STRUCTURE

The fundamental period of vibration of the structure should next be calculated. The effect of cracking on the section properties of the beams and columns should be included; for example using the effective second moments of area given in Method 1 of Step 3. Also, frames with poorly detailed beam-column joints may undergo a significant reduction in stiffness due to diagonal tension cracking of joints and bond slip of longitudinal bars passing through the joints. For example, Hakuto et al (12,17) tested a poorly detailed beam-column joint which modelled an actual 1950s design but used deformed bar reinforcement and was without axial load on the column.

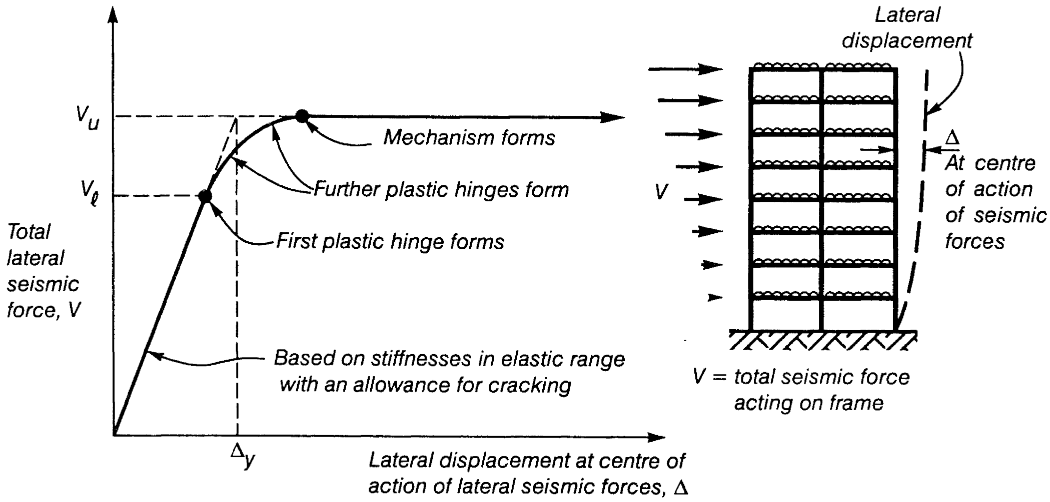


FIGURE 6 ***Typical Lateral Force-Lateral Displacement Relation for a Moment Resisting Frame***

It was found that, after two or three lateral load cycles to about 70% of the yield displacement, to obtain agreement with the measured frame displacements the calculated displacements due to flexure needed to be found using effective second moments of area of about 0.3 of the gross second moment of area of the members and by multiplying the member contributions due to flexure by 1.2 to account for the additional deformations of the beam-column joint.

STEP 6 : ESTIMATION OF THE REQUIRED STRUCTURAL DISPLACEMENT FACTOR

Having determined the probable seismic coefficient $C_h(T,\mu)$ available for the structure and the fundamental period of vibration T_1 for horizontal seismic forces the required structural (displacement) ductility factor μ can be estimated from the inelastic seismic acceleration spectra for the appropriate importance of the structure, seismic zone, site subsoil conditions and any other factors.

Inelastic seismic acceleration spectra have been specified in very few seismic codes. However, they can be determined from elastic acceleration spectra using the equal displacement concept for long period structures and a modified equal energy concept for short period structures. For example, in New Zealand it was proposed for bridges (20) some years ago that the seismic coefficients obtained from inelastic acceleration spectra $C_h(T,\mu)$ for horizontal seismic forces can be related to the seismic coefficients obtained from elastic (μ = 1) acceleration spectra $C_h(T,e)$ as follows:

For $T_1 > 0.7$ seconds

$$C_h(T,\mu) = \frac{C_h(T,e)}{\mu} \qquad (11)$$

For $T_1 < 0.7$ seconds

$$C_h(T,\mu) = \frac{C_h(T,e)}{\frac{T_1(\mu-1)}{0.7} + 1} \qquad (12)$$

Equation 11 is obtained from the equal displacement concept. Equation 12 gives a transition between the equal displacement concept at $T_1 = 0.7$ seconds through the equal energy concept at a smaller period to no reduction in seismic coefficient at $T_1 = 0$ (equal acceleration concept).

