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SEISMIC TORSIONAL RESPONSE AND DESIGN PROCEDURES FOR A CLASS OF SETBACK FRAME BUILDINGS

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SUMMARY

This paper presents results of an analytical study of the inelastic earthquake torsional response of a class of setback frame buildings. In the first part of the study, the modal response spectrum analysis procedure is utilized to determine the yielding strengths of structural members in an idealized but representative setback frame building model. Results are then presented for the inelastic dynamic response of this setback building model subjected to an ensemble of six earthquake ground motions. The results indicate that the modal response spectrum analysis procedure is inadequate for preventing excessive response leading to concentration of damage in vulnerable structural members, such as those in the tower near the notch and those in the base (the part of the structure below the tower) near the perimeter at the opposite side of the tower. The second part of the study develops a modified equivalent static force procedure for strength design of such setback frame buildings. Response analyses show that the proposed procedure results in improved and satisfactory inelastic performance of the selected class of setback frame buildings, having a wide range of realistic configurations.

INTRODUCTION

Setback buildings are a common form of construction in civil engineering practice, due to both functional and architectural requirements. Setback shapes may be introduced for stylistic reasons, whilst at the same time energy conservation requirements may result in a functional interest in setbacks for daylighting purposes. As a result, urban planning regulations in some cities dictate the need for vertical setbacks in tall buildings.[1] Setback buildings represent a type of irregular configuration in which discontinuity in the distribution of mass, stiffness and strength occurs at the line of setback, namely the notch (Figure 1). The abrupt change in the vertical distribution of these key structural properties often results in irregularities in plan, giving rise to horizontal torsional eccentricities and stress concentrations near the notch. Examples of the poor performance of setback buildings have arisen in past earthquakes.[2,3] In most cases, the poor performance has been attributed to torsional effects and to damage concentration due to inelastic action near the notch or in the tower.[4,5]

In a recent study, Shahrooz and Moehle[6] investigated the inelastic earthquake response of setback plane frame buildings. Their study focused on the response parallel to the setback so that torsion was not a consideration. It was concluded that the linear elastic modal response spectrum analysis procedure (henceforth referred to as the modal analysis procedure) does not lead to notably improved inelastic seismic performance of setback plane frames, compared with that of such frames designed by the equivalent static force procedure (henceforth referred to as the static force procedure). They further concluded that for certain tower to base area ratios and tower to base height ratios, both procedures are inadequate to prevent damage concentration near the notch and in the tower.

Building codes[7–11] generally prescribe the above two approaches, namely the modal analysis procedure and the static force procedure, as alternatives for the analysis of structures under earthquake loading. To restrict the application of the static force procedure, codes also specify regularity conditions (in the vertical

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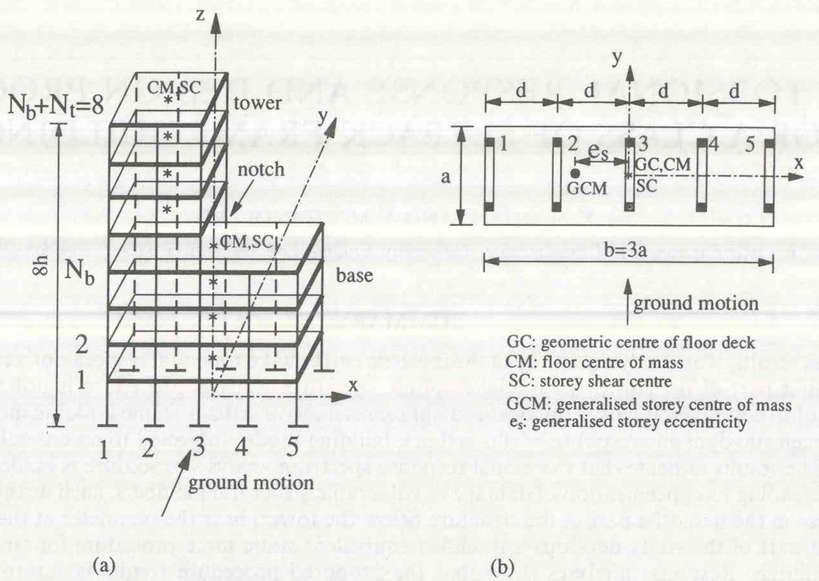


Figure 1. (a) Idealized setback frame building model, and (b) plan view of the base

and horizontal planes) to classify buildings into two categories, namely regular and irregular. The static force procedure is applicable only to regular buildings, whilst the modal analysis procedure can be employed for the analysis of buildings in both categories. Having vertical geometric irregularity, setback buildings are usually classified as irregular by building codes. For instance, UBC 91[7] specifies that buildings having setbacks wherein the plan dimension of the tower is less than 75 per cent of the base should be classified as vertically irregular buildings and consequently only a dynamic method, usually consisting of the modal analysis procedure, is permitted to be employed for design purposes.

