EARTHQUAKE ENGINEERING AND STRUCTURAL DYNAMICS, VOL. 22, 431–445 (1993)

INELASTIC SEISMIC RESPONSE OF CODE-DESIGNED MULTISTOREY FRAME BUILDINGS WITH REGULAR ASYMMETRY

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SUMMARY

Based on an asymmetric multistorey frame building model, this paper investigates the influence of a building's higher vibration modes on its inelastic torsional response and evaluates the adequacy of the provisions of current seismic building codes and the modal analysis procedure in accounting for increased ductility demand in frames situated at or near the stiff edge of such buildings. It is concluded that the influence of higher vibration modes on the response of the upper-storey columns of stiff-edge frames increases significantly with the building's fundamental uncoupled lateral period and the magnitude of the stiffness eccentricity. The application of the equivalent static torsional provisions of certain building codes may lead to non-conservative estimates of the peak ductility demand, particularly for structures with large stiffness eccentricity. In these cases, the critical elements are vulnerable to excessive additional ductility demand and, hence, may be subject to significantly more severe structural damage than in corresponding symmetric buildings. It is found that regularly asymmetric buildings excited well into the inelastic range may not be conservatively designed using linear elastic modal analysis theory. Particular caution is required when applying this method to the design of stiff-edge frame elements in highly asymmetric structures.

INTRODUCTION AND OBJECTIVES

Torsional response coupled with the translational response of structures has the effect of increasing the deformation and strength demand in certain earthquake load-resisting elements. These effects, in turn, can result in a serious pounding problem between buildings and excessive structural and non-structural damage or even collapse. In asymmetric buildings, inelastic hysteresis behaviour due to yielding, unloading and reloading of structural elements, as well as stiffness and strength deterioration in reinforced concrete structures under cyclic loading, changes the relative stiffness of the structural elements and alters the vibration periods. Element stiffness changes, in turn, shift the centres of rigidity of the building and affect the torsionally coupled response of inelastic structures, triggering behaviour different from that of linear elastic systems. Recent studies[1–5] of inelastic torsional effects in code-designed single-storey buildings have drawn important conclusions regarding the vulnerability of certain load-resisting elements to excessive ductility demand or greatly increased deformation compared with symmetric or torsionally balanced structural systems, and the means by which these can be controlled within acceptable limits. Lessons from recent earthquakes[6,7] also suggest that the individual resisting elements of such buildings should be designed with

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0098–8847/93/050431–15$12·50

Received 19 June 1992
Revised 3 November 1992

realistic load levels, accounting for the different behaviour, caused by yielding, from that predicted by linear elastic analysis, and that the elements be detailed carefully to ensure sufficiently ductile response.

This paper presents a parametric investigation of the inelastic response of a particular form of asymmetric multistorey buildings. Since the torsional provisions in seismic building codes determine the horizontal distribution of earthquake loading among the resisting elements, their adequacy should be examined in terms of the objectives of satisfactory control over additional displacement ductility demand caused by structural asymmetry, and consistent protection for both symmetric and asymmetric structures against structural damage. The high proportion of collapsed or severely damaged buildings arising in recent earthquakes due to structural asymmetry suggests that inadequacies exist in the torsional provisions of codes and further assessment of these provisions is, therefore, necessary. The primary aim of this paper is, therefore, to identify in what way and to what extent these code torsional provisions are inadequate, thereby providing a basis for studies leading to their effective improvement. Such a study is presented in a companion paper,[8] which examines the influence on inelastic torsional effects of element design loadings and strength distribution based on the code-type equivalent static force procedure. A proposal is also made to improve the effectiveness of this procedure for the design of multistorey regularly asymmetric frame buildings, as considered in the present study.

MULTISTOREY REGULARLY ASYMMETRIC FRAME BUILDING MODEL

Most previous studies on the earthquake response of asymmetric buildings, in both the elastic and inelastic ranges, have been based on single-storey asymmetric building models. In studying the elastic response of regularly asymmetric buildings in which the centres of mass and the centres of rigidity lie on two vertical lines, a single-storey asymmetric building model is considered sufficient to investigate the effects of torsional coupling. The earthquake response of multistorey asymmetric buildings may be described in terms of vibration modes containing coupled translational and torsional components. In regularly asymmetric buildings such modes are grouped into pairs, each consisting of a translationally dominated vibration mode and the corresponding torsionally dominated mode. Procedures have been developed[9] to determine the maximum response quantities for such coupled lateral and torsional vibration modes of multistorey asymmetric buildings, by combining those of the corresponding torsionally uncoupled multistorey building and the torsionally coupled single-storey system with identical eccentricity. However, these procedures cannot generally be extrapolated to the inelastic earthquake response of asymmetric multistorey buildings. In the latter case, since the structure's response is both non-linear and inelastic, the vibration periods and mode shapes change with time and the normal co-ordinate uncoupling the equations of motion no longer exists. The principle of superposition, on which the above procedures are based, is, therefore, valid only over a very short time increment in which the structure's dynamic properties are assumed constant, but is not, generally, valid for predicting the structure's inelastic response over the entire response history.

Therefore, a multistorey model is needed to study more completely the inelastic earthquake response of realistic regularly asymmetric buildings and to develop effective design measures. This paper employs such a model to re-evaluate certain of the main conclusions of earlier comparable studies based on single-storey building models. Included is a study of the adequacy of the modal analysis procedure, which is widely relied upon by codes to design for the effects of structural asymmetry and irregularity. Particular attention is focused on the vertical distribution of the peak displacement ductility demand in columns. This parameter characterizes the effect of asymmetry on the dynamic response and indicates the ability of codes to specify lateral strength distributions which acceptably control or limit additional inelastic deformation.