Inelastic acceleration spectra for horizontal seismic forces have been specified since 1992 in the New Zealand standard for general structural design and design loadings for buildings NZS 4203:1992(4). For example, Figure 7 shows inelastic hazard acceleration spectra used for intermediate soil sites (soils intermediate between hard and flexible) in New Zealand (4). Equations 11 and 12 were used to determine the seismic coefficients $C_h(T,\mu)$ for $T_1 \geq 0.45$ seconds. For $T_1 < 0.45$ seconds the seismic coefficients have been maintained at the values at $T_1 = 0.45$ seconds to account for the effect of any reduction in stiffness during earthquake shaking of initially very stiff structures. As an example, if the importance factor, zone factor and other factors are all unity and if for instance it was found that the fundamental period of vibration T_1 of the structure is 1.0 second and that the available probable seismic coefficient $C_h(T,\mu)$ is 0.17, the structural (displacement) ductility factor required of the structure would be $\mu = 3.0$. The use of $\mu > 6$ is not permitted by NZS 3101:1995(5) and hence if $\mu > 6$ is found to be needed the structure will require strengthening.

Note that if the building structure composed of moment resisting frames is unsymmetrical in plan the structural (displacement) ductility factor μ required for any particular frame in the building structure may be greater than calculated above due to the torsional deformations of the structure. Then an increased μ value including the effect of torsion will need to be estimated. A method for assessing the torsional response of ductile building structures which are unsymmetrical in plan has recently been published by Paulay (21).

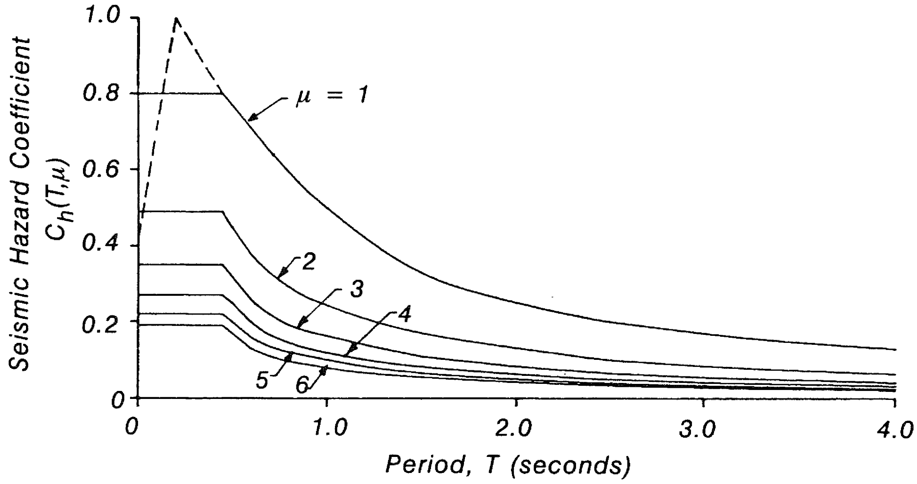


FIGURE 7 ***Seismic Hazard Acceleration Spectra for Intermediate Soil Sites Used in New Zealand (4)***

STEP 7 : ASSESSMENT OF WHETHER THE PLASTIC HINGES HAVE SUFFICIENT AVAILABLE DUCTILITY TO MATCH THE REQUIRED STRUCTURAL DUCTILITY

This step involves estimating the likely plastic hinge rotations and/or section ductilities associated with the required structural (displacement) ductility factor μ and checking whether the plastic hinges have sufficient ductility to match that demand. If sufficient rotation capacity at the plastic hinges is available, then, subject to a satisfactory shear and bond check in Step 7, the frame does not need to be retrofitted. If sufficient rotation capacity at the plastic hinges is not available, the frame will need to be retrofitted.

The required structural (displacement) ductility factor μ is given by Δ_u/Δ_y , where Δ_u is the maximum required lateral displacement and Δ_y is the yield displacement which can be defined as shown in Figure 6.

