

A MODIFIED STATIC PROCEDURE FOR THE DESIGN OF TORSIONALLY UNBALANCED MULTISTOREY FRAME BUILDINGS

A. M. CHANDLER*

Department of Civil and Environmental Engineering, University College London, Gower Street, London WC1E 6BT, U.K.

AND

X. N. DUAN†

School of Civil and Structural Engineering, Nanyang Technological University, Nanyang Avenue, Singapore 2263

SUMMARY

Seismic building codes include design provisions to account for the torsional effects arising in torsionally unbalanced (asymmetric) buildings. These provisions are based on two alternative analytical procedures for determining the design load for the individual resisting structural elements. A previous study has shown that the linear elastic modal analysis procedure may not lead to conservative designs, even for multistorey buildings with regular asymmetry, when such structures are excited well into the inelastic range of response. The equivalent static force procedure as recommended by codes may also be deficient in accounting for additional ductility demand in the critical stiff-edge elements. This paper addresses the non-conservatism of existing static torsional provisions and examines aspects of element strength distribution and its influence on inelastic torsional effects. A recommendation is made for improving the effectiveness of the code-type static force procedure for torsionally unbalanced multistorey frame buildings with regular asymmetry, leading to a design approach which estimates conservatively the peak ductility demand of edge elements on both sides of the building. The modified approach also retains the simplicity of existing code provisions and results in acceptable levels of additional lateral design strength. It has recently been adopted by the new Australian earthquake code, which is due to be implemented early in 1993.

INTRODUCTION

It is widely accepted that the two main objectives of earthquake-resistant design are to ensure safety of life and to protect property. The design philosophy adopted by codes requires that buildings in seismically prone regions be designed to resist a major earthquake without collapse or failure and to resist moderate earthquakes with little or no structural damage. Furthermore, these objectives should be achieved at the lowest possible additional costs; otherwise, the objective of protecting property would be undermined.

Results presented in a companion paper¹ and other related studies²⁻⁵ have shown that the static torsional provisions of certain building codes, together with the alternative linear elastic modal analysis method, may be deficient with respect to the primary objective of earthquake-resistant design referred to above, applicable to buildings responding inelastically to a major earthquake. In particular, the provisions of the New Zealand code (NZ 92, Reference 6) and the European draft seismic code EC8 (Reference 7), and to a lesser extent the Canadian code (NBCC 90, Reference 8) may underestimate the strength demand of resisting elements at the stiff edge of the structure, in cases of intermediate or large eccentricity. Excessively high additional displacement ductility demand and increased risk of failure may, therefore, arise in these cases, compared

*Reader in Earthquake Engineering.

†Research Fellow.

with corresponding symmetric or torsionally balanced^{4,5} buildings. In contrast, the torsional provisions of the Mexico 87 code⁹ include additional requirements which increase significantly the strength capacity of stiff-side elements. These provisions have been found¹⁰ to lead to much lower peak ductility demand of such elements in idealized single-storey stiffness-asymmetric buildings compared with the reference symmetric systems, and the same effect has been observed for stiff-side frame elements in the lower storeys of short period and medium-period multistorey asymmetric buildings.¹ However, in view of the large increase in the total lateral design strength of such buildings compared with that of corresponding reference symmetric systems,¹¹ by a factor of up to 3.5 for structures with large eccentricities, the Mexico 87 code provisions are generally considered to be overly conservative and, hence, uneconomical for design.

Of the major codes considered in a recent series of analytical studies,¹⁻⁵ only the torsional provisions of UBC 88 (Reference 12) have been shown to provide consistent protection against structural damage, to both asymmetric buildings and corresponding symmetric or torsionally balanced reference systems, based on the asymmetric case on achieving uniform control of the additional ductility demand in edge resisting elements. However, numerous studies (References 2 and 13, for example) have shown that the UBC 88 provision along with others such as NZ 92 and to a lesser extent NBCC 90, may significantly underestimate the elastic strength demand and, hence, deformation of flexible-edge resisting elements. These codes may, therefore, be non-conservative when used in damageability (or serviceability) limit state design for small or moderate earthquakes, referred to above.

In view of the above discussion, neither the static torsional provisions included in current seismic building codes nor the alternative dynamic modal analysis method are considered sufficient to ensure a reasonably conservative design of asymmetric buildings in both the elastic and inelastic ranges of response. Employing the relevant results of previous detailed studies of the earthquake response of code-designed asymmetric structures, as well as a series of new analyses based on both single-storey and multistorey building models, this paper develops a recommendation for a modified static design approach, aimed at the prevention of excessive ductility demand arising due to structural asymmetry. The features of this proposed approach, which forms the basis of the static torsional provisions of the draft new Australian earthquake code,¹⁴ are that it is widely applicable, gives conservative estimates of the design loading and deformation of individual resisting elements, retains simplicity for ease of code use and results in acceptable values of cost-related additional lateral strength (overstrength).

SELECTION OF DESIGN ECCENTRICITY PARAMETERS

Primary design eccentricity

When determining the design loading for flexible-edge elements, which are the critical elements in the case of the elastic response of asymmetric buildings, adequate allowance must be made for the dynamic amplifying effect acting on the static torsional moment, represented in codes by a factor, A_1 , applied to the stiffness eccentricity, e_s , to determine the primary dynamic eccentricity. Such amplification arises due to the intensively studied torsional coupling effect.¹⁵⁻²⁰ An amplification factor $A_1 = 1.5$ has been incorporated into the Mexico 76 and 87 codes and in NBCC 90. This amplification factor (based on the early work of Rosenblueth and Elorduy¹⁵), or any other linear amplification factor, is highly simplistic and fails to account for the non-linear relationship, in the elastic range of response, between the dynamic torsional moment and the static stiffness eccentricity, e_s . Furthermore, the important influence of the uncoupled torsional to lateral frequency ratio, Ω , on elastic torsional coupling¹⁶ is also not accounted for by this or other linear static provisions. Therefore, such provisions may underestimate the elastic strength demand or deformation of flexible-edge elements, especially for asymmetric buildings having small to moderate static eccentricity and Ω close to, or slightly greater, than unity.¹³ However, the amplification factor of 1.5 matches the results of elastic dynamic analyses reasonably well for buildings with intermediate or large static eccentricities.

On the other hand, the draft Eurocode EC8 primary dynamic eccentricity expression matches the results of elastic dynamic analyses very accurately¹³ over the full range of realistic static eccentricities, for torsionally stiff buildings with $\Omega \geq 1.2$. It also provides conservative estimates of the elastic strength demand of resisting elements at the flexible edge in buildings with intermediate or low torsional stiffness ($\Omega < 1.2$). However, this

primary dynamic eccentricity expression is generally considered to be too complex in form, and for this and other reasons it is difficult to apply in practical design situations.¹¹

Rutberg and Pekau¹⁷ evaluated the mean plus one standard deviation of the peak dynamic (elastic) displacement of flexible-edge elements in asymmetric buildings for five selected earthquake records and, hence determined the expressions for the required lateral design loading. The results were then compared with design displacements calculated using codified expressions for base shear and torsional moment. The code-type design torsional moment was determined as the product of the design lateral force and the primary design eccentricity, e_{D1} . The latter, in addition to the above-mentioned dynamic eccentricity, includes an accidental eccentricity βb , where b is the floor plan dimension measured perpendicular to the earthquake and the factor $\beta = 0.05$ or 0.1 . It was concluded¹⁷ that the required primary dynamic eccentricity amplification factor A_1 , is in the range of 2.5–3.0 for small eccentricities, e_s , and approaches a value just above unity for large eccentricities. Adopting a similar approach, Chandler and Hutchinson¹⁸ proposed the following formula as the primary design eccentricity expression:

$$e_{D1} = A_1 e_s + 0.05b \tag{1a}$$

where

$$A_1 = [2.6 - 3.6(e_s/b)], \quad A_1 \geq 1.4 \tag{1b}$$

This second-order design eccentricity expression gives an estimate of the peak flexible-edge displacement, which is close to the mean plus one standard deviation of the peak responses obtained from linear elastic analysis using a large number of earthquake records. It corresponds to an amplification factor of about 2.6 at small static eccentricities, with the amplification reducing linearly to a minimum value of 1.4, which applies for large eccentricities, $e_s \geq 0.33b$ (see Figure 1). It is also simple in form and straightforward to apply. For