General properties

Many multistorey buildings constructed with vertical and lateral load-resisting frames are characterized by the following features. Firstly, all floors have the same geometry in plan; secondly, the locations of columns in all storeys are the same, and, thirdly, the distribution of stiffness along the height of the building is nearly uniform. Considering these common features, the multistorey regularly asymmetric frame building model illustrated in Figure 1 is assumed to have the following properties:

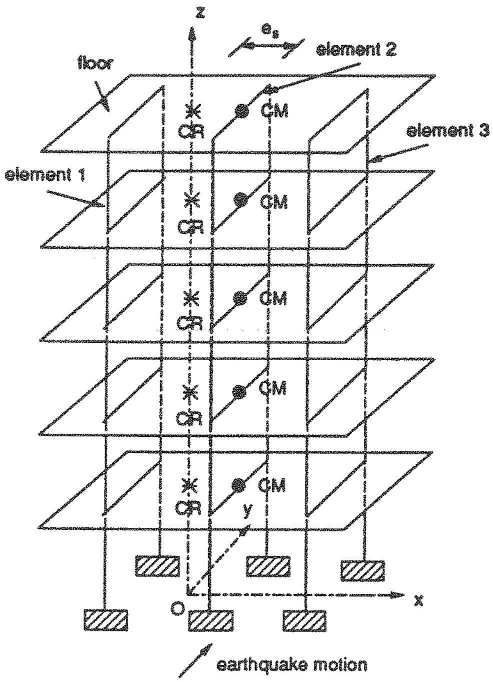

Figure 1. Idealized multi-storey regularly asymmetric frame building model

(1) The model is multistorey and monosymmetric. The distribution of mass, stiffness and strength is symmetric about the x-axis but may be asymmetric about the y-axis.
(2) The floors are rectangular, with a typical aspect ratio a/b equal to 1/3, and consist of perfectly rigid diaphragms (both in-plane and in flexure), supported on massless inextensible columns. The centres of mass, CM, of all floors lie on a vertical line, passing through the geometric centres of the floors. All floors have the same mass, m, and radius of gyration, r, the latter taken about the vertical axis passing through their centres of mass.
(3) There are three planar frame elements oriented parallel to the y-axis, the direction of ground motion. Transverse frames are excluded, and the three frames are assumed to have stiffness in their acting planes (termed the lateral direction) only.
(4) The flexural stiffness of columns and beams is uniform along the height of the building. Furthermore, the flexural stiffness of beams is considered to be very high relative to that of columns, so that each frame can be considered as a 'shear beam' for computational purposes. Hence, the lateral stiffness matrices of all frames are proportional to each other, a condition termed proportional framing.[10] As a result, the floor centres of rigidity, CR (as defined in Reference 13) lie on a vertical line,[11] separated by static eccentricity, e_s, from the vertical line passing through CM at each floor, as shown in Figure 1. The centres of rigidity are defined as the set of points at floor levels through which the given set of lateral design forces induce only translation of the floor diaphragms. Structures having proportional framing with centres of rigidity on a single vertical line are termed regularly asymmetric buildings, for which the uncoupled torsional to translational frequency ratios Ω associated with each pair of coupled lateral and torsional vibration modes (as defined earlier) are equal. Furthermore, for such buildings the locations of the centres of rigidity are independent of the vertical distribution of lateral load.[9,13]
(5) The moment–curvature relationship of all beams and columns is assumed to be bilinear hysteretic with 3 per cent strain hardening. Yielding of beams and columns is defined in terms of the end yielding moments in pure bending. The interaction between bending moment and axial force in columns is neglected.

Structural idealisation

The shear building model defined above is generally regarded as the simplest form of structura representation of multistorey buildings. This model has nevertheless been shown[10] to provide a simplifie computational approach leading to a good first approximation of the torsional moments (and stati eccentricities) of such buildings. For each storey, the structural elements providing the resistance to latera earthquake loading may be considered independently to determine the centre of element stiffness, CS Similarly, when the building undergoes pure translation, for the storey in question the position of th resultant of the element shear forces defines the shear centre. For the shear building idealization, the term centre of stiffness (CS) and shear centre (both defined as in Reference 13), each taken on a storey-to-store basis, are interchangeable. A definition of CS on this basis has been adopted in the draft European seismi code EC8 (Reference 12). The distance measured horizontally from the vertical line through the centres c floor mass to the shear centre or stiffness centre at a particular storey is identical to the earlier definition c static eccentricity, e_s. Hence CR and CS (defined for each floor and storey, respectively) lie on a single vertica line. The eccentricity e_s, therefore, represents the so-called storey eccentricity,[13] and its product with th storey shear force leads to design torsional moments for each storey according to the procedures of seismi codes.

It is widely recognized that two virtually independent failure or collapse mechanisms of moment-resistin frames can occur when they are excited well into the inelastic range. According to whether the plastic hinge form mainly in the columns or in the beams, the structure may develop either a column sidesway or bear sidesway mechanism. The ideal beam sidesway mechanism, on which the capacity design procedure[14] c codes is based, may be difficult to achieve in practice when consideration is given to the flexural strengt contributed by the floor system. The possible (though undesirable) column sidesway mechanism shoul therefore, be considered as the worst possible scenario, as in the present study. Hence, the idealized mode studied herein is considered sufficient to meet the objectives of the study, outlined above. The analysis c inelastic torsional effects in multistorey buildings with properties defined in accordance with the principles c capacity design is the subject of further ongoing research.

MODEL PROPERTIES AND DESIGN LOADING

In order to investigate the effect of the fundamental lateral period on the inelastic earthquake response c multistorey asymmetric buildings, three models having 3, 5 and 8 storeys with typical fundamenta uncoupled lateral periods of $T_y = 0{\cdot}3$, $0{\cdot}5$ and $1{\cdot}0$ sec respectively, have been employed. The torsionall uncoupled (symmetric) multistorey reference system corresponding to the actual torsionally couple (asymmetric) building model is defined with coincident centres of mass and rigidity at all floor levels, but a other properties being identical to the building under consideration. For all models considered in this pape the viscous damping for each mode of the first modal pair is taken to be 5 per cent of critical damping. Sinc the vertical distribution of stiffness is considered uniform, the total lateral storey stiffness K_{yi} ($i = 1,$ 3, $\cdots N$, where N is the total number of storeys) is the same for each storey. Given the fundamenta uncoupled lateral period T_y $(= 2\pi/\omega_y)$, K_{yi} can be determined by solving the eigenproblem

$$[K_y]\{\phi_i\} = \omega_y^2[M]\{\phi_i\} \qquad (1$$

where $[K_y]$ is the system's lateral stiffness matrix, $[M]$ the mass matrix (equal masses at each floor level), an $\{\phi_i\}$ the modal shape corresponding to the ith vibration mode.