Any one of three static methods may be used to check the rotation capacity of the plastic hinges:

Method 1

A simplistic approach along the lines of that suggested previously (7) is to compare the detailing of the structure with that recommended by current codes for ductile structures and to assess the available ductility on that basis. Using the detailing requirements of NZS 3101:1995(5) this procedure is as follows:

For potential plastic hinge regions in beams of frames where a beam sidesway mechanism is shown to be likely:

- Where the stirrups are effectively anchored and the stirrup spacing satisfies $s \leq d/2$ and $s \leq 6d_b$, an available structural (displacement) ductility factor of $\mu = 6$ may be assumed for the frame, where d = effective depth of beam and d_b = diameter of longitudinal bars.

- Where the stirrups are not effectively anchored and/or $s > d/2$ or $s > 16d_b$, then an available $\mu = 2$ only may be assumed.

- Intermediate values of μ may be estimated according to the existing detailing of the members based on the above.

For potential plastic hinge regions at the base of columns where a beam sidesway mechanism is shown to be likely, or for frames of one or two storeys in height, where a column sidesway mechanism is likely:

- Where the hoops are effectively anchored and hoop or spiral spacing satisfies $s \leq d/4$ and $s \leq 6d_b$, and where the ratio of volume of transverse reinforcement/volume of concrete core $\geq 0.01\ \{1 + (2N^*/0.7f'_cA_g)\}$ and where the confined length of column at the column base $\geq h\ \{1 + (2N^*/0.7f'_cA_g)\}$, then an available structural (displacement) ductility factor of μ = 6 may be assumed for the frame, where N^* = axial compressive load on the column, f'_c = probable compressive cylinder strength of the concrete, and A_g = gross area of the column.

- Where the hoops are not effectively anchored and/or $s > d/2$ or $s > 16d_b$, then an available $\mu = 2$ only may be assumed.

For potential plastic hinge regions in columns of frames of more than two storeys in height where a column sidesway mechanism is likely, very high plastic hinge rotations can be required of the critical regions of columns. An available μ of only 1.5 should be assumed unless a more detailed analysis (see Methods 2 and 3) is conducted.

Method 2

A more accurate approach would be first to determine the available curvature ductility factors ϕ_u/ϕ_y at the plastic hinges taking account the amount of confining reinforcement present, where ϕ_u = available ultimate curvature and ϕ_y = curvature at first yield, as shown in Figure 8. The available ϕ_u/ϕ_y can be found from (3):

$$\phi_y = \varepsilon_y/(d - kd) \qquad (13)$$

$$\phi_u = \varepsilon_{cu}/c \qquad (14)$$

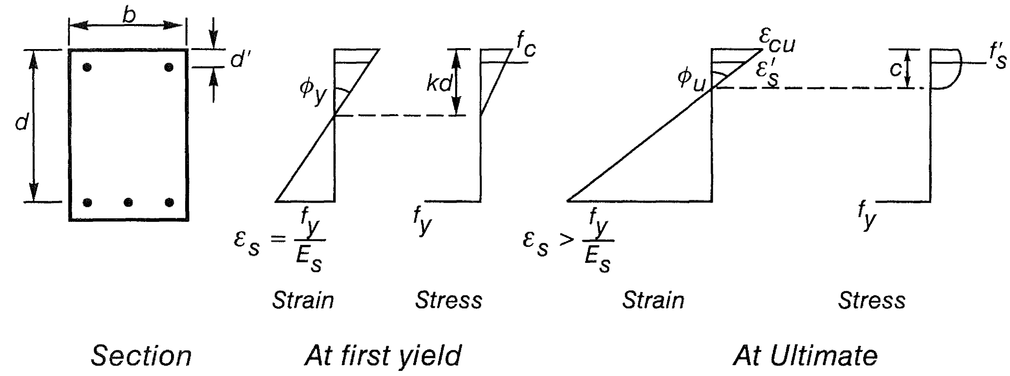


FIGURE 8 ***Doubly Reinforced Beam at Stages of First Yield and Ultimate Curvatures***

where ε_y = yield strain of longitudinal reinforcement, d = effective depth of tension reinforcement, kd = neutral axis depth at ϕ_y, c = neutral axis depth at ϕ_u, , and ε_{cu} = extreme fibre concrete compressive strain at ϕ_u which can be taken to be 0.005 for unconfined concrete (22) and a higher value for confined concrete (23, 24, 25, 8).