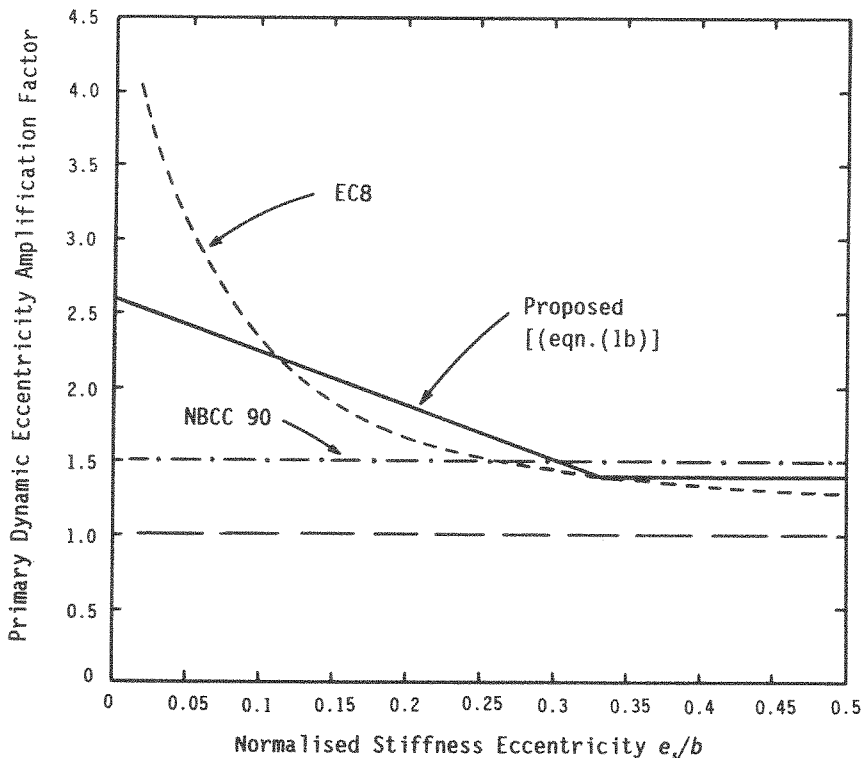


Figure 1. Primary dynamic eccentricity amplification factor of the proposed static design procedure (identical to that of the draft new Australian earthquake code AS 1170-4), in comparison with Eurocode EC8 and the Canadian code NBCC 90

these reasons it has been adopted in the present study as the recommended primary design eccentricity expression. The primary design eccentricity expression of equation (1) leads to a reasonably conservative design of flexible-edge elements in the elastic range of response, as discussed in Reference 19 with regard to the draft new Australian earthquake code. The results of dynamic analyses given below are used to evaluate the corresponding inelastic performance (based on peak displacement ductility demand) of these elements in both single-storey and multistorey buildings, for structures subjected to severe earthquake excitations.

The primary dynamic eccentricity amplification factor A_1 , given in equation (1b), has been plotted in Figure 1 in comparison with the corresponding amplification factors from the NBCC 90 code (namely 1.4) and Eurocode EC8. In presenting results for the latter code, it is assumed that the aspect ratio b/a of the building (with the dimension a measured parallel to the loading direction) is 3:1, and that the ratio of torsional to lateral stiffnesses is such that the corresponding uncoupled frequency ratio Ω is less than 1.2, the range in which greatest torsional effects arise. The EC8 code specifies very high amplification factors for structures with $e_s \leq 0.1b$, but is slightly less conservative than the proposed static provision of equation (1) for buildings with intermediate eccentricities. For large eccentricities ($e_s \geq 0.3b$), the provisions give approximately the same result, noting that for $e_s \geq 0.33b$ the minimum amplification of 1.4 applies for the proposed provision. The NBCC 90 provisions are markedly less conservative than the above-mentioned cases for eccentricities $e_s \leq 0.25b$, but, in contrast, for eccentricities $e_s > 0.3b$ the amplification factor of 1.5 is the largest of the three cases considered.

Secondary design eccentricity

Studies of the inelastic response of code-designed asymmetric buildings^{1-5, 10, 11} have demonstrated that additional inelastic ductility demand may arise in stiff-edge elements compared with symmetric systems when large strength reductions are permitted. These effects are particularly significant for structures with intermediate or large eccentricities, and may be accentuated by higher mode contributions in the response of multistorey asymmetric frame buildings, as studied in the companion paper¹ for structures responding well into the inelastic range. It was concluded in Reference 1 that the linear elastic modal analysis procedure may give non-conservative results when used to specify the design loading for stiff-edge elements. This method does not, therefore, provide an effective distribution of element strength under such loading conditions. Hence, the appropriate specification of design loading for stiff-edge elements is a key issue. Since the secondary design eccentricity expression of building codes usually controls the design loading of stiff-edge elements,²⁰ an improvement to the static design method may be achieved by selecting an appropriate secondary design eccentricity expression, coupled if necessary with changes to the base shear provisions for structures with intermediate or large eccentricities, in order to increase the design loading for resisting elements at the stiff edge. A combination of these approaches has been developed in this paper.

In the case of elastic response, it has been found^{17, 20} that, except for torsionally flexible structures, the following secondary design eccentricity expression provides a good estimate of the dynamic peak displacement and, hence, the elastic strength demand, of elements on the stiff side of the centre of rigidity, CR:

$$e_{D2} = A_2 e_s - 0.05b, \quad \text{where } A_2 = 0.5 \quad (2)$$

It should be noted that equation (2) is similar to that stipulated by NBCC 90, the accidental eccentricity component in the latter case being assigned the larger value, $0.1b$. The ability of this provision to control additional ductility demand in stiff-edge elements has been studied previously for single-storey buildings²⁻⁵ and for a particular type of multistorey asymmetric building,¹ and a modified form is recommended in this paper for reducing the additional ductility demand for buildings with intermediate or large eccentricity.

INELASTIC PERFORMANCE OF SINGLE-STOREY BUILDINGS

The proposed static torsional provisions represented by the design eccentricity expressions of equations (1) and (2) have firstly been assessed in application to a 3-element single-storey, stiffness-asymmetric building model with a rigid floor diaphragm, as illustrated in Figures 2 and 4 of Reference 1. The lateral load-resisting elements are assumed to be bilinear with 3 per cent strain hardening, and the element spacing, d (with

elements located symmetrically about the centre of mass, CM), is selected such that the uncoupled torsional to lateral frequency ratio Ω is unity. Elements 2 and 3 (the latter being the flexible-edge element since it is situated farthest from CR) are assumed to have identical elastic stiffnesses. The stiffness eccentricity e_s is introduced by increasing the stiffness of element 1 (the stiff-edge element) compared with that of the symmetric reference model. The 5 per cent damped Newmark–Hall median response spectrum, illustrated in Figure 3 of Reference 1, is used to define the elastic strength demand for the purposes of design. The base shear force, V_{y0} , or total lateral strength defined by design codes when following the static procedure has been calculated in accordance with the inelastic design spectrum shown in the figure referred to above. The design spectrum is derived from the elastic spectrum (which has been scaled to a peak ground acceleration of $0.3g$) using a force reduction factor $R = 4$.