Having determined the lateral storey stiffnesses K_{yi}, the horizontal distribution of stiffness and, therefor the frame column stiffnesses are determined on a storey-to-storey basis. For the typical floor plan shown i Figure 2, it is assumed that the stiffnesses of frame elements 2 and 3 (at the centre and flexible edg respectively) are equal, the stiffness eccentricity e_s being introduced by increasing appropriately the stiffnes of element 1 (at the stiff edge). The element spacing, d, is determined from the condition that the ratio Ω of th uncoupled torsional to lateral fundamental frequencies of the building has the value unity. This value leads t significant inelastic torsional response[15] and is representative of buildings with moderate torsional stiffnes In general, inelastic torsional response effects have been found to be less sensitive to changes in the paramete

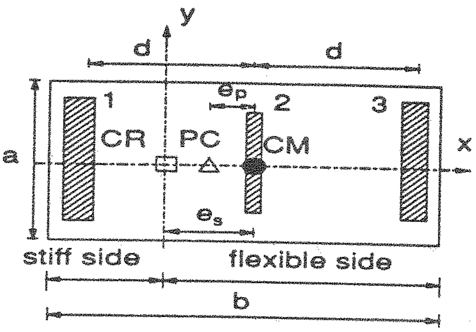

Figure 2. Asymmetric floor plan configuration

Ω compared to elastic torsional coupling effects,[16] and the value $\Omega = 1{\cdot}0$ generally leads to conservative predictions of peak element response (displacement or ductility demand) compared to models having relatively high torsional stiffness and $\Omega > 1{\cdot}0$. Buildings with low torsional stiffness ($\Omega < 1{\cdot}0$) are subject to very high torsional responses,[4] and the avoidance of such structural configurations is highly recommended in design. Seismic building codes apply a penalty to such designs by specifying large increases in strength for flexible-edge elements (element 3 in the present study), to accommodate the increased displacement at this side of the building and to avoid the development of excessive ductility demand.

The 5 per cent damped Newmark–Hall median elastic response spectrum,[17] shown in Figure 3 scaled to a peak ground acceleration of $0{\cdot}3g$, has been employed as the design spectrum representing the elastic strength demand. The design base shear has been calculated in accordance with the inelastic design spectrum shown in Figure 3, together with the structure's fundamental uncoupled lateral period. The inelastic design spectrum corresponds to a force reduction factor $R = 4$ (typical of buildings designed to respond well into the inelastic range) and has a constant amplitude for periods less than 0·5 sec, reducing hyperbolically for longer periods. The control period (0·5 sec) is appropriate for stiff-soil records as employed in the analyses described below, and is typical of the corresponding design spectra of codes.

The base shear is distributed vertically in accordance with the procedures specified in the design codes of Europe[12] (the draft Eurocode EC8), Canada[18] (NBCC 1990), the United States[19] (UBC 1988), the Federal District of Mexico[20] (1987) and New Zealand[21] (NZS 4203: 1992). In the case of the 3-storey and the 5-storey models, all codes except New Zealand (NZ 92) assume a linear distribution of the base shear force over the building height, increasing to a maximum at the top floor. For the 8-storey model having $T_y = 1{\cdot}0$ sec, the Canadian and U.S. codes NBCC 90 and UBC 88 require a concentrated force, F_t, of 7 per cent of the base shear to be applied at the top of the building, the remainder being distributed linearly over the height. The NZ 92 code specifies F_t to be 8 per cent of the base shear, independent of building height or period. Such increases of design force at the top of the building are intended to allow for deviations in the fundamental vibration mode shape from a linear variation, and for the effects of higher modes of vibration, which are particularly significant (based primarily on the results of elastic response studies) in the upper parts of the building. For the buildings considered in this study, neither Eurocode EC8 nor the Mexico 87 code requires consideration of the top force. It should be noted that for long-period multistorey buildings, for instance, when the fundamental period T exceeds 3·9 sec for structures located in the Mexico City lake bed zone (Zone III), the Mexico 87 code requires the addition of a second-order term to the linear function when specifying the vertical distribution of the design base shear. This is intended to account for the influence of higher modes, but is not applicable in the cases of the short- and medium-period buildings considered in the present study.

Except for the Mexico 87 code, all the remaining codes encourage the use of modal analysis to determine the vertical and horizontal distributions of the earthquake lateral load. Further, for analysis of irregularly asymmetric buildings, the use of modal analysis (rather than the static force procedure) is mandatory. The static procedure generally leads to a conservative estimate of the base shear compared to modal analysis.[15]

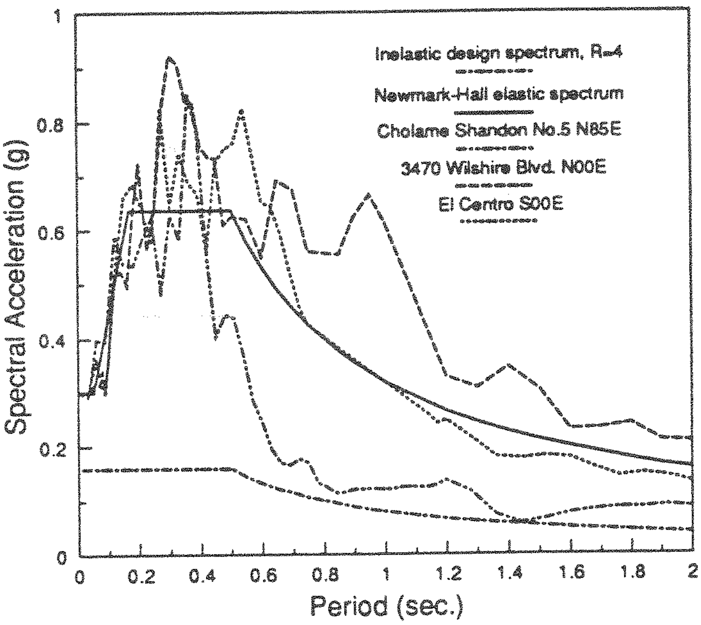

Figure 3. Elastic 5 per cent damped acceleration response spectra of selected earthquake records, and corresponding medi
Newmark–Hall elastic and inelastic design spectra

Hence, when using modal analysis, certain codes require that the strength of all structural elements be scal such that the total strength at the building's base is at least equal to (NBCC 90 and NZ 92), or at least 90 p cent (UBC 88) of, the base shear determined by the static force procedure. The Mexico 87 code is less stringe and requires that the total strength at the building's base should be at least 80 per cent of the static value. should also be noted that Eurocode EC8 does not require such a scaling up of element strength. Finally, f the purposes of this study, the influence of gravity effects in determining the lateral design strength columns has not been explicitly considered.