A conservative value for ε_{cu} for confined concrete is given by (23):

$$\varepsilon_{cu} = 0.004 + 0.9\, p_s \frac{f_{yt}}{300} \qquad (15)$$

where p_s = ratio of volume of transverse reinforcement to volume of concrete core and f_{yt} = probable yield strength of the transverse reinforcement.

A less conservative value for confined concrete is given by (8):

$$\varepsilon_{cu} = 0.004 + \frac{1.4\ p_s f_{yt} \varepsilon_{su}}{f'_{cc}} \qquad (16)$$

where ε_{su} = ultimate steel strain and f'_{cc} = compressive strength of the confined concrete.

The available structural (displacement) ductility factor $\mu = \Delta_u/\Delta_y$ may then be found from the mechanism by pushing the mechanism laterally until the critical available ϕ_u/ϕ_y value is first reached. This is a simplification since not all plastic hinges in the mechanism form simultaneously (see Figure 6) because for one reason, the vertical profile of horizontal displacement of the frame will not be linear, for example as a result of the effect of the higher modes of vibration. That is, the drift (lateral displacement of a storey divided by the storey height) will not be the same for each storey. However, an approximation for the available μ may be found.

Using this approach (3) the available μ may be found approximately (3) for a beam sidesway mechanism (see Figure 9a), from

$$\Delta_u = \Delta_y + \frac{2}{3}\theta_p n \ell_c$$

$$\therefore \mu = 1 + 2\frac{\theta_p n \ell_c}{3\Delta_y} \qquad (17)$$

and for a column sidesway mechanism, assuming that the affected storey is below the point of action of V_u (see Figure 9b), from

$$\Delta_u = \Delta_y + \theta_p \ell_c$$

$$\therefore \mu = 1 + \frac{\theta_p \ell_c}{\Delta_y} \qquad (18)$$

where the plastic hinge rotation in Equations 17 and 18 is given by

$$\theta_p = (\phi_u - \phi_y) l_p \qquad (19)$$

and the equivalent plastic hinge length can be taken to be conservatively (22) as $l_p = 0.5h$ or more accurately (8) as

$$l_p = 0.08z + 0.022\, d_b f_y \qquad (20)$$

where in Equations 17 to 19 Δ_u and Δ_y are the ultimate (maximum) and first yield horizontal displacements, respectively, calculated at a height of $0.67nl_c$ (that is, at the centroid of a triangular distribution of seismic forces), n = number of storeys of frame, l_c = storey height, z = distance from the point of contraflexure in the member to the section of maximum moment in the plastic hinge region, d_b = diameter of longitudinal reinforcement, f_y = probable yield strength of longitudinal reinforcement, and h = depth of member at the plastic hinge.

For example, if static linear-elastic push-over analysis indicates that the first yield horizontal displacement at the centre of action of the seismic force (Δ_y in Figure 6) is given by $0.2n\phi_y l_c^2$ and if n = number of storeys of the frame = 5, ϕ_y = yield curvature of columns, l_c = storey height = 6h, h = column depth and $l_p = 0.5h$, then for a critical available $\phi_u/\phi_y = 10$ the available μ for a beam sidesway mechanism given by Equations 17 and 19 is $\mu = 3.5$ and for a column sidesway mechanism given by Equations 18 and 19 is $\mu = 1.8$.

This example illustrates, as expected, that for a given available curvature ductility factor at the plastic hinges the available structural (displacement) ductility factor is much less for a column sidesway mechanism than for a beam sidesway mechanism (3).