Element strength distribution from static torsional provisions

The effect of various static torsional provisions on element strength distribution is illustrated in Figure 2. The design strengths have been normalized to the corresponding values in a reference system, defined such that the element strengths are in direct proportion to their stiffnesses, that is, with no consideration given to the torsional effect arising due to the eccentricity e_s between CR and CM. The element strength ratios plotted in Figure 2 are, therefore, indicative of the allowance for the torsional effect made by the proposed static provisions [Figure 2(a)] and those of the existing UBC 88 and NBCC 90 codes [Figure 2(b)]. It should be noted that accidental eccentricity effects have been neglected for both the reference systems and those with asymmetry. This procedure is considered justified for analyses of this type,²¹ as discussed in Reference 1, since none of the factors from which such effects arise have been included in the analyses presented below.

When equation (2) is applied, some reduction in element 1 strength arises compared with the reference system [Figure 2(a)], especially for buildings with intermediate or large eccentricities. For example, a maximum of about 25 per cent reduction in element 1 strength arises in buildings with $e_s/b = 0.35$. However, element 2 and to a greater extent element 3 experience significant increases in design strength, intended to account both for increases in elastic strength demand and the additional inelastic deformation of elements on the flexible side of the structure. Related studies of code-designed structures²⁻⁵ have shown that flexible-edge displacements may be up to 3 or 4 times greater (compared to the symmetric cases) in highly asymmetric systems with relatively low torsional stiffness, and that this ratio is not affected significantly by the design

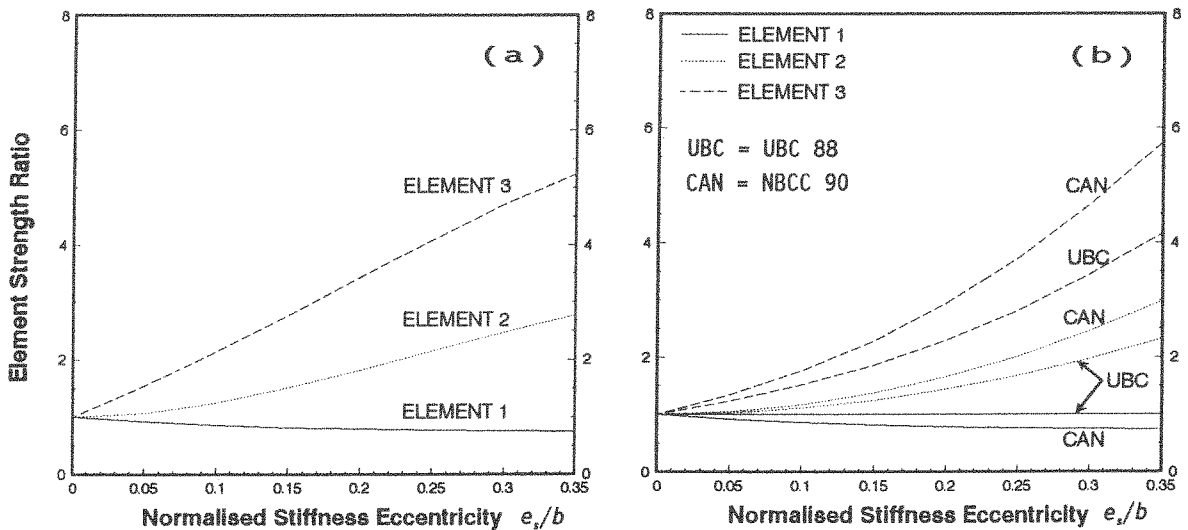


Figure 2. Element strength ratios for structures designed according to (a) the proposed static procedure based on equations (1) and (2), and (b) the UBC 88 and NBCC 90 static torsional provisions

strength of this element. It is observed from Figure 2(a) that the design strength of element 3 increases by ratios greater than 4, when $e_s \geq 0.25b$, which implies a degree of conservatism for design of the flexible-edge element in the inelastic range.

Comparison is made in Figure 2(b) between the element strength ratios resulting from the static torsional design provisions of UBC 88 and the Canadian code NBCC 90. The latter gives the same result for element 3 as in the present study, as expected, given that the secondary dynamic eccentricity A_2e_s , where $A_2 = 0.5$ applies in both cases. However, UBC 88 allows no reduction of strength for this element. The amplification factor $A_1 = 1.5$ in NBCC 90 gives significantly greater increases in the design strengths of elements 2 and 3 particularly at large eccentricities, compared with UBC 88 in which no amplification is required ($A_1 = 1.0$). Nevertheless, at $e_s/b = 0.35$, even the UBC 88 provision results in a strength ratio greater than 4, for the flexible-edge element 3. For the same eccentricity, NBCC 90 gives a strength ratio of 5.7 for element 3, which is slightly higher than the proposed static method [equation (1), see Figure 2(a)], where the corresponding ratio is 5.2.

Inelastic response study

Three selected strong-motion earthquake records from stiff soil sites have been used in the analyses. These have been fully described in Reference 1, and consist of the Imperial Valley 1940 earthquake, El Centro S00E record; the San Fernando 1971 earthquake, 3470 Wilshire Blvd. N00E record; and the Parkfield 1966 earthquake, Cholame Shandon No. 5 N85E record. The 5 per cent damped response spectra of these records (each scaled to $0.3g$ peak ground acceleration) have been shown in Figure 3 of Reference 1, in comparison with the elastic and inelastic design spectra. The El Centro spectrum closely matches the elastic design spectrum over the full period range considered, whilst for periods greater than about 0.5 sec the Wilshire Blvd. and Cholame Shandon records have spectral amplitudes much higher and much lower than the elastic design spectrum, respectively. The response parameter characterizing the peak inelastic deformation of the critical edge elements of the structure is the displacement ductility demand, shown in Figure 3 of the present paper for single-storey structures with small, intermediate and large stiffness eccentricities, namely $e_s = 0.1b$, $0.2b$ and $0.3b$. In order to reduce the differences in element ductility demand (in the medium- and long-period ranges) between the results obtained using the various earthquake records, and thereby facilitate more straightforward comparison, the records have been appropriately scaled in order that the ductility demand of the symmetric reference systems (with equal values for each element) has a value close to or just below the target design value of 4 in the above period ranges.²²

The recommended static procedure leads to satisfactory inelastic performance of asymmetric buildings having small eccentricity. However, the peak ductility demand of the stiff-edge element 1 in asymmetric buildings having intermediate and large eccentricities is excessive in the medium- to long-period range ($T_1 > 0.5$ sec), compared with the reference symmetric systems. This trend is consistent for all three selected earthquake records. The results clearly show that equation (1) gives conservative control of element 3 ductility demand, over the full range of eccentricities. In fact, this provision may be considered somewhat overconservative for buildings with intermediate or large e_s , which in the long-period range may have peak ductility demands of only 1.5–3. This results from the allowance for significantly increased strength in this element (compared with the reference system), shown clearly in Figure 2(a). Nevertheless, since the yield displacement of this element has been increased in accordance with its increased strength, the peak displacement at the flexible edge of the building may still be increased significantly (as discussed above), by a combination of the torsional effect and inelastic response.