HORIZONTAL DISTRIBUTION OF STOREY DESIGN STRENGTH

The element strengths are specified in accordance with code torsional provisions. These specify the use of tv design eccentricities, adopting the more unfavourable design force for the element under consideration. T so-called primary and secondary design eccentricities, e_{D1} and e_{D2}, respectively, account for the increased the decreased strength demand in certain elements. They define the locations, relative to either the stor shear centre or floor centre of rigidity CR, through which the design lateral load (storey shear) V_{y0} must applied to induce the design storey torque acting about a vertical axis through the centre of rigidity of t floor being considered (Figure 4). The building codes listed in Table I specify primary and secondary desi eccentricities in the standard form:

$$e_{D1} = e_{d1} + e_a \quad (2$$

$$e_{D2} = e_{d2} - e_a \quad (2$$

in which e_{d1} and e_{d2} are termed the dynamic eccentricities, which take into account the dynan amplification of the static eccentricity. These usually take the form of $e_d = \alpha e_s$, where α is $\geqslant 1{\cdot}0$ for t primary case and $\leqslant 1{\cdot}0$ for the secondary case. In Table I, the term e_1 in the Eurocode EC8 formula is t additional eccentricity accounting for the dynamic amplification of the static eccentricity e_s. The equatic for calculating e_1 are given in Reference 12 and are dependent, unlike the remaining codes, on the buildin

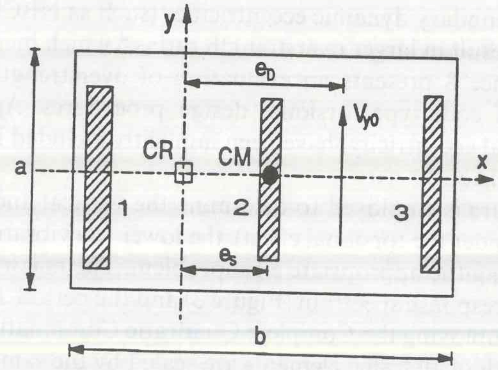

Figure 4. Definition of the storey design eccentricity e_D as given in codes

Table I. Building code regulations for torsional design eccentricities

Building code	Primary design eccentricity	Secondary design eccentricity
Eurocode[12] (EC8: 1989)	$1{\cdot}0e_s + e_1 + 0{\cdot}05b$	$1{\cdot}0e_s - 0{\cdot}05b$
Canada[18] (NBCC: 1990)	$1{\cdot}5e_s + 0{\cdot}1b$	$0{\cdot}5e_s - 0{\cdot}1b$
USA[19] (UBC: 1988)	$1{\cdot}0e_s + 0{\cdot}05b$	$\begin{cases} 1{\cdot}0e_s - 0{\cdot}05b, & e_s \leq 0{\cdot}05b \\ 0, & e_s > 0{\cdot}05b \end{cases}$
Mexico[20] (1987)	$1{\cdot}5e_s + 0{\cdot}1b$	$1{\cdot}0e_s - 0{\cdot}1b$
New Zealand[21] (1992)	$1{\cdot}0e_s + 0{\cdot}1b$	$1{\cdot}0e_s - 0{\cdot}1b$

plan aspect ratio and torsional to lateral stiffness ratio. These equations are based directly on linear elastic modal analysis theory[22] and, hence, their effectiveness when employed for inelastic torsional design is questionable. For UBC 88, the secondary design eccentricity e_{D2} is either negative (for $e_s < 0{\cdot}05b$) or zero, since the so-called negative shears, which reduce the strength demand of elements on the stiff side of CR (Figure 1), are prohibited by this code.

The second term, e_a, in equations (2a) and (2b) usually takes the form of βb, that is a fraction of the dimension of the building perpendicular to the earthquake loading, where $\beta = 0{\cdot}05$ or $0{\cdot}10$. This term is the so-called accidental eccentricity intended to account for all uncertainties in design which may result in additional torsional effects, and the rotational component of the ground motion. Normally, none of these effects are considered explicitly in design, and correspondingly they have been omitted from the dynamic analyses presented below. This procedure is considered justified for analyses of this type,[23] provided consistency is maintained by omitting accidental torsional effects from the determination of element strengths for both the symmetric (reference) models and asymmetric or torsionally unbalanced cases. It should be noted that this has not always been the case in previous studies of inelastic torsional effects in code-designed buildings. The position of the resultant of the element design strengths defines the storey centre of strength or plastic centroid, PC (Figure 2), and, hence, the corresponding strength eccentricity, e_p. For the majority of building codes, including those considered in this study, the resulting strength eccentricity is small compared to the stiffness eccentricity e_s, and, hence, PC is located close to CM. In the Mexico 87 code,[20] special provisions are included which specify a minimum strength eccentricity of $e_s - 0{\cdot}1b$ (for a force reduction factor $R = 4$), or zero, whichever is the greater. Further, in this code the design force (base shear) is increased by 25 per cent for highly asymmetric buildings with eccentricity $e_s > 0{\cdot}1b$. This is effected by multiplying the force reduction factor R by a factor of 0·8.

The application of code torsional provisions results in an overstrength ratio ($> 1{\cdot}0$) for the design storey shears (total storey strengths) compared to the reference symmetric system. Codes with large differences between the primary and the secondary dynamic eccentricities (such as NBCC 90, see Table I), and/or larg accidental eccentricity, tend to result in larger overstrength ratios,[4] which increase with the magnitude of th stiffness eccentricity, e_s. Reference 8 presents an evaluation of overstrength and element strength ratic arising from the application of code-type torsional design procedures. Appropriate overstrength ratic (neglecting the effect of accidental eccentricity) have been implicitly included in the present study in applyin the various code torsional provisions.

If the modal analysis procedure is employed to determine the vertical and the horizontal distribution c element strength (that is, to consider the torsional effect), the lower few vibration periods T_i and mode shape $\{\phi_i\}$ are first computed by solving the appropriate eigenproblem. The maximum modal response quantitie are determined from the elastic response spectrum (Figure 3) and the period T_i. The total maximum respons quantities are then computed employing the Complete Quadratic Combination (CQC) procedure.[24] Finall the lateral force resisting strength of all frame elements are scaled by the same proportion such that the tot lateral strength of the first storey equals the (inelastic) base shear calculated by the static procedure.