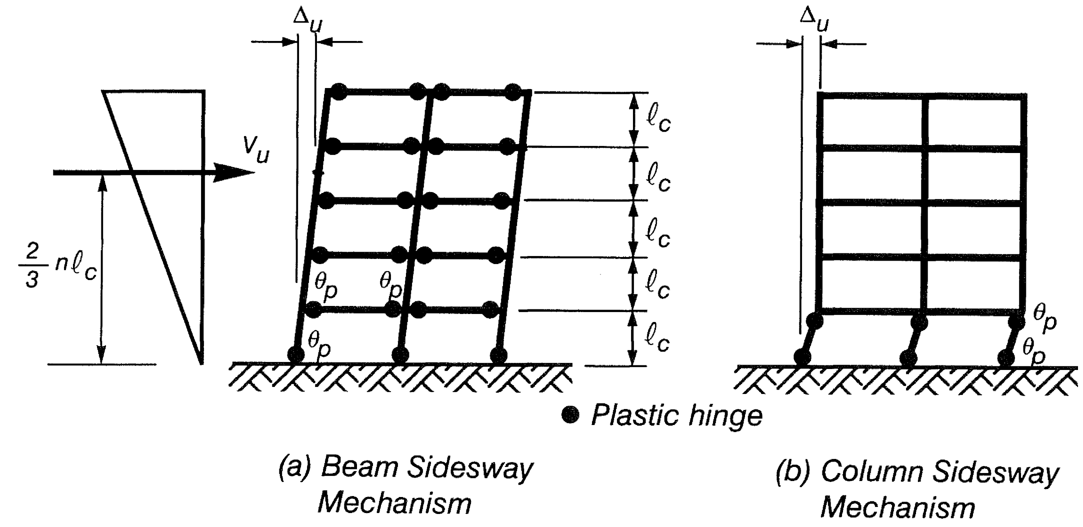


FIGURE 9 ***Calculation of Ultimate (Maximum) Lateral Displacement***

Method 3

The most complete approach for determining the available μ is to use a static nonlinear structural push-over analysis in which the lateral seismic forces on the frame are gradually increased until the available ultimate curvature ϕ_u is first reached at the critical plastic hinge. The available ϕ_u values at the plastic hinges than are found from Equation 14.

This method is particularly essential for frames in which mixed sidesway mechanisms form (see Figure 3c) since such frames cannot easily be analysed by the simpler methods.

In all three methods, if the available μ does not at least equal the required μ the frame will need to be retrofitted.

STEP 8 : CHECK OF THE EFFECT OF DUCTILITY DEMAND ON THE SHEAR STRENGTH OF BEAMS, COLUMNS AND BEAM-COLUMN JOINTS AND BOND STRENGTH

The shear strength of beams and columns in plastic hinge regions, and of beam-column joints when plastic hinging occurs adjacent to the joint, depends on the level of the imposed ductility (8,12,17,18). Hence a mechanism which initiates with flexural plastic hinges may degenerate into plastic hinges with shear failure as the ductility demand increases. Column shear failure is very serious since it can lead to catastrophic collapse of the structure. Joint shear failure is less likely to cause catastrophic collapse but will result in extreme softening of the frame.

The required structural (displacement) ductility factor μ found in Step 6 was calculated using the flexural and shear strengths determined in Step 2 which assumes that no degradation of strength occurs due to cyclic lateral loading in the post-elastic range. Degradation of shear strength may reduce the lateral force capacity of the frame and its effect should be checked. The required curvature ductility factors ϕ_u/ϕ_y can be determined in Step 7 from the required μ. The next step is to determine the reduced shear strengths at those ϕ_u/ϕ_y values. Hopefully the reduced shear strengths will not decrease the lateral force capacity of the frame. However, if the reduced shear strengths are found to be less than the shear forces occurring when the flexural strengths develop at the plastic hinges and/or beam-column joints, the frame will need to be retrofitted.

Also, the strength of lap splices in longitudinal reinforcement in plastic hinge regions, and the bond strength of poorly anchored bars passing through beam-column joints, will tend to degrade during imposed cyclic loading in the post-elastic range. An available structural (displacement) ductility factor μ of greater than 2 cannot be assumed if lap splices in the longitudinal reinforcement exist in plastic hinge regions unless they are heavily confined.