Building codes allow structural members to be loaded well into the inelastic range to utilize their deformation ductility and energy dissipation capacities for ultimate limit state design. However, an explicit inelastic earthquake response analysis is usually not required by codes. Strictly speaking, the modal analysis procedure is only applicable to structures responding elastically to earthquake ground motions. Its applicability to inelastic systems is questionable. In a recent study, the authors[12] have demonstrated that the modal analysis procedure is not appropriate to ensure satisfactory inelastic performance of regularly asymmetric frame buildings. Correspondingly, there is a requirement to evaluate the applicability of the modal analysis procedure to setback buildings responding inelastically to strong earthquake ground motions.

Although torsion has been recognized as one of the sources giving rise to poor seismic performance of setback buildings, analytical and experimental studies on their torsional seismic response are few. Tso and Yao[13] in their recent study have investigated the elastic seismic load distribution in setback frame buildings considering torsion. However, systematic theoretical studies on inelastic torsional effects in setback buildings have not been presented in the literature. The present study intends, firstly, to investigate the influence of a setback on the inelastic earthquake response of a class of setback frame buildings, secondly to evaluate the adequacy of the modal analysis procedure for the selected class of setback frame buildings excited well into the inelastic range, and finally to propose a new static force design procedure to improve the inelastic seismic performance of this class of setback frame buildings.

SETBACK FRAME BUILDING MODEL

The coupled lateral-torsional response of setback buildings depends on the relative proportion and size of the separate parts of the building, namely the tower and the base. In addition, the symmetry or asymmetry in plan of the tower and the base affects the torsional response. If the tower, or the base, or both are asymmetric in plan, a considerable number of parameters are needed to quantify the plan asymmetry of the structure as a whole. This results in greater complexity of structural behaviour and dynamic analysis. As a first step towards analytical investigations of this research topic, this study focuses on a relatively simple building system in which the tower and base are themselves independently symmetric in plan. Torsional response therefore arises exclusively due to the eccentric location of the tower relative to the base.

Assumptions in the model definition

A five-element, eight-storey medium-rise setback frame building model, as shown in Figure 1, has been employed as the analytical model. The model is assumed to have the following characteristics:

(1) The distribution of mass, stiffness and strength of the model is symmetric about the global x-axis but is asymmetric about the global y-axis due to the setback. The tower and base each have symmetric distributions of mass and stiffness about two horizontally perpendicular axes.
(2) The floors are rectangular in plan, and consist of perfectly rigid diaphragms (both in-plane and in flexure) supported on massless columns. The horizontal dimensions of the tower and base, measured parallel to the y-axis, are identical (equal to a). The N_b floors within the base have a representative aspect ratio a/b equal to 1/3 (Figure 1). The tower to base area ratio, R_a, defined by Shahrooz and Moehle[6] as the ratio of the horizontal dimension of the tower parallel to the x-axis to the corresponding dimension of the base, is the parameter utilized to quantify the horizontal proportion of the tower relative to the base. Likewise, the tower to base height ratio, R_h, which in concept is similar to the level of the setback as defined in Reference 6, is defined as the ratio of the height of the roof above the notch (namely the height of the tower) to the height of the notch above the top level of the footings (namely the height of the base). The parameter R_h characterises the vertical proportion and size of the tower relative to the base. For the selected five-element eight-storey setback building model, R_a has been assigned three realistic values: 0·25, 0·5 and 0·75. Additionally, four values of R_h are considered in this study; they are 0·6 (3:5, $N_b = 5$), 1·0 (4:4, $N_b = 4$), 1·67 (5:3, $N_b = 3$) and 3·0 (6:2, $N_b = 2$). These values of R_a and R_h cover a wide range of setback configurations encountered in engineering practice.
(3) The mass of the model is lumped at, and assumed to be uniformly distributed over, the floors. Therefore, the centre of mass, CM, of a floor deck coincides with the geometric centre, GC, of the floor. All floors in the base have the same mass, m_b, and radius of gyration, r_b, the latter taken about the vertical line passing through CM of the floor under consideration. Likewise, all floors in the tower have the same mass, m_t, and radius of gyration, r_t. Consequently, the centres of mass of all floors in the base lie on a vertical line and those in the tower lie on another vertical line, as illustrated in Figure 1(a).
(4) There are five plane frame load resisting elements in the base oriented in the lateral direction, parallel to the y-axis. The number of load-resisting elements in the tower depends on the parameter R_a and has three realistic values: 2, 3, and 4, corresponding to R_a values of 0·25, 0·5 and 0·75, respectively. Transverse frames are excluded (see discussion below). Frame elements are assumed to have stiffness in their acting planes only.
(5) All storeys are assumed to have identical storey height, h. The elastic cross-section flexural stiffnesses of columns and beams are uniform both vertically along the height of the building and horizontally across the individual frames. It is further assumed that the flexural stiffness of beams is very high relative to that of columns, so that each frame can be considered as a 'shear beam' for computational purposes. Therefore, this setback frame building model belongs to a special class of irregular buildings, commonly described as 'shear buildings',[14] in which the locations of storey shear centres, SC, as shown in Figure 1(a), are load independent and can be determined on a storey-by-storey basis. Figure 1(a) indicates that the floor centres of mass of the chosen model do not lie on one vertical line, and neither do the storey shear centres. As a result, setback buildings are a type of irregular building for which the static force procedure is not permitted by building codes as a method of structural analysis. In design practice, column sizes of multistorey buildings may be slightly tapered vertically and may vary

horizontally from frame to frame, resulting in either the tower or base (or both) being plan-asymmetric. However, this category of setback frame buildings does not belong to the class of buildings considered in this study and hence is beyond the scope of this paper.