GROUND MOTION INPUT AND THE INELASTIC RESPONSE PARAMETER

The non-linear equations of motion[15] have been solved by a step-by-step numerical integration method.[2 The time interval of the numerical integration has been selected to be 1/30th of the smallest elastic mod period of the first three pairs of coupled lateral–torsional vibration modes. This is sufficiently small to ensu stable and accurate numerical integration of the response contributed from at least the first six couple vibration modes of the asymmetric multistorey building.[15]

Three strong-motion earthquake records have been selected as ground motion input, namely, the Imperi Valley 1940 earthquake, El Centro S00E record, the San Fernando 1971 earthquake, 3470 Wilshire Blv N00E record and the Parkfield 1966 earthquake, Cholame Shandon No. 5 N85E record. These records ha been scaled to a common peak ground acceleration of $0{\cdot}3g$, and their 5 per cent damped elastic respon spectra have been plotted in Figure 3, in comparison with the corresponding Newmark–Hall elastic desig spectrum.

All the records were recorded on stiff-soil sites, but have significantly different ratios of peak grour acceleration, a, to peak ground velocity, v, and, hence, represent three distinct categories of earthqual ground motions. The Cholame Shandon record, with a relatively high ratio a/v of $1{\cdot}82g/(\mathrm{m/s})$, has a grour acceleration exhibiting large-amplitude, high-frequency oscillations in the strong-motion phase, and has ve high spectral amplitudes in the very short period range but very low spectral amplitudes for periods larg than 0·25 sec. Since the fundamental elastic periods of the structures considered in this paper are $\geqslant 0{\cdot}3$ se and the elastic strength demand using the Parkfield record for such structures is much smaller than th derived from the Newmark–Hall elastic design spectrum (Figure 3), results obtained using this record ha not been presented herein (see Reference 15 for details). The El Centro record, with $a/v = 0{\cdot}96g/(\mathrm{m/s})$, has acceleration spectrum similar to the standard Newmark–Hall type design spectrum, and the Wilshire Blv record $[a/v = 0{\cdot}61g/(\mathrm{m/s})]$ is representative of ground motions containing a few severe, long-duratic acceleration pulses. The latter record has very high spectral accelerations in the medium- and long-peric ranges, with amplitudes significantly exceeding those of the elastic design spectrum. Hence, in the dynam analyses, particularly for the 8-storey building with $T_y = 1{\cdot}0$ sec, the inelastic element responses a considerably larger when employing the Wilshire Blvd. record than those resulting from the El Cent record. Since the aim of this study is to observe response characteristics for multistorey buildings and compare with previous studies of equivalent single-storey asymmetric buildings (rather than to obta specific numerical responses or to treat the results in a statistical manner) and, furthermore, in order to ensu reasonable computational economy whilst allowing a range of torsional design provisions to be consider the use of these two representative strong-motion earthquake records is considered to be justified.

One of the most important indicators of seismic damage to structural elements (columns and beams) ductile moment-resisting frames is the curvature ductility demand μ_c, which is defined as the ratio of t maximum curvature reached at a plastic hinge to the curvature when this hinge starts developing. Anoth important inelastic response parameter in such frames is the maximum storey drift. Most codes require th the latter response parameter (which is non-uniformly distributed in the case of asymmetric buildings) be limited to an acceptable level, since excessive drift may result in increased p–Δ effects, structural instability, and even total collapse. If a column sidesway mechanism has been developed in such a frame, then the curvature ductility demand of a particular element and the corresponding column end-to-end displacement are related directly to the element displacement ductility demand μ. The local (column curvature) ductility demand may be much higher (by a ratio of 2 to 3) than the displacement ductility demand if a column sidesway mechanism develops in a frame.[15] In view of these relationships, this paper employs the element displacement ductility demand, μ, as the characteristic parameter describing the peak inelastic earthquake responses of asymmetric multistorey buildings.

INELASTIC TORSIONAL RESPONSE AND EVALUATION OF DESIGN PROVISIONS

Inelastic dynamic time history analyses have been carried out to investigate the inelastic earthquake response of multistorey regularly asymmetric frame buildings designed in accordance with the static procedure (lateral and torsional provisions) of current major building codes, or the modal analysis procedure. These results have been compared with the response of equivalent symmetric (reference) building models designed according to the same procedures. The results presented in Figures 5–9 show the distributions of the element ductility demand, μ, over the height of the frame located at the stiff edge (element 1 in Figures 1 and 2). From earlier studies of the inelastic torsional response of single-storey buildings[2–4,16] it is evident that in stiffness asymmetric buildings, the stiff-edge element can be subjected to excessive additional ductility demand compared to symmetric or torsionally balanced systems, and that certain codes do not at present specify lateral strength distributions which limit this additional demand to acceptable values. In contrast, elements at the flexible edge (element 3) may be subjected to excessive inelastic deformation of up to three or even four times that of symmetric systems[2,4,26] but are not generally subject to any additional ductility demand. Hence, in the present study attention is focused on the additional ductility demand of the stiff-edge element, since it is this element which is, in general, more vulnerable to earthquake damage due to excessive cyclic yielding.[23] The results shown in Figures 5 and 6 relate to 3-storey and 5-storey buildings with uncoupled fundamental lateral periods of 0·3 sec and 0·5 sec, respectively, and having an intermediate stiffness eccentricity of $e_s = 0{\cdot}2b$. Figures 7–9 present the results for the 8-storey building with fundamental uncoupled period of 1·0 sec, and having small, intermediate and large stiffness eccentricities with values $e_s = 0{\cdot}1b$, $0{\cdot}2b$