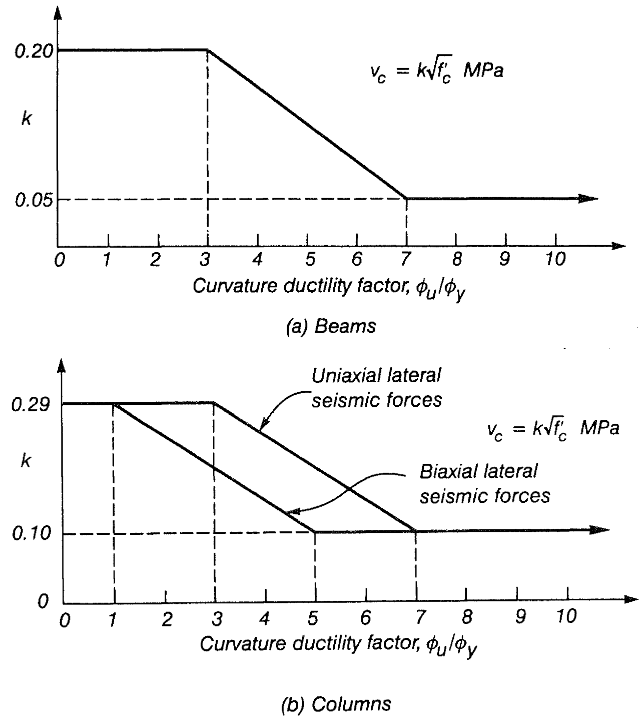


FIGURE 10 ***Degradation of the Nominal Shear Stress Resisted by the Concrete Shear Resisting Mechanisms of Members With Imposed Cyclic Curvature Ductility Factor***

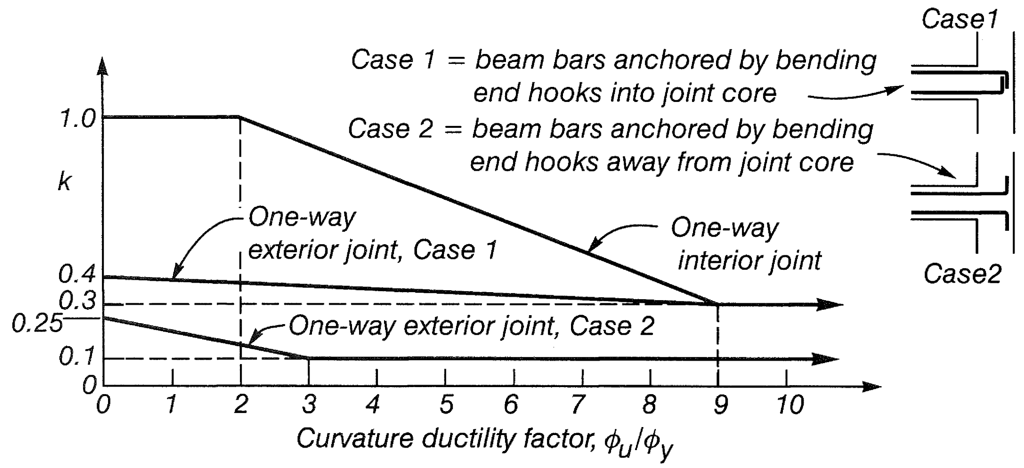


FIGURE 11 ***Degradation of the Nominal Shear Stress Resisted by the Concrete Shear Resisting Mechanisms of Beam-Column Joints With Imposed Cyclic Curvature Ductility Factor***

Degradation of Shear Strength of Beams and Columns

The degradation of the shear strength in plastic hinge regions is due to the degradation of the nominal shear stress v_c resisted by the concrete mechanisms (3). The nominal shear stress which can be resisted reduces with increase in ductility imposed by cyclic loading. Figure 10a and b show proposals for the degradation of the nominal shear stress carried by the concrete, $k\sqrt{f'_c}$ MPa, of beams and columns, as proposed for beams by Priestley (8) and by consideration of the tests by Hakuto et al (12,17), and as proposed for columns by Priestley et al (8,18), expressed in terms of the imposed curvature ductility factor ϕ_u/ϕ_y. The probable shear strength is given by Eqs. 2 to 4 with the appropriate values of k substituted. The value of v_c is as given in Step 2 when the imposed curvature ductility factor is zero. It then reduces linearly during the range of curvature ductility factors shown in the figure, and finally maintains a residual value. The difference between the magnitudes of v_c for beams and columns in Fig. 10 is attributed to the distributed longitudinal reinforcement of columns. Further test evidence is needed, particularly for beams.

Degradation of Shear Strength of Beam-Column Joints

The nominal horizontal joint shear stress v_{ch} resisted by the concrete diagonal compression strut crossing the joint core has been found experimentally to reduce with increase in ductility imposed in the adjacent plastic hinge regions by cyclic loading (8,12,17). Figure 11 shows the degradation in k proposed. The probable horizontal shear force that can be resisted is as given in Step 2 when the curvature ductility factor is zero. It then reduces linearly during the range of curvature ductility factors shown in the figure, and finally maintains a residual value. It is to be noted that interior joints are not as vulnerable as exterior joints. Exterior joints with the 90° hooks at the end of the longitudinal beam bars bent away from the joint core (that is, the ends of the top bars are bent up and the ends of the bottom bars are bent down) do not perform well because the beam bar hooks do not properly engage the corner to corner diagonal compression strut (12,17).