(6) Beams are assumed to be rigid, having very high yielding strengths compared with those of columns. Columns are modelled using a single-component element model.[15] Yielding of columns may take place only in concentrated plastic hinges at column ends. Since the effect of the axial force on the yielding moment of columns is small and the P–Δ effect is minimal in medium-rise frame buildings, the interaction between bending moment and axial force in columns has been ignored for the sake of simplicity. Furthermore, a simplified approach has been used to account for the P–Δ effect in columns. Hence, yielding moments of columns are set equal to the bending moments at column ends obtained from structural analysis, as described in the following sections of the paper. Second-order bending moments in columns arising due to the P–Δ effect are considered by adding geometric stiffness to the column stiffness, using the axial forces induced by gravity loads. The moment–rotation relationship of plastic hinges is assumed to be similar to the Takeda-type model including 5 per cent strain hardening and stiffness degradation.

(7) The foundation is assumed to be rigid. Therefore, soil–structural interaction effects have been neglected. The ground motion is considered to be unidirectional, acting parallel to the y-axis and is identical at all points over the foundation plan.

Discussion of structural idealization

Transverse elements (those oriented perpendicular to the direction of ground excitation) are excluded in the definition of the setback frame building model. This simplification is supported by recent studies[16,17] employing a single-storey building model having resisting elements in both principal directions and subjected to bidirectional ground motion input. References 16 and 17 have concluded that unidirectional analysis of inelastic torsional effects, considering only the lateral load-resisting elements and the corresponding unidirectional earthquake ground motion input, gives valid and reasonably accurate results compared with those obtained by fully bidirectional analysis.

The rigid beam assumption used in the setback frame building model inevitably leads to a column sidesway mechanism, in which plastic hinges are developed at column ends rather than at beam ends, in the frame elements. Although the capacity design procedure[18] requires a strong-column weak-beam design approach in order to have plastic hinges formed at beam ends, the idealized beam sidesway mechanism may be difficult to achieve in practice due to a number of reasons, as cited in previous publications by the authors[12,19] and in Reference 20. As a result, plastic hinges inevitably form at column ends. Therefore, the possible column sidesway mechanism, whilst undesirable, should be considered as the worst possible scenario, in the sense that the ductility demands of columns in frames of this type are larger than when a beam sidesway mechanism occurs. Also, a column hinging mechanism permits a simpler model to be adopted for analysis. Hence, for these reasons, as a first step towards understanding the inelastic earthquake torsional response of setback frame buildings and the global distribution of additional inelastic response amongst the lateral resisting frames, the idealised model employed in this paper is considered appropriate. The effect of the capacity design procedure on the inelastic earthquake torsional response of setback frame buildings is the subject of ongoing research and is beyond the scope of the present paper.

MODEL PROPERTIES AND THE MODAL ANALYSIS PROCEDURE

The eight-storey setback frame building model is assumed to have a representative fundamental lateral period of one second when torsion is ignored. A torsionally uncoupled symmetric eight-storey frame building model without a setback and having the same fundamental lateral period as that of the setback frame building model is defined as the reference system, in which torsion and setback effects do not arise. The cross-section flexural stiffness of columns in the reference model is also assumed to be uniform both vertically along the height and horizontally across the resisting elements. For all models considered in this paper, the viscous damping for the first two modes, one dominated by translation parallel to the y-axis and the other dominated by torsion, is taken to be 5 per cent of critical damping. Therefore, a Rayleigh damping matrix has been obtaned by assuming the damping matrix to be proportional to both the mass and the tangent stiffness matrices.

The total lateral storey stiffness is K_{yi} ($i = 1, 2,\ldots, N_b,\ldots, N_b + N_t$, where N_b and N_t are the number of storeys in the base and tower respectively, and $N_b + N_t = 8$). Since the cross-section flexural stiffness of columns is uniform vertically, K_{yi} is identical for storeys in the tower ($i = N_b + 1, N_b + 2, \ldots, N_b + N_t$) and also for those in the base ($i = 1, 2,\ldots, N_b$). Given the tower to base area ratio R_a, which determines the relationship between the total lateral storey stiffnesses in the base and tower, together with the tower to base height rato R_h, and assuming the fundamental lateral period T_y ($= 2\pi/\omega_y$, ignoring torsion) to be one second, K_{yi} can be determined by solving the following eigenproblem, considering the lateral displacements of the floors only:

$$[K_y]\{v\}_i = \omega_y^2[M]\{v\}_i \tag{1}$$

where $[K_y]$ is the global lateral stiffness matrix, $[M]$ the global mass matrix, and $\{v\}_i$ the mode shape corresponding to the ith lateral vibration mode. The flexural stiffnesses of columns are calculated by distributing K_{yi} equally to the columns in the storey being considered.