The results shown in Figure 3 illustrate the difficulty arising in asymmetric structures when attempting to specify element strength distributions which satisfy both elastic and inelastic design criteria with reasonable (and consistent) levels of conservatism. This problem has still to be satisfactorily resolved, with regard to the performance of both the flexible-edge element (which is the critical element with regard to elastic displacement and corresponding strength demand) and the stiff-edge element (which is the critical element with regard to additional inelastic ductility demand). These issues are the subject of ongoing research studies.

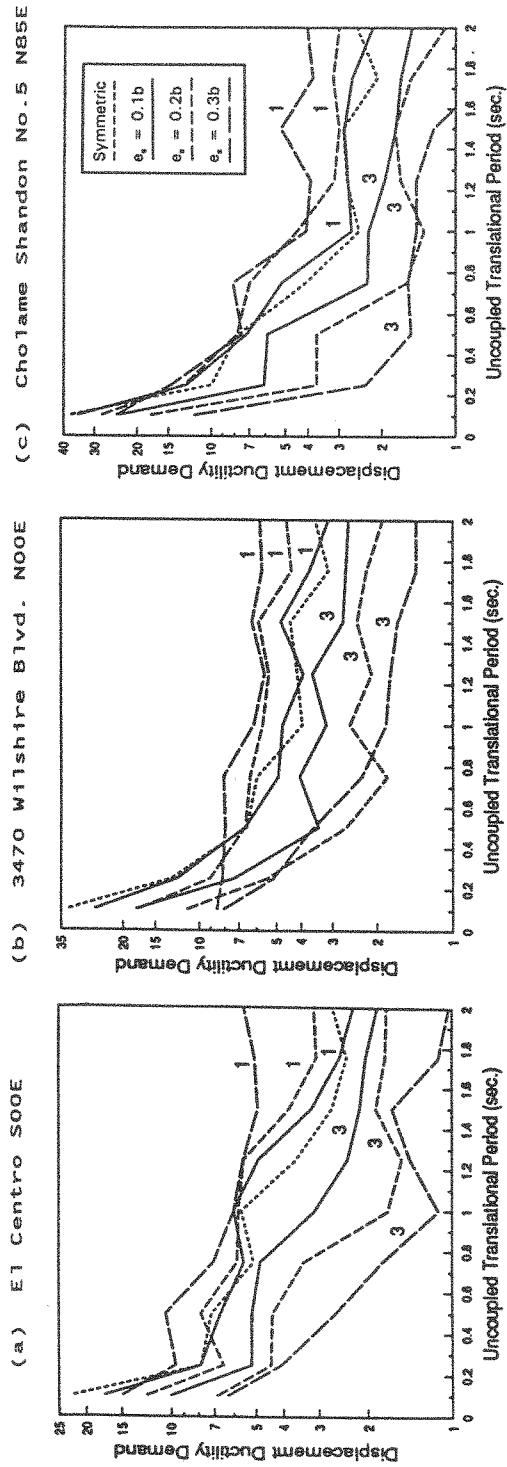


Figure 3. Peak ductility demand of resisting elements 1 and 3 of single-storey buildings designed in accordance with the proposed static procedure

MODIFICATION OF LATERAL DESIGN FORCE

Force reduction factor R

The inelastic performance of asymmetric buildings having intermediate and large stiffness eccentricities can be improved, as commented earlier, by adequately increasing the yield strength of stiff-edge resisting elements. For buildings designed by the static force procedure, this could be achieved by reducing the factor $\lambda_2 = 0.5$ applied to e_s in equation (2), for highly asymmetric structures. However, this procedure would tend to undermine the simplicity of the secondary design eccentricity expression, which in its recommended form has been shown to give satisfactory inelastic performance for buildings with $e_s \leq 0.1b$, as well as leading to an acceptable elastic response^{17,20} over a wide range of the stiffness eccentricity e_s . It is, therefore, considered desirable to preserve the form of the secondary design eccentricity, as given in equation (2).

An alternative method of increasing the design strength of stiff-edge elements is to employ a smaller force reduction factor, R , for asymmetric buildings having intermediate or large eccentricities. Parametric studies of the inelastic response of single-storey plan-asymmetric buildings¹¹ have concluded that using a smaller force reduction factor and, hence, increasing the total strength of asymmetric buildings not only decreases the overall inelastic response of resisting elements but also reduces the significance of additional displacement ductility demand in stiff-edge elements, that is, the effect of stiffness eccentricity is reduced. The regularity requirements of the Mexico 87 code⁹ incorporate such a procedure, as evaluated in Reference 1. This code stipulates that if the stiffness eccentricity e_s exceeds $0.1b$, then the force reduction factor, Q (which is equivalent to R for medium- to long-period structures²²), should be multiplied by a factor of 0.8, which is equivalent to increasing the design base shear by 25 per cent.

Figure 4 presents the peak ductility demand arising in elements 1 and 3 of asymmetric buildings with intermediate ($e_s = 0.2b$) and large ($e_s = 0.3b$) stiffness eccentricities, compared with that of the corresponding reference symmetric systems. In order to assess the influence of the regularity requirement of the Mexico 87 code, outlined above, and to consider its more widespread use in code static procedures, these structures have been designed initially using a base shear determined in accordance with the Mexico 87 code design spectrum,²³ with a force reduction factor $Q = 4$. This design base shear has then been increased by 10, 25 and 50 per cent from the codified value. The 25 per cent increase is in direct accordance with the code recommendations applying when $e_s > 0.1b$. Finally, the yielding strength distribution amongst the resisting

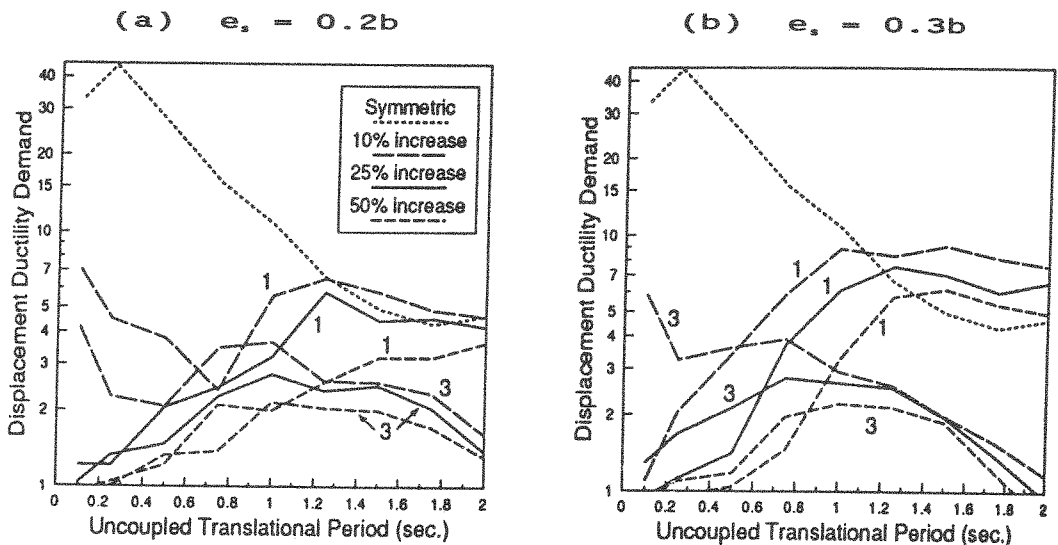


Figure 4. Peak ductility demand of resisting elements 1 and 3 of single-storey buildings designed in accordance with the Mexico 87 code inelastic design spectrum (force reduction factor $Q = 4$) with an increased design base shear of 10, 25 and 50 per cent, and the proposed static torsional design procedure (Mexico City 1985 SCT1 EW earthquake record)

elements is determined by applying the increased base shear at distances from CR equal to the proposed design eccentricity expressions of equations (1) and (2), taking the worst case for each element. Note, however, that for the reasons given above the accidental eccentricity $0.1b$ has been neglected in distributing the design strength. For these analyses, the Mexico City (1985) SCT1 EW record has appropriately been used as the ground motion input.