The values of k in Figure 11 are for one-way frames and are expected to be conservative for two-way frames. The test evidence on which Figure 11 is based is limited. The k = 1.0 for interior joints at low ductility was suggested by Hakuto et al (12,17) on the basis of the results of five beam-column joints without joint shear reinforcement tested by five separate investigators in New Zealand, USA and Japan. The k = 0.4 for exterior joints, with beam bar hooks turned into the joint core, at low ductility was suggested by Priestley (8). A test conducted by Hakuto et al (12,17) on this type of joint without shear reinforcement reached k = 0.31 and maintained it during beam plastic hinging up to large ductility factors. A higher k was not reached in this test since the maximum joint shear was governed by the amount of beam reinforcement. The k = 0.25 for exterior joints, with beam bar hooks turned away from the joint core, at low ductility was based on the maximum value for k reached in a test conducted by Hakuto et al (12,17). In this test the value for k was found to degrade rapidly down to about one-half of the initial value with imposed ductility.

With regard to interior joints, when the column depth h to beam bar diameter d_b is less than the value specified in NZS 3101:1995(5), some degradation in bond could be expected. For example, when f'_c = 20 MPa and f_y = 300 MPa, NZS 3101:1995 requires the h/d_b ratio not to be less than 25 for one-way frames. If this requirement is not complied with it is expected that the resulting bar slip would reduce the stiffness of the frame but possibly aid the shear transfer in the beam-column joint (8,12,17). However, two interior beam-column joints without joint shear reinforcement have been tested by Hakuto et al (12,17): one with h/d_b = 25 and f'_c = 53 MPa satisfying the requirement of NZS 3101:1995, and the other with h/d_b = 19 and f'_c = 33 MPa not satisfying the requirement of NZS 3101:1995. Although slip commenced at a lower ductility factor for the unit with the lower h/d_b ratio, the lateral load versus lateral displacement hysteresis loops for the two units were almost identical for cyclic displacements up to a structural (displacement) ductility factor μ of 6. The maximum horizontal joint shear stresses were 0.47 $\sqrt{f'_c}$ and 0.6 $\sqrt{f'_c}$ MPa for the two units. Again, further test evidence is required to improve the accuracy of Figure 11.

STEP 9 : CHECK INTER-STOREY DRIFT

The interstorey drift should be checked to ensure that it is not so large as to introduce significant P-Δ effects or to significantly damage non-structural elements. Ideally the frame should be stiff enough to satisfy the drift limitations of NZS 4203:1992 (4) which have been specified to satisfy those requirements.

EXTENSION OF THE APPROACH TO A GENERAL RANGE OF STRUCTURES

The above assessment procedure for moment resisting frames can be generalised for structures containing frames and walls. A Study Group of the New Zealand National Society of Earthquake Engineering has prepared a document of guidelines for seismic structural assessment including wall structures which is at present in draft form (26).

CONCLUSIONS

1 A static force-based step by step procedure for the seismic assessment of moment resisting reinforced concrete frames can be developed. The assessment procedure determines the probable lateral load strength of the critical mechanism of post-elastic deformation of the frame. The required structural (displacement) ductility factor for that lateral load strength is then estimated from inelastic acceleration response spectra and the frame checked to determine whether that ductility is available.

2 For frames designed before the 1970s, account needs to be taken of the likely seismic behaviour of beams, columns and beam-column joints with the substandard reinforcement details typical of design at that time. Models for the degradation of shear strength as a function of ductility are needed and have been proposed.

3 Further research into the degradation of shear strength of members and beam-column joints as a function of the imposed ductility is needed to refine the models.

ACKNOWLEDGEMENTS

The financial assistance provided by the Earthquake and War Damage Commission (Project No. 91/15) is gratefully acknowledged. The Building Industry Authority of New Zealand and the Tokyu Construction Company of Japan is also thanked for their financial support. The comments of Professor MJN Priestley of the University of California at San Diego are also gratefully acknowledged.

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