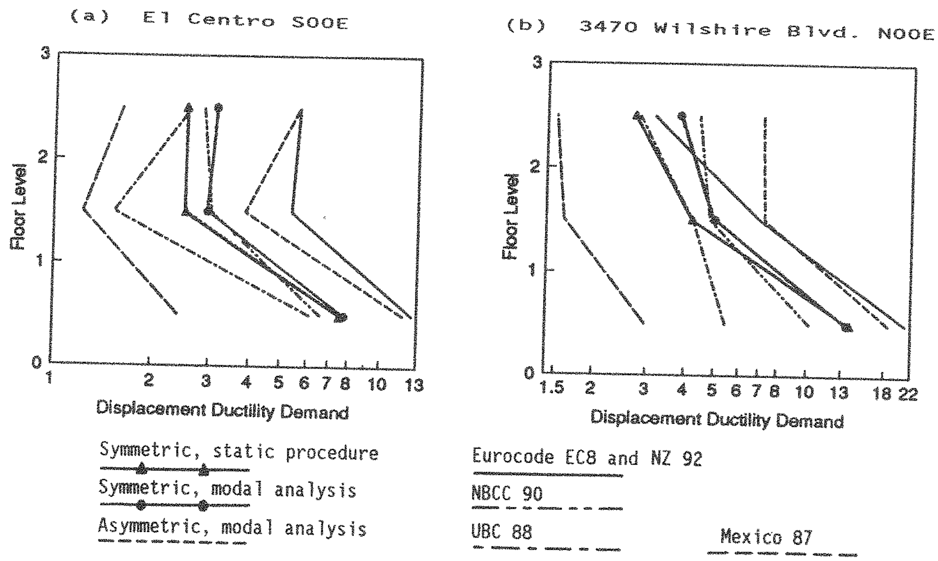

Figure 5. Peak displacement ductility demand of element 1 of the 3-storey model having intermediate stiffness eccentricity ($e_s = 0{\cdot}2b$)

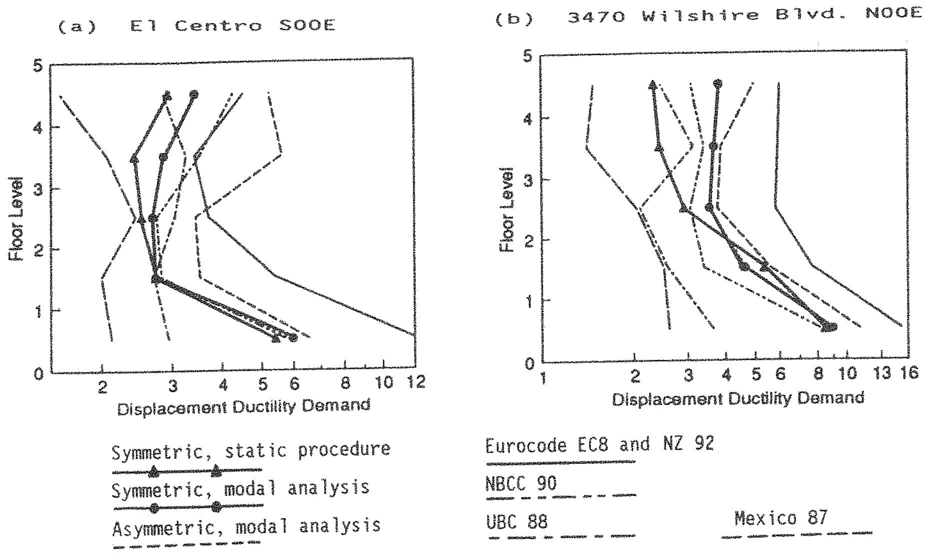

Figure 6. Peak displacement ductility demand of element 1 of the 5-storey model having intermediate stiffness eccentricity ($e_s = 0{\cdot}2b$

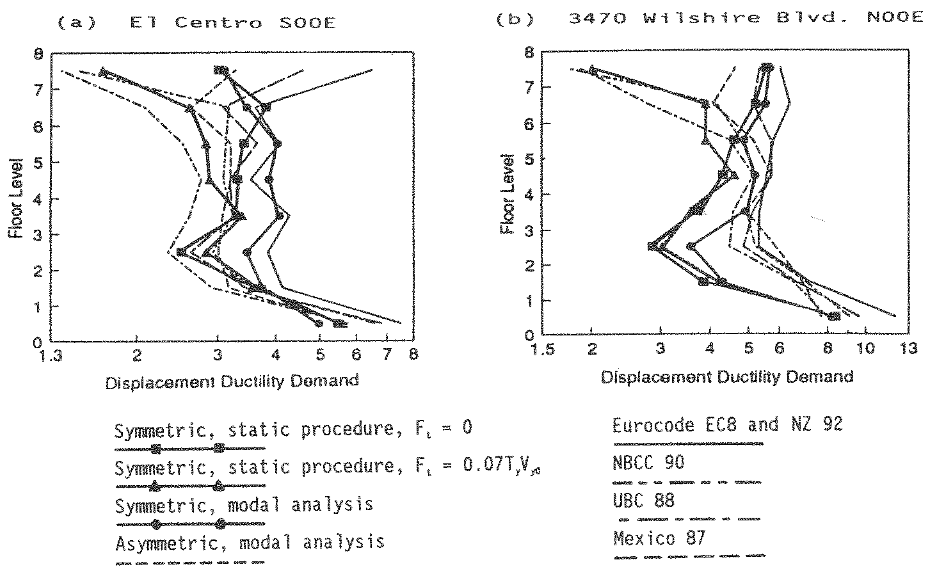

Figure 7. Peak displacement ductility demand of element 1 of the 8-storey model having small stiffness eccentricity ($e_s = 0{\cdot}1b$)

and $0{\cdot}3b$, respectively. In all cases the uncoupled torsional to lateral frequency ratio, Ω, is taken to be 1·0, a the force reduction factor $R = 4$.