The results shown in Figure 4 demonstrate that whilst a 10 per cent increase in the design base shear is adequate for buildings with $e_s = 0.2b$ (intermediate eccentricity), it nevertheless still results in significant additional ductility demand of element 1 for lateral periods $T_y > 1.0$ sec, in the case of buildings with large eccentricity ($e_s = 0.3b$). On the other hand, although a 50 per cent increase in the design base shear results in satisfactory inelastic performance of element 1 in buildings with $e_s = 0.3b$, it is overly conservative for buildings with $e_s = 0.2b$. An increase of 25 per cent in the design base shear generally results in reasonably conservative and, hence, satisfactory inelastic performance of the stiff-edge element in asymmetric buildings. This conclusion is based on the results given in Figure 4, for example, and more detailed studies presented in Reference 11.

Strength factor applied to stiff-edge element

Increasing the design base shear results in a proportional increase in the design strength of element 3 and, hence, in view of earlier comments, may exaggerate the conservatism of the design procedure in relation to flexible-edge elements. Hence, it is desirable that the strength factor for the design of highly asymmetric buildings should instead be applied only to the stiff-edge element. This should logically be carried out after applying the secondary design eccentricity as defined in equation (2). A suitable form of this loading or strength factor is illustrated in Figure 5, with a linearly increasing value (to a maximum of 1.25) for structures with eccentricities e_s between $0.1b$ and $0.2b$.

Figure 6 presents the results of analyses carried out incorporating the strength factor for element 1, with the nominal design strength (V_{y0}) determined in accordance with the Newmark–Hall inelastic spectrum with force reduction factor $R = 4$. It is observed that the 25 per cent increase in design strength of element 1 in systems with $e_s = 0.2b$ and $0.3b$ leads to acceptable additional ductility demand in this element, the values not generally exceeding a ratio of 1.2 compared to the reference symmetric case. The performance of element 3 is, as expected, similar to that presented in Figure 3. In fact, the effect of the strength increase in

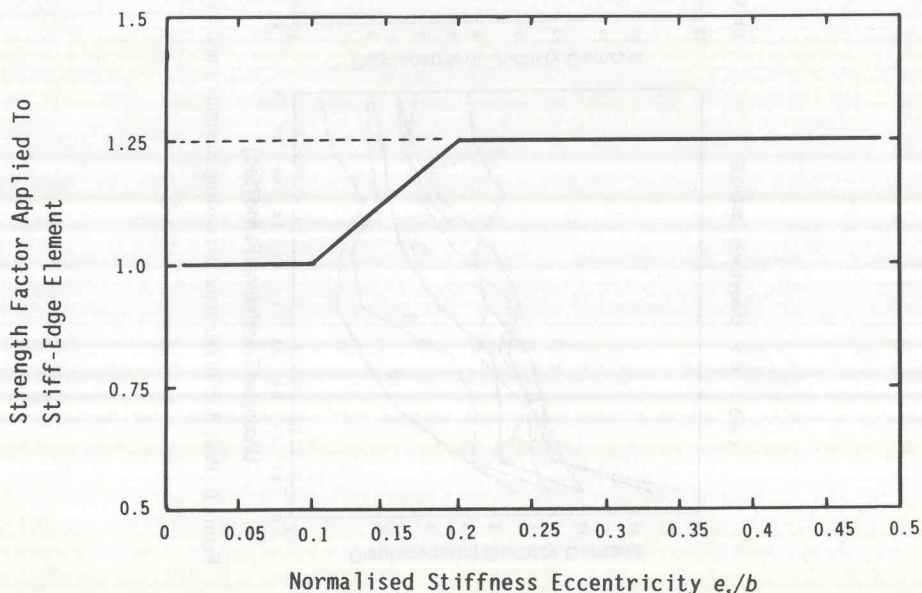


Figure 5. Factor applied to the design strength of element 1 determined from static torsional provisions

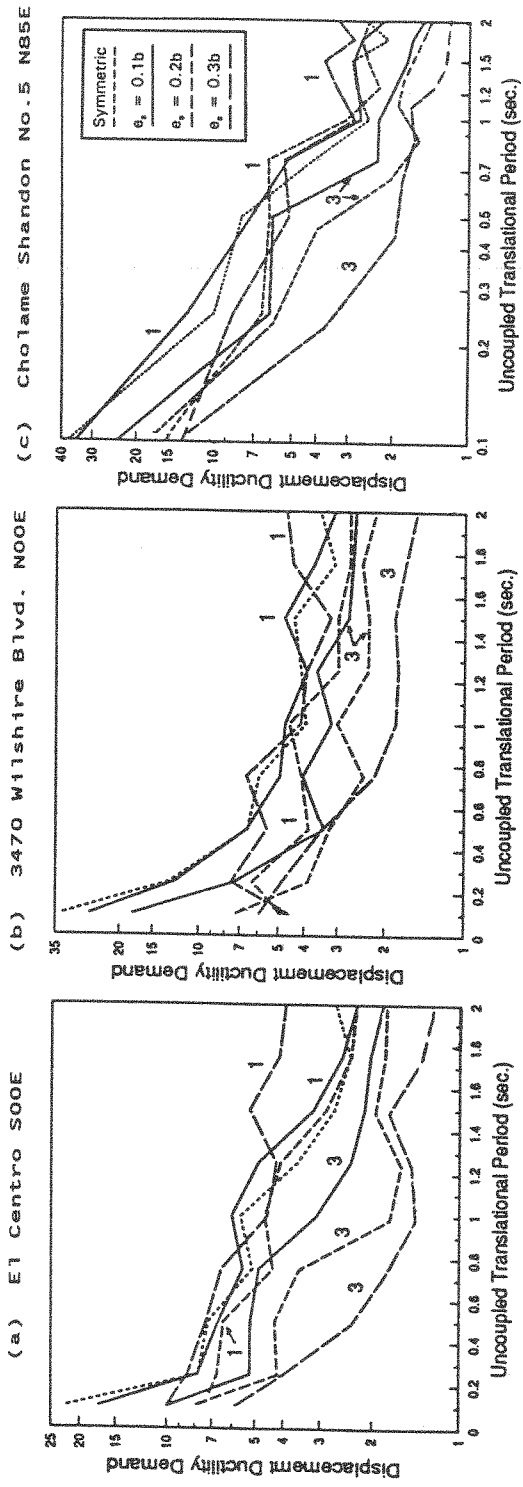


Figure 6. Peak ductility demand of resisting elements 1 and 3 of single-storey buildings designed by the proposed static procedure with a load factor 1.25 applied to element 1 strength for cases $e_s \geq 0.2b$

element 1 for the cases $e_s \geq 0.2b$ tends to result in slightly higher ductility demand in element 3 and, hence, reduce the degree of conservatism for this element compared with earlier results.

Effect of strength factor on horizontal strength distribution

Figure 7 shows the element strengths and total lateral strength based on the proposed static approach (with and without inclusion of the strength factor applied to element 1), each normalized to the nominal design base shear (V_{y0}) obtained from the inelastic design spectrum. The total lateral strength exceeds the nominal value by an overstrength ratio, OS, as plotted in the figure. For the symmetric case ($e_s = 0$), $OS = 1.0$ and each element is assigned one-third of the nominal design strength. As the eccentricity increases, with corresponding increases in the stiffness of element 1, the strength allocated to this element also increases, to a value of about $0.6V_{y0}$ (before application of the strength factor), when the eccentricity $e_s = 0.15b$. There is a small decrease in element 2 strength with increasing e_s . The form of the primary dynamic eccentricity expression, with a reducing amplification factor with increasing e_s (Figure 1), leads to element 3 strength increasing to a maximum of about $0.55V_{y0}$ when $e_s = 0.2b$, thereafter decreasing as a result of the highly reduced stiffness of this element for large values of e_s .