The frame elements in the base are identical and equally spaced, with a distance d between two adjacent elements [Figure 1(b)]. Since the aspect ratio a/b of the base is taken to be 1/3, the torsional to lateral frequency ratio of the base is approximately 1·2, which is a value common in engineering practice and represents moderately torsionally stiff buildings.

Two equivalent approaches proposed by Tso,[21] namely the generalized floor eccentricity and the generalized storey eccentricity, have been widely accepted by researchers to describe plan-asymmetry in multistorey buildings.[12,14,22,23] Storey eccentricity is defined as the horizontal distance between the resultant of all lateral forces acting above the storey being considered and the shear centre of that storey, whilst floor eccentricity is defined as the horizontal distance between the floor centre of mass and the generalized centre of rigidity at the floor being considered. The generalized floor centres of rigidity are the set of points at floor levels such that when a given set of lateral loads are applied at these points, the floors translate only, without rotations. In the present paper, the storey eccentricity is employed since it is much less sensitive to lateral load distribution changes[21] and therefore is a better measure of structural asymmetry than the floor eccentricity. Given R_a, R_h and the vertical distribution of the lateral loads, the following may be determined, for each storey: the location of storey shear centre, x_s, the location of the resultant of all lateral forces acting above the storey under consideration, x^*, and the storey eccentricity e_s ($= x^* - x_s$). These can be determined by using widely available structural analysis programs.

For the class of setback frame buildings considered in this study, the above calculations are straightforward and can be carried out manually on a storey-by-storey basis without the need for computer programs. For instance, if $R_a = 0{\cdot}5$, $R_h = 1{\cdot}0$, and if the lateral loads are distributed in accordance with the UBC 91[7] and NBCC 90[8] codes, namely 7 per cent of the total lateral load is applied at the top floor level and the remainder is distributed linearly throughout the building height, results for x_s, x^* and e_s are obtained as in Table I. This table shows that the storey eccentricity in all storeys in the tower is equal to zero. Therefore, the tower may be viewed as a symmetric system excited by both lateral and rotational motions at the setback level, arising from the coupled lateral-torsional response of the base. Table I also shows that, in the base, the storey shear centres coincide with the geometric centres of floor decks, whilst the resultants of lateral loads have offsets, equal to the storey eccentricities, from the storey shear centres. Hence, the base may be viewed as a 'mass' eccentric system if the location of the resultant of lateral loads acting above a storey is interpreted as the generalized centre of mass of that storey (GCM), as illustrated in Figure 1 (b).

The 5 per cent damped Newmark–Hall median elastic response spectrum, scaled to a peak ground acceleration of $0{\cdot}3g$ and shown in Figure 2, has been employed as the design spectrum representing the elastic strength demand of a SDOF system. The inelastic design base shear force corresponding to the static force procedure, V_0, is calculated simply by dividing the elastic base shear force, V_e, by a force reduction

Table I. Location of storey shear centres and storey eccentricities ($R_a = 0{\cdot}5$, $R_h = 1{\cdot}0$)

Storey	Location of shear centre x_s	Location of resultant of lateral forces above (x^*)	Storey eccentricity e_s
8	$-d$	$-d$	0
7	$-d$	$-d$	0
6	$-d$	$-d$	0
5	$-d$	d	0
4	0	$-0{\cdot}787d$	$-0{\cdot}787d$
3	0	$-0{\cdot}678d$	$-0{\cdot}678d$
2	0	$-0{\cdot}62d$	$-0{\cdot}62d$
1	0	$-0{\cdot}595d$	$-0{\cdot}595d$

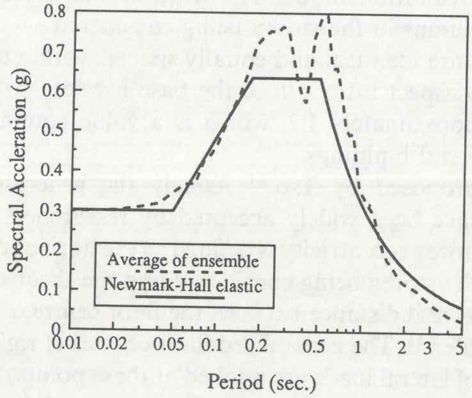


Figure 2. Comparison of the average response spectrum of the ensemble of 6 selected records with the Newmark–Hall elastic design spectrum

factor R ($R = 4$, a value typical for ductile moment resisting frame buildings designed by code procedures to respond well into the inelastic range). Hence

$$V_0 = \frac{V_e}{R} = \frac{S_a(\xi, T_y)\,W}{Rg} \tag{2}$$

in which $S_a(\xi, T_y)$ is the value of the elastic design spectrum at the model's fundamental lateral period T_y (taken as 1·0 sec), ξ is the damping ratio (taken as 5 per cent), g is the gravitational acceleration, and W is the total weight of the building model.