Symmetric (reference) systems

In determining the design storey shears of the multistorey symmetric frame buildings (which are th distributed uniformly amongst the three lateral load-resisting elements), four approaches have be employed. These are the modal analysis procedure, the linear and parabolic vertical distributions of the ba shear (the latter for the Mexico 87 code only) and, finally, a concentrated force at the top of the building pl a linear distribution of the remainder of the base shear over the height. The latter is applied generally for t

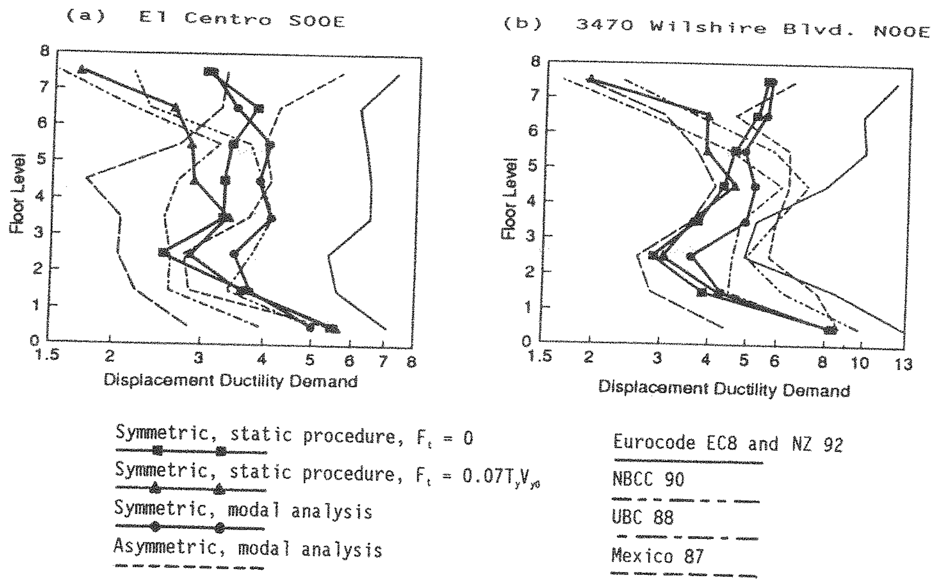

Figure 8. Peak displacement ductility demand of element 1 of the 8-storey model having intermediate stiffness eccentricity ($e_s = 0{\cdot}2b$)

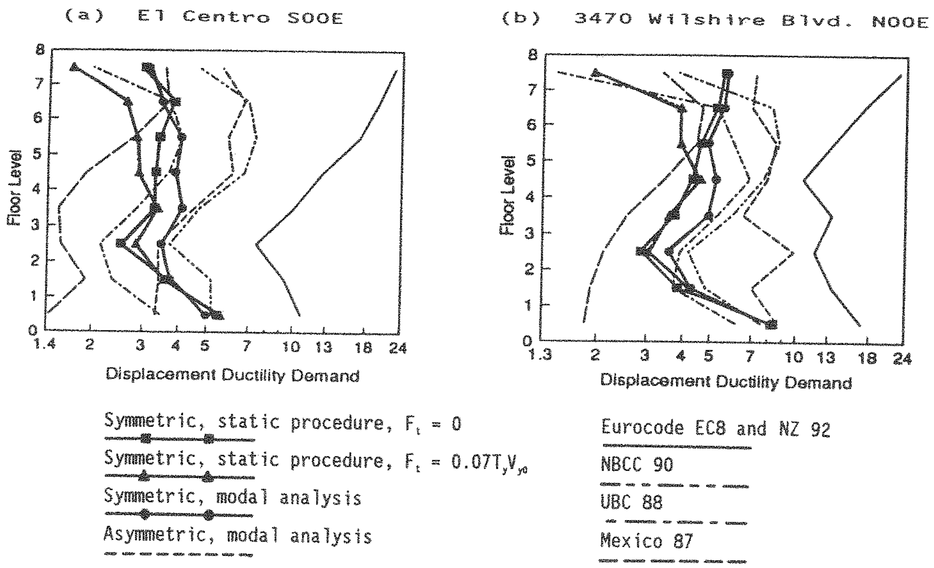

Figure 9. Peak displacement ductility demand of element 1 of the 8-storey model having large stiffness eccentricity ($e_s = 0{\cdot}3b$)

8-storey model only, with the exception of the NZ 92 code. For clarity, the results in Figures 5 and 6 (corresponding to relatively short-period systems) have been presented only for the first three approaches. To facilitate comparison, the storey shear at the first storey is kept constant, and equal to that from the equivalent static force method. Neither the horizontal nor the vertical design strength distributions have been altered by this normalization procedure.

The results for the 8-storey symmetric model (Figures 7–9) show clearly that the top concentrated force reduces significantly the displacement ductility demand of the upper storeys compared with the results from modal analysis or the linear distribution of base shear. This is because the design storey shears of the upper storeys are increased substantially due to the application of the top force. For symmetric buildings, the modal analysis procedure is adequate, resulting in a nearly constant ductility demand over the building height, wit values close to the design or target displacement ductility of 4, except for higher values in the first storey. Th results for the linear distribution of base shear lie between those for modal analysis and the static procedu with a linear distribution of the base shear plus a top force, being close to the former in the upper storeys an approaching the latter in the lower storeys (Figures 7–9). The above observations suggest that the stat procedure with a linear distribution of the base shear plus a top force (for fundamental periods exceedin 0·I sec) is the preferred approach, being reasonably conservative and simple to apply. The linear distributio of base shear with no top force is adequate for the shorter-period buildings, since the tendency for ductiliti to increase in the upper levels of the building is less apparent for these cases (Figures 5 and 6).

Code torsional provisions and modal analysis

The results shown in Figures 5, 6, 8 and 9 indicate that the Eurocode EC8 provision, which allow relatively large reductions in element 1 design strength when the stiffness eccentricity is intermediate c large,[8] leads to a very significant increase in the displacement ductility demand of element 1 in building having $e_s = 0{\cdot}2b$ or $0{\cdot}3b$. This provision is also somewhat non-conservative even for buildings with a sma eccentricity ($e_s = 0{\cdot}1b$, Figure 7). Note from Table I that the secondary design provisions of the EC8 and N 92 codes are identical when the accidental eccentricity component (for both symmetric and asymmetr systems) is neglected. Hence, the dynamic response of the stiff-edge element in systems designed according t the NZ 92 static torsional provisions is similar to that shown in Figures 5–9 for systems designed accordi to Eurocode EC8, except in the upper storeys, where the top force employed by the NZ 92 code reduc significantly the peak element ductility demand, as discussed above. In contrast, the UBC 88 provision which do not allow any reduction in the strength capacity of resisting elements at the stiff edge, are adequa in all cases. The modal analysis procedure, applied to determine the vertical distribution of lateral force a the torsional effect, is adequate only for asymmetric buildings having a small eccentricity (Figure 7), and cases with intermediate or large eccentricity leads to excessive ductility demand in element 1 (see Figure 9, f example). The same comments apply to the NBCC 90 provisions, which generally give results which a similar to those obtained by modal analysis.