The overstrength ratio, OS, peaks at a value of about 1.4, at $e_s = 0.25b$. This peak value is in general agreement with that of existing code provisions.^{2-5,11} For very large eccentricity ($e_s = 0.35b$), the overstrength ratio associated with the proposed static procedure (not including the element 1 strength factor) is about 1.35, which is slightly smaller than for either the UBC 88 or NBCC 90 codes, for which $OS = 1.42$ in both cases.¹¹ The effect of the strength factor applied to element 1 is also shown in Figure 7, with OS increasing almost uniformly to a value just greater than 1.5 at $e_s = 0.2b$; thereafter, OS remains approximately constant with increasing e_s . The element 1 strength becomes proportionally larger for $e_s > 0.1b$, increasing to a value of $0.75V_{y0}$ at $e_s = 0.35b$.

The peak overstrength ratios quoted above are increased by approximately 0.12 when an accidental eccentricity component of $0.05b$ is included, as in the proposed static procedure and the UBC 88 code. The corresponding increase is approximately 0.25 for codes employing an accidental eccentricity of $0.1b$, for example, NBCC 90.

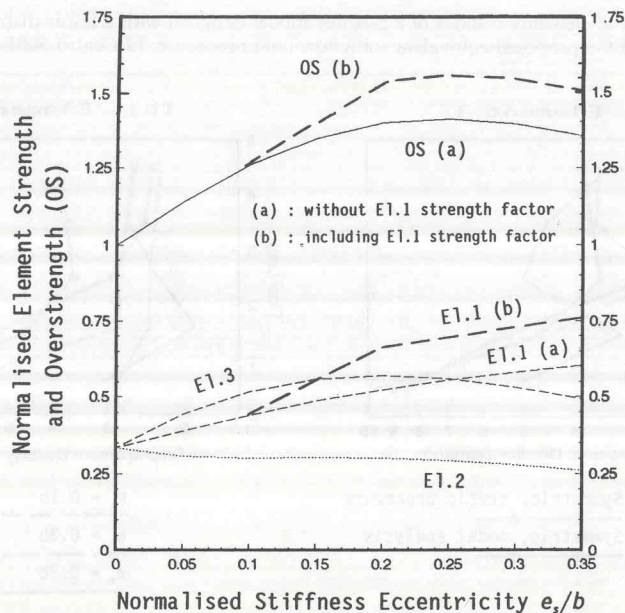


Figure 7. Element strength distribution and total lateral overstrength ratios for buildings designed by the proposed static procedure

APPLICATION OF MODIFIED STATIC PROCEDURE TO MULTISTOREY BUILDINGS

The modified static torsional design approach developed above is intended to provide a method for the horizontal distribution of lateral storey design strength in multistorey regularly asymmetric frame building with idealized properties as described in Reference 1. In this section, an evaluation is made of the effectiveness of this design procedure for inelastic torsional design of such buildings. Initially, a 5-storey model with fundamental uncoupled lateral period T_y of 0.5 sec is employed for this purpose, with the earthquake lateral loading assumed to increase linearly towards the top of the building, as in the majority of code design provisions. For the symmetric (reference) buildings, the linear elastic modal analysis approach has also been used as a method of distributing the base shear over the building height.

The peak displacement ductility demands of columns in frame elements 1 and 3 are presented in Figures 8 and 9, for buildings with $e_s = 0.1b, 0.2b$ and $0.3b$, and for two earthquake input records, namely El Cent

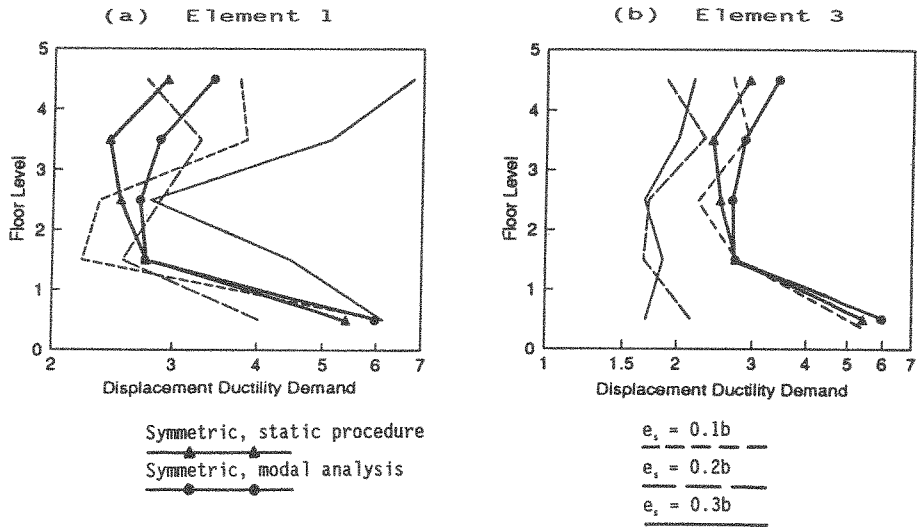


Figure 8. Peak ductility demands of elements 1 and 3 of a 5-storey model designed with a linear distribution of base shear over the height and the proposed equivalent static torsional procedure; El Centro S00E record

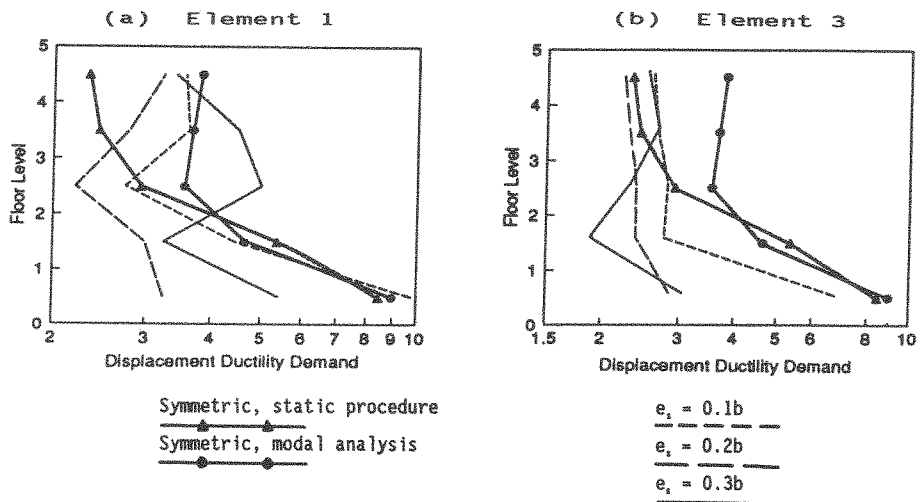


Figure 9. Peak ductility demands of elements 1 and 3 of a 5-storey model designed with a linear distribution of base shear over the height and the proposed equivalent static torsional procedure; 3470 Wilshire Blvd. N00E record

S00E and 3470 Wilshire Blvd. N00E, respectively. It is apparent that the modified static torsional design approach leads to lower ductility demands for element 3 compared with the corresponding symmetric building. However, despite the strength factor applied to columns in frame element 1, the static approach in the form developed for single-storey buildings leads to significant additional ductility demand at certain levels in element 1 when the stiffness eccentricity is large ($e_s = 0.3b$). This is apparent in the upper storeys for the building subjected to the El Centro record, and in the intermediate storeys for response to the Wilshire Blvd. record. However, for buildings with $e_s \leq 0.2b$, the proposed approach leads to satisfactory inelastic performance of both elements 1 and 3.