The modal analysis procedure is used to carry out dynamic analysis to determine member strengths, namely the yielding moments of columns at the various storeys, for both the setback frame building model and the reference model. The procedure used in this paper can therefore be summarized by the following sequence of steps:

Step A: An eigenproblem is solved to obtain the periods and vibration modes of the structure, considering two degrees-of-freedom for each floor, namely the lateral displacement of CM, v, and the rotation about CM, θ, as follows

$$[K]\{\phi\}_i = \omega_i^2 [M]\{\phi\}_i \tag{3}$$

in which $\{\phi\}_i^T = \{(v_1)_i, (\theta_1)_i, (v_2)_i, (\theta_2)_i, \ldots\}$ is the ith vibration mode, and $[K]$ and $[M]$ are the global stiffness and mass matrices, respectively.

Step B: The maximum modal response quantities of interest are calculated. These are associated with the lowest few modes, based on the above-mentioned elastic design spectrum.

Step C: The effective modal weight W_{ei} that is participating in the global y-direction, associated with the ith mode is calculated:

$$W_{ei} = \frac{(\sum_{j=1}^{N_b+N_t} W_j(v_j)_i)^2}{\sum_{j=1}^{N_b+N_t} W_j(v_j)_i^2} \tag{4}$$

and the elastic base shear force associated with the ith mode is determined from

$$V_i = \frac{S_a(\xi, T_i) W_{ei}}{g} \tag{5}$$

in which T_i is the ith modal period. The number of modes considered is such that the sum of the effective modal weights exceeds 99 per cent of the structure's total weight.

Step D: The combined elastic modal base shear force, V_m, is determined by combining the individual elastic modal base shear forces using the CQC modal combination rule.

Step E: All elastic member forces associated with individual modes obtained in Step B are scaled down by the same proportion, namely by a factor equal to V_0/V_m, such that the combined scaled elastic modal base shear force V_m is equal to the inelastic base shear V_0 calculated using the static force procedure, as described above. The objective of using this scaling factor V_0/V_m, rather than $1/R$ as in the static force procedure, is to ensure that the combined inelastic modal base shear force is equal to, and not lower than, the inelastic static base shear force, V_0.

Step F: The inelastic design member forces and other quantities of interest are obtained by combining the scaled modal quantities of each contributing mode using the CQC rule.

It should be noted that although Steps A–D and Step F are commonly incorporated into the design procedures of building codes, various national and regional codes specify different minimum values for the combined inelastic modal base shear force in Step E. For example, NBCC 90[8] requires that the combined inelastic modal base shear force should be equal to the static inelastic base shear. UBC 91[7] specifies that the former should be at least 80 per cent of the latter for regular buildings and at least 90 per cent for irregular buildings. In NZ 92,[9] the corresponding minimum values are 80 per cent for regular buildings and 100 per cent for irregular buildings. On the other hand, Eurocode 8[10] does not specify a minimum value for the combined inelastic modal base shear force, the scaling factor being effectively equal to $1/R$.

The elastic modal analysis procedure usually results in V_m having smaller values than V_e. Figure 3 illustrates the variation of the ratio V_m/V_e. It is observed that V_m is in order of 65–75 per cent of V_e for the setback configurations considered in this study. The resulting scaling factor V_0/V_m is about 0·35, which is significantly higher than $1/R$ (equal to 0·25). This implies that the requirement that the combined inelastic modal base shear force be equal to the inelastic static base shear force results in much less reduction in the elastic strength demand of resisting elements compared with using the force reduction factor R.

When torsional response arises in building structures, the maximum strength demands of the two edge elements (elements 1 and 5 in Figure 1) do not occur simultaneously. As a result, the sum of element lateral strengths in the first storey is considerably greater than the design inelastic static base shear V_0 if the scaling factor V_0/V_m is used, a property termed overstrength. Figure 4 shows the overstrength factor O_s, defined as the ratio of the sum of element lateral strengths in the first storey to V_0, as functions of R_a and R_h. The results indicate that O_s is substantially greater than unity.

The torsional provisions of building codes account for possible rotational components within the ground excitation pattern, together with uncertainities in the distribution of mass and stiffness, and other accidental effects not accounted for explicitly in design. This is achieved by stipulating that, when applying the modal analysis procedure, the centre of mass at each floor should be displaced from its nominal location in each direction by a distance equal to the accidental eccentricity e_a (specified as either $0{\cdot}1b$ or $0{\cdot}05b$). Since neither the rotational component of the ground excitation nor any of the mentioned uncertainities could be

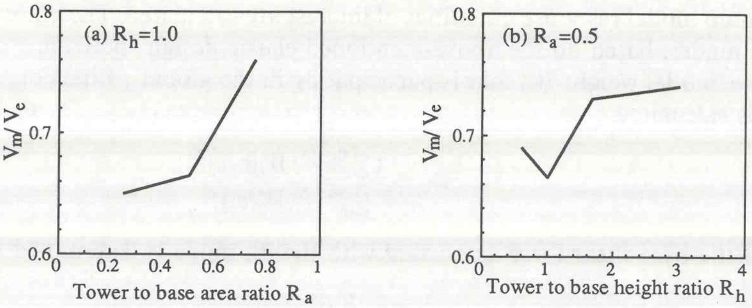


Figure 3. Variation with R_a and R_h of the elastic modal base shear force V_m normalised to the elastic static base shear force V_e