The Mexico 87 code provisions are reasonably conservative for buildings with small eccentricity but a overconservative for columns in the lower storeys of frame element 1 in buildings having intermediate large eccentricity. However, Figures 8 and 9 show that for relatively long-period asymmetric frame buildin with $e_s = 0{\cdot}2b$ or $0{\cdot}3b$, the Mexico 87 provisions result in an increasing ductility demand in element 1 wi storey level, the values in the upper storeys approaching those of the corresponding symmetric multistor buildings designed either in accordance with the static procedure, with a linear distribution of the base shea or by the modal analysis procedure. Therefore, for the upper storeys of such buildings, the Mexico 87 co provisions are no longer overconservative as in single-storey asymmetric buildings,[26] although they do le to large increases in the total strength in all storeys. Differences between the results for single-storey a multistorey buildings can be attributed to the approach of a simple linear distribution of the base shear ov height as employed by the Mexico 87 code, and the increased influence of higher modes on the inelas response of the upper storeys in medium- to long-period multistorey buildings.

Figures 5–9 also show that, generally, the inelastic response of the upper storeys increases with increasi values of the stiffness eccentricity and the fundamental uncoupled lateral period of the building. Therefore, medium- and long-period multistorey asymmetric frame buildings, whether designed on the basis of t considered code provisions or the modal analysis procedure, the columns in the upper storeys of frames at t stiff edge are more vulnerable to structural damage than those in the lower storeys.

DISCUSSION ON THE APPLICATION OF MODAL ANALYSIS IN DESIGN

The results of this study indicate that the modal analysis procedure may be non-conservative for the inelas design of multistorey regularly asymmetric buildings having an intermediate or large stiffness eccentricity. gives reasonable results only for multistorey symmetric buildings and those asymmetric buildings in whi the stiffness eccentricity is small ($e_s \leqslant 0{\cdot}1b$). Strictly speaking, the modal analysis method is applicable to the analysis of linear elastic systems only. The extension of this procedure to the analysis of inelastic systems, such as earthquake-resistant buildings responding to strong earthquake ground motions, is based on the assumption that non-linear structural response can be determined to an acceptable degree of accuracy by linear analysis of the building.[27] However, period elongation and the shifting of the centres of rigidity in asymmetric buildings excited well into the inelastic range result in inelastic behaviour fundamentally different from that predicted by linear elastic theory. The results presented in this paper indicate that the above assumption is, therefore, no longer valid in the case of highly asymmetric buildings.

Similarly, a study of the inelastic earthquake response and the design of set-back plane frames[28] showed that the modal analysis procedure and the UBC 88 static force procedure result in similar values and distributions of the column curvature ductility demand, and that both these methods are inadequate to prevent concentration of structural damage in columns near the set-back level. Hence, it was concluded that there is no apparent advantage in the use of the modal analysis procedure to design such structures. Instead, a modified static force procedure was proposed, that amplifies design forces in the tower in order to prevent damage concentration in the set-back frames. It appears from the results of the present study that the efficient design of asymmetric frame structures may also be best achieved by the application of a modified static force procedure, provided adequate criteria are established to classify (and possibly limit) the asymmetry or degree of irregularity in buildings. Such a proposal is presented in Reference 8. Partly in recognition of the limitations of the modal analysis procedure for the design of buildings to withstand severe ground motions, the Canadian code NBCC 90 has removed the option of determining the base shear by modal analysis as an alternative approach to the static force procedure. In addition, the Mexico 87 code does not permit the modal analysis procedure to be used to consider torsional effects, namely, the horizontal distribution of the earthquake lateral load.

CONCLUSIONS

Based on an asymmetric multistorey frame building model, this paper has investigated inelastic torsional response effects and has enabled a general evaluation to be made of the adequacy of the provisions of current seismic building codes and the modal analysis procedure in accounting for increased inelastic element ductility demand in such buildings. The following conclusions and recommendations can be made based on the study presented:

1. Unlike elastic studies, a single-storey structural model is not on its own sufficient to investigate completely the inelastic torsional effects in multistorey regularly asymmetric buildings. A multistorey model should, therefore, be employed for this purpose; the present study considers an idealized shear building model representative of frame buildings with rigid floor diaphragms.
2. The inelastic response of the upper-storey columns of frames at the stiff edge of asymmetric buildings increases significantly with the building's fundamental uncoupled lateral period and the magnitude of the stiffness eccentricity.
3. In equivalent static design, the application of a concentrated force at the top of a building reduces significantly the inelastic response of the upper-storey columns compared to the modal analysis procedure and a simple linear distribution of the design base shear. This top concentrated force is essential to control the inelastic response of the upper-storey columns in medium-period and to some extent for short-period asymmetric buildings, but is not included in certain current building codes (Eurocode EC8 and Mexico 87, for example) except for tall, long-period structures.
4. The application of the equivalent static torsional provisions of Eurocode EC8, NZ 92 and to a lesser extent NBCC 90 to the inelastic design of asymmetric buildings may lead to non-conservative estimates of response, particularly for structures with intermediate or large stiffness eccentricity. In these cases, the critical stiff-edge elements are vulnerable to excessive additional ductility demand and, hence, may suffer significantly more severe structural damage than in corresponding symmetric buildings.

5. Unlike other codes, UBC 88 does not allow any reduction of element design loading due to the favourable effects of torsion and, therefore, leads to no additional ductility demand in asymmetric buildings.
6. The provisions of the Mexico 87 code give special protection to the stiff-edge element and as a result are overly conservative for this element in short-period regularly asymmetric buildings and for the lower storey columns of this element in medium-period buildings. However for relatively long-period buildings, whilst the Mexico 87 provisions again overestimate the strength demand of the lower storey columns, the reverse is true for the upper-storey columns of the stiff-edge element.
7. Regularly asymmetric buildings excited well into the inelastic range may not be conservatively designed using linear elastic modal analysis theory. Particular caution is required when applying this method to the design of frame elements on the stiff side of CR in highly asymmetric structures. It is recommended that an alternative method of design leading to more consistent and conservative estimates of the peak inelastic response be formulated for this category of structures (and for other vulnerable structural forms) by means of an improved static force procedure. A proposal based on this approach has been presented in a subsequent study.[8]

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.

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