Some codes recommend the application of a concentrated design force at the top of a building, as a modification to the linear vertical distribution of design loading.^{6, 8, 12} Apart from the NZ 92 code, this top force is not required for buildings with fundamental periods lower than 0.7 sec. In Reference 1, it was concluded that this top force has an important effect in reducing the inelastic response of frame elements in the upper storeys. It was found that the ductility demand in such elements can be excessively high when a simple linear distribution of earthquake design loading is assumed, due to the increased contribution of the higher modes to the structure's total response, which arises in inelastic structures with relatively short elastic periods due to period elongation on yielding. It is concluded from the results shown in Figures 8 and 9 that the top force, F_t , is required even for short-period structures, as in the NZ 92 code,⁶ which requires F_t to be 8 per cent of the base shear, irrespective of the period.

On the basis of the above arguments, the 5-storey model having a large stiffness eccentricity ($e_s = 0.3b$) has been redesigned by applying a top concentrated force F_t of either 0, 5, 10 or 20 per cent of the base shear, and distributing the remainder of the base shear linearly over the height. The modified static procedure (including the strength factor of 1.25 for element 1) is again employed for the horizontal distribution of the storey shears. Figure 10 shows that the application of the top force reduces significantly the inelastic response of columns in the upper storeys of frame element 1. It also decreases slightly the peak ductility demand of columns in the intermediate storeys, but has essentially no effect on the inelastic response of columns in the lower storeys. Whilst a value of 5 per cent of the base shear for F_t is insufficient to reduce the additional ductility demand in the upper storeys to acceptable levels, a value of 20 per cent of the base shear may be considered overly conservative for this purpose. The results corresponding to a top force of 10 per cent of the base shear are close to those of the corresponding symmetric building, and, for the example 5-storey building, this value (which is only slightly higher than that recommended by the NZ 92 code) is, therefore, considered appropriate.

Finally, analyses have been made of an 8-storey model with fundamental uncoupled lateral period $T_y = 1.0$ sec, designed by applying a top concentrated force F_t equal to 10 per cent of the base shear and

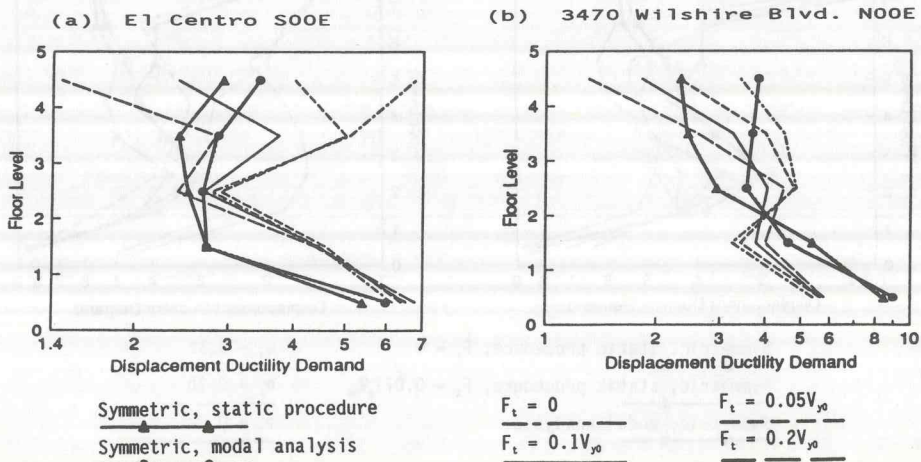


Figure 10. Peak ductility demand of element 1 of a 5-storey model with $e_s = 0.3b$ and designed by the proposed static torsional design procedure, with a top force F_t of 0, 5, 10 and 20 per cent of the base shear, plus a linear distribution of the remainder of the base shear

distributing the remainder of the base shear linearly over the height. The proposed static procedure is again used for the horizontal distribution of storey shears. The results given in Figures 11 and 12 indicate that the above approach gives satisfactory inelastic performance of both the stiff-edge and flexible-edge elements. In particular, the additional ductility demand for element 1 when $e_s = 0.3b$, which arises in the middle to upper storeys, is now limited to acceptable values (compare Figures 11 and 12 with Figures 8 and 9, for example).

It is debatable whether the design of element 3 is over-conservative for structures with $e_s \geq 0.2b$, but this can be corrected only by reducing the provision made in the primary design eccentricity [equation (1)] for increased strength in this element to accommodate the elastic strength demand when designing for moderate earthquakes. The need to study the effect of the variation of the parameters defining the primary design

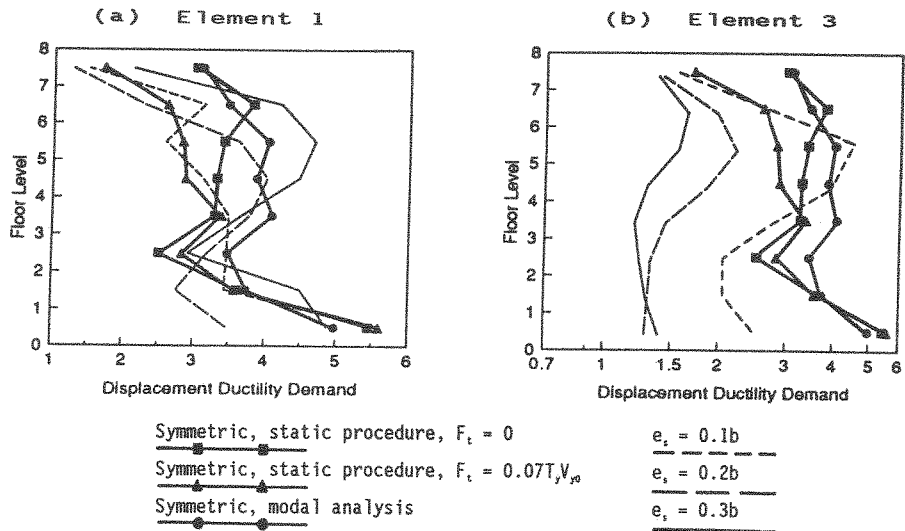


Figure 11. Peak ductility demands of elements 1 and 3 of an 8-storey model designed in accordance with the proposed static torsional procedure, with a 10 per cent top force together with a linear distribution of the remainder of the base shear; El Centro S00E record