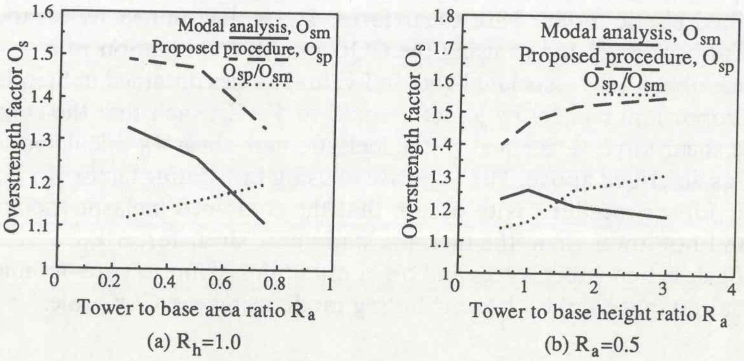


Figure 4. Variation of the overstrength factor O_s with R_a and R_h

appropriately simulated in the presented analytical studies, accidental eccentricity has not been considered in this paper in applying the modal analysis procedure, for either the setback building model or the reference building model. This argument has been further justified in Reference 12. Another view is that since code torsional provisions are applied in their entirety in practice, the codified accidental torsional allowance should be included when determining element design strengths.[17,24] It should be noted that when the modal analysis procedure is employed to determine the element strengths, the code accidental torsional allowance is intended only to account for accidental effects not explicitly considered in the modal analysis procedure. It is not the purpose of codes to use a part of the accidental torsional allowance to compensate for potential unconservatism in the modal analysis procedure, and then to use the remainder to account for accidental effects, as may be the case when code static torsional provisions are used (see References 24 and 25). Since this paper focuses on the evaluation of the modal analysis procedure for inelastic design of setback frame buildings, the codified accidental torsional allowance has been ignored.

GROUND MOTION INPUT AND INELASTIC RESPONSE PARAMETER

In order to minimise the dependency of the inelastic response on the characteristics of individual earthquake records, an ensemble of six strong-motion records from California, Japan and Europe has been selected as the ground motion input. These records have been obtained from rock or stiff soil sites and have intermediate ratios of peak ground acceleration, A, to peak ground velocity, V. The shapes of their 5 per cent damped elastic acceleration response spectra are similar to that of the median Newmark–Hall design spectrum employed in this paper. The mean 5 per cent damped elastic acceleration response spectrum of this ensemble of records, scaled to a common peak ground acceleration of 0·3 g, is superimposed in Figure 2. It is observed that a close match between the two curves in Figure 2 is achieved, particularly in the vicinity of T_y (1·0 sec).

Table II. List of earthquake records used

Earthquake event	Date	Mag.	Site	Epic. dist. (km)	Comp.	Max. acc. A(g)	Max. vel. V(m/s)	A/V	Soil cond.
Imperial Valley, CA	18 May 1940	6·6	El Centro	8	S00E	0·348	0·375	0·93	Stiff soil
Kern County, CA	21 July 1952	7·6	Taft Lincoln School Tunnel	56	S69E	0·179	0·198	0·9	Rock
San Fernando, CA	9 Feb., 1971	6·4	3838 Lankershim Blvd., L.A.	24	S90W	0·15	0·152	0·99	Rock
Whittier, CA	1 Oct., 1987	6·1	Griffith Park Observatory, L. A.	21	N00E	0·124	0·131	0·94	Rock
Near E. Coast of Honshu, Japan	16 Nov., 1974	6·1	Kashima Harbour Works	38	N00E	0·070	0·072	0·97	Stiff soil
Monte-Negro, Yugoslavia	15 April 1979	7·0	Albatros Hotel, Ulcinj	17	N00E	0·171	0·202	0·85	Rock

Hence, the effect of mismatch between the shape of the design spectrum and that of the response spectrum of the ground motions can be excluded as a cause for poor inelastic performance of key structural elements. At $T_y = 1{\cdot}0$ sec, the standard deviation of the spectral accelerations corresponding to the six records is equal to 25 per cent of the mean value. Some relevant parameters of the six records are given in Table II.

The plastic hinge rotational ductility demand (hereafter referred to as the ductility demand) proposed in Reference 6 has been employed in this paper as the inelastic response parameter quantifying the damage sustained by structural members. As a result of the rigid beam assumption, the contraflexural points in columns are at their midheight points. Hence, the ductility demand μ_θ can be expressed as

$$\mu_\theta = \frac{\theta_u}{\theta_y} = \frac{\theta_y + \theta_p}{\theta_y} = 1 + \frac{6EI\theta_p}{M_y h} \tag{6}$$

in which θ_u is the maximum rotation; θ_y the storey drift angle at yield ($= \Delta/h$, $\Delta =$ interstorey drift); θ_p the computed plastic hinge rotation; EI the initial flexural stiffness of the member cross-section; M_y the yielding moment; and h the storey height.

INELASTIC RESPONSE AND EVALUATION OF THE MODAL ANALYSIS PROCEDURE

The inelastic response of the setback frame building model subjected to the selected ensemble of earthquake records has been analysed numerically. The central difference method has been used for the step-by-step time history analysis. The time increment of the numerical integration has been selected to be 1/30th of the smallest elastic modal period of the first three pairs of coupled lateral-torsional vibration modes. This is sufficiently small to ensure stable and accurate numerical integration of the response contributed from at least the first three pairs of coupled vibration modes of the setback frame building model.

The average of six ductility demands corresponding to the ensemble of six earthquake records, $\bar{\mu}_\theta$, has been calculated for all columns in all the frame elements and used to quantify the response distribution throughout the building. Distributions of computed average ductility demands over height for selected elements are presented in Figure 5 for six setback configurations, namely selected combinations of R_a and R_h. The average ductility demands of the reference frame building model without setback, in which $\bar{\mu}_\theta$ is identical for all elements, have also been computed and shown in Figure 5 for comparison. In relation to Figure 5, the

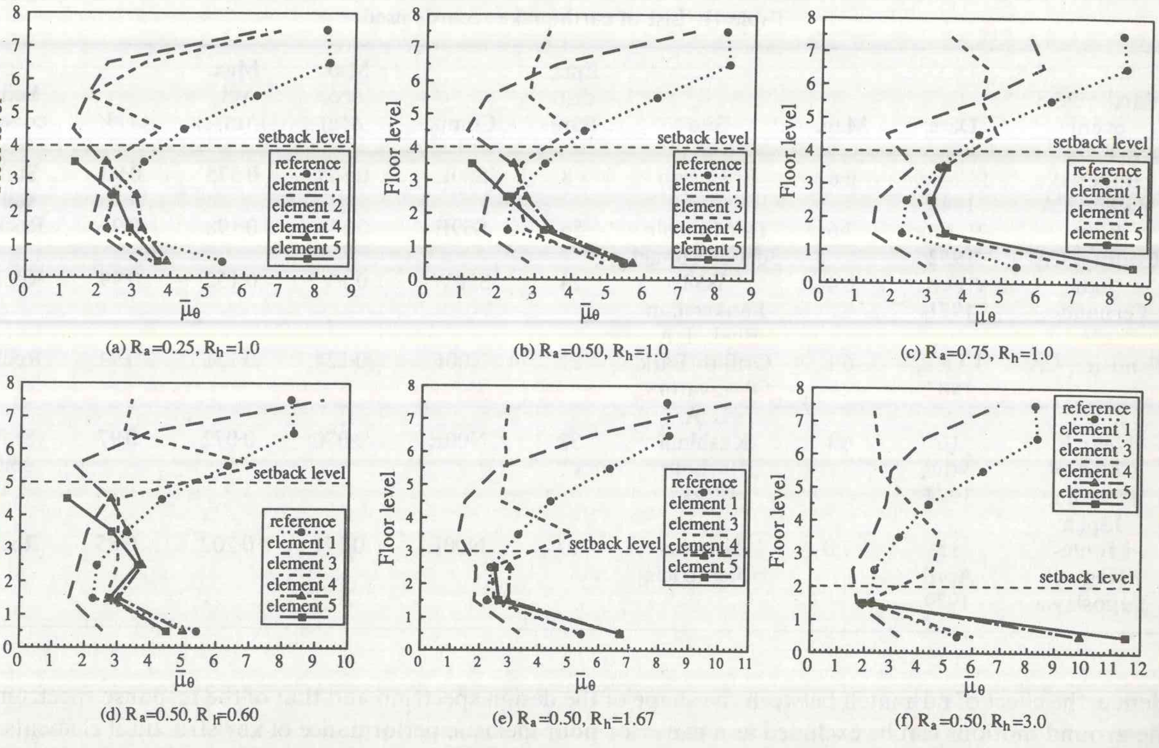


Figure 5. Average ductility demands of selected resisting elements of the setback frame building model designed by the modal analysis procedure

standard deviations of computed ductility demands corresponding to the six records range from 10 to 50 per cent of the mean values, being consistent with the scatter of the spectral accelerations of the six records at 1 sec period.

The tower

Torsional response is induced in the tower by the rotational motion input at the setback level, the latter arising from the torsional response of the base (the part of the structure below the tower). It can be seen from Figure 5 that the edge element nearest to the notch is the critical element where concentration of inelastic action in the tower occurs. For all six setback configurations, there is a dramatic increase in the ductility demands (leading to damage concentration) in columns of these critical elements (short dashed lines in Figure 5), compared with the ductility demand of the same element in the neighbouring storeys. When compared with the ductility demand of the reference system at the same storey level, the ductility demands in columns of the tower near the notch can be significantly higher, by a factor of 1·5 to 2, if the tower to the base height ratio is large [R_h equals 1·67 and 3·0, see Figures 5(e) and 5(f)]. However, if R_h is equal to or less than 1·0, the ductility demands in the above-mentioned columns are lower than that of the reference system at the same storey level.