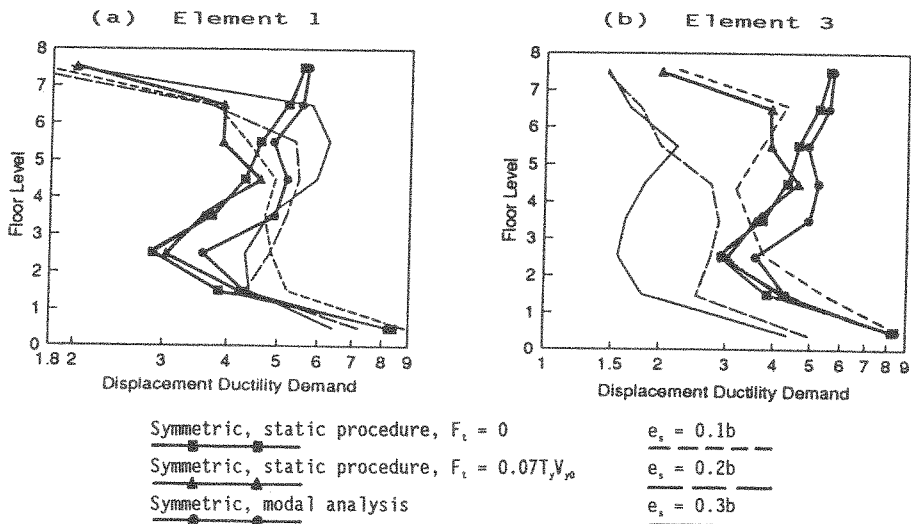


Figure 12. Peak ductility demands of elements 1 and 3 of an 8-storey model designed in accordance with the proposed static torsional procedure, with a 10 per cent top force together with a linear distribution of the remainder of the base shear; 3470 Wilshire Blvd. N00E record

eccentricity, over a range of values of the force reduction factor R (with $R = 1$ representing structures designed to reach the threshold of yielding), is apparent from the results presented in this paper. Reference 2 contains results for single-storey buildings, and recommends that the design eccentricity should be defined differently for elastic and inelastic systems. The present paper has confirmed the desirability of this approach, for both single-storey and multistorey buildings, whilst presenting a unified static torsional design approach giving reasonably conservative results for asymmetric-plan systems in both the elastic and inelastic ranges, when comparing peak element ductility demands with reference symmetric-plan systems.

CONCLUSIONS AND RECOMMENDATIONS

Based on the results of a companion paper,¹ it was concluded that the linear elastic modal analysis procedure recommended by codes for the design of regularly asymmetric buildings is inadequate when such buildings are excited well into the inelastic range, even if the total strength obtained by modal analysis has been scaled up to be the same as that of corresponding symmetric structures. It was, therefore, concluded that a solution for achieving satisfactory inelastic performance of asymmetric buildings without carrying out an inelastic dynamic analysis can best be achieved by improving the static force procedure rather than relying on linear elastic modal analysis.

Based on results obtained for both single-storey and multistorey building models, the latter idealized as regularly asymmetric frame structures, a modified equivalent static force procedure for torsional design has been developed. This approach gives satisfactory elastic and inelastic performance of structures with a range of stiffness eccentricities, and also gives consistent protection against excessive ductility demand arising in both symmetric and asymmetric buildings. Other advantages of this approach are that it retains simplicity for ease of code use and results in acceptable increases in total strength, compared with corresponding symmetric buildings.

The approach may be considered overly conservative for the design of flexible-edge elements in buildings excited well into the inelastic range, when such elements have been designed in order to ensure control over elastic strength demand in response to moderate earthquakes. Accordingly, the equivalent static method should be further examined, with a view to making refinements to accommodate the different design criteria for elastic and inelastic systems. The design of multistorey asymmetric buildings with structural systems based primarily on structural (shear) walls, or dual frame-wall structures, should also be investigated, in order to assess the range of application of the proposed static method.

ACKNOWLEDGEMENT

The authors gratefully acknowledge the support received from the British Council for the Ph.D. research studies of Dr. X. N. Duan, carried out at University College London between 1988 and 1992, under the Technical Co-operation Training Scheme between the U.K. and the People's Republic of China.

REFERENCES

1. X. N. Duan and A. M. Chandler, 'Inelastic seismic response of code-designed multistorey frame buildings with regular asymmetry', *Earthquake eng. struct. dyn.* **22**, 431–445 (1993).
2. A. K. Chopra and R. K. Goel, 'Evaluation of torsional provisions in seismic codes', *J. struct. eng. ASCE* **117**, 3762–3782 (1991).
3. A. Rutenberg, M. Eisenberger and G. Shohet, 'Inelastic seismic response of code-designed single storey asymmetric structures', *Eng. struct.* **14**, 91–102 (1992).
4. W. K. Tso and T. J. Zhu, 'Design of torsionally unbalanced structural systems based on code provisions I: ductility demand', *Earthquake eng. struct. dyn.* **21**, 609–627 (1992).
5. T. J. Zhu and W. K. Tso, 'Design of torsionally unbalanced structural systems based on code provisions II: strength distribution', *Earthquake eng. struct. dyn.* **21**, 629–644 (1992).
6. Standards Association of New Zealand, 'Code of practice for general structural design and design loadings for buildings: NZS 4203', Wellington, 1992.
7. Commission of the European Communities, 'Eurocode No. 8: design for structures in seismic regions, part 1; general and building', Report EUR 12266 EN, Brussels, 1989.
8. Associate Committee on the National Building Code, 'National Building Code of Canada 1990', National Research Council of Canada, Ottawa, Ontario, 1990.
9. R. Gomez and F. Garcia-Ranz, 'The Mexico earthquake of September 19, 1985—complementary technical norms for earthquake resistant design, 1987 edition', *Earthquake spectra* **4**, 441–459 (1988).

- 10 W. K. Tso and H. Ying, 'Lateral strength distribution specification to limit additional inelastic deformation of torsionally unbalanced structures', *Eng. struct.* **14**, 263-277 (1992).
- 11 X. N. Duan, 'Inelastic response and effective design of asymmetric buildings under strong earthquake loading', *Ph.D. Thesis*, University of London, 1991.
- 12 *International Conference of Building Officials*, 'Uniform building code; 1988 edition', Whittier, California, 1988.
- 13 A. M. Chandler and G. L. Hutchinson, 'Evaluation of code torsional provisions by a time history approach', *Earthquake eng. struct. dyn.* **15**, 517-536 (1987).
- 14 Standards Australia, 'Australian standard: minimum design load on structures. Part 4: earthquake loads', *Standards Australia Committee for Loading on Structures*, AS 1170.4 (draft), Sydney, 1992.
- 15 E. Rosenblueth and J. Elorduy, 'Response of linear systems to certain transient disturbances', *Proc. 4th world conf. earthquake eng. and struct. dyn.*, Santiago, Chile, 1969, Vol. 1, pp. 185-196.
- 16 A. M. Chandler and G. L. Hutchinson, 'Torsional coupling effects in the earthquake response of asymmetric buildings', *Eng. struct.* **8**, 222-236 (1986).
- 17 A. Rutenberg and O. A. Pekau, 'Seismic code provisions for asymmetric structures: a re-evaluation', *Eng. struct.* **9**, 255-264 (1987).
- 18 A. M. Chandler and G. L. Hutchinson, 'A modified approach to earthquake resistant design of torsionally coupled buildings', *Bull. N.Z. nat. soc. earthquake eng.* **21**, 140-153 (1988).
- 19 G. L. Hutchinson, A. M. Chandler and J. L. Wilson, 'The draft Australian earthquake standard AS 1170.4: evaluation of the torsional design provisions', *Aust. civ. eng. trans.* (in press).
- 20 A. M. Chandler and G. L. Hutchinson, 'Evaluation of the secondary torsional design provisions of earthquake building codes', *Proc. Instn. civil engs.*, London, Part 2, **85**, 587-607 (1988).
- 21 A. M. Chandler and X. N. Duan, 'Evaluation of factors influencing the inelastic seismic performance of torsionally asymmetric buildings', *Earthquake eng. struct. dyn.* **20**, 87-95 (1991).
- 22 N. M. Newmark and R. Riddell, 'Inelastic spectra for seismic design', *Proc. 7th world conf. earthquake eng.*, Istanbul, Turkey, 1980, Vol. 4, pp. 129-136.
- 23 A. H. Hadjian, 'An evaluation of the ductility reduction factor Q in the 1976 regulations for the Federal District of Mexico', *Earthquake eng. struct. dyn.* **18**, 217-231 (1989).