The base (below the tower)

The eccentric location of the tower gives rise to torsional eccentricities in the base (Table I). As previously mentioned, the base may be viewed as a 'mass' eccentric system in which the eccentric locations of the seismic lateral forces in the tower give rise to corresponding eccentricities in the base. Recent studies[26,27] on the inelastic response of single-storey mass eccentric systems concluded that if the strength distribution amongst the resisting elements is determined in accordance with the torsional provisions of certain static force procedures such as those in the 1976 edition of the Mexican code and the 1992 edition of the New Zealand code, the perimeter element at the same side as the storey shear centre SC (measured from the centre of mass CM) is the critical element which experiences much higher additional ductility demand, compared with the corresponding symmetric reference system. It was also found that the ductility demand for the perimeter element at the opposite side of SC (measured from CM) is significantly smaller than that of the reference system. If the location of the resultant of seismic lateral forces acting above a storey is interpreted as the generalized centre of mass of that storey [GCM in Figure 1(b)], then similar observations can be made on the basis of the results presented in Figure 5. The ductility demand of the perimeter element on the opposite side of the storey shear centres (measured from GCM), namely element 1 in Figure 1(b), as shown by the long-dashed lines in Figure 5, is always smaller than that of the reference system. However, the ductility demand of the perimeter elements on the same side as the storey shear centres [measured from GCM, namely elements 4 and 5 in Figure 1(b)] is greater than that of the reference system in many cases, as shown by the lines with solid square and solid triangle markers in Figure 5, as well as the short-dashed line in Figure 5(c). In particular, the ductility demand of columns in the first storey of elements 4 and 5 can be significantly greater, by a factor of 1·5 to 2, than that of the reference system, for setback configurations in which the tower to base area ratio is large [$R_a = 0{\cdot}75$, see Figure 5(c)] or the tower to base height ratio is large [$R_h = 1{\cdot}67$ and 3, see Figures 5(e) and (f)].

Evaluation of the modal analysis procedure

Ideally, an analysis procedure used in design should make conservative estimates of the strength demands of structural members in setback buildings, such that firstly damage concentration does not occur, and secondly the ductility demand of this type of irregular buildings is smaller than or around the same as that of the corresponding reference building without setback. Results shown in Figure 5 suggest that the modal analysis procedure in the majority of cases fails to achieve these objectives, across a range of setback configurations. This is despite the fact that the setback building model is designed with substantial overstrength, and the strengths of columns are reduced less significantly from the corresponding elastic strength demands (determined by the modal analysis procedure), compared with those obtained by reducing the elastic strength demand by a constant force reduction factor. Consequently, for certain setback configurations, damage concentration occurs in the tower near the notch and in the base near the perimeter at the opposite side of the tower.

Similar conclusions to those cited above have been reached previously by the authors[12] studying the inelastic earthquake response of multistorey frame buildings with regular asymmetry, as well as by Shahrooz and Moehle[6] studying the inelastic earthquake response of setback plane frame structures. It should be noted that most current seismic building codes, except the Mexico 87 code,[11] consider the modal analysis procedure to be applicable to the inelastic design of buildings of any general configuration. The modal analysis procedure is therefore widely employed for the inelastic design of buildings having large stiffness eccentricities or having horizontal or vertical irregularities. Therefore, such code regulations imply that the modal analysis procedure provides a satisfactory method for inelastic design of asymmetric and irregular buildings, without the need to question its limits of application. Results given in this paper and in References 6 and 12 have shown that in many situations this is not the case.

PROPOSED MODIFIED DESIGN PROCEDURE

The earthquake resistant design philosophy adopted by current building codes requires that buildings in seismically prone regions be designed, primarily, to resist a major earthquake without collapse or failure, and secondly, to resist moderate earthquakes with little or no structural damage. Although the modal analysis procedure is known to satisfy the secondary objective referred to above for design of setback buildings responding elastically to moderate earthquakes, it may be deficient with respect to the primary objective of earthquake resistant design stated above, for design of setback frame buildings responding inelastically to a major earthquake. An appropriate solution to the strength design of setback frame buildings may be the development of an improved static force procedure, similar to that proposed by the authors[22] fo design of multistorey asymmetric buildings with regular asymmetry, and that proposed by Shahrooz an Moehle[6] for design of setback plane frame structures.

On the basis of results presented in the previous section, it is evident that the strengths of columns i elements 4 and 5 near the footings, as well as those of columns in the tower near the notch should b increased appropriately, in order to achieve satisfactory inelastic performance of these columns. As a fir step towards the above objective, this paper aims to develop an improved static force procedure for th strength design of the class of setback frame buildings considered herein. The proposed design procedure based on the generalised storey eccentricity concept,[21] and the modified vertical distribution of the inelast static base shear force proposed in Reference 6. The proposed new static procedure may be summarized b the sequence of steps given below. The procedure is given in a form appropriate to design, and it implementation for the purpose of the present study is described in the following section.

Step 1: The setback frame building's fundamental lateral period T_y is estimated (ignoring torsional effects using the simplified, empirical methods suggested in building codes.

Step 2: The design inelastic base shear force V_0 is determined based on the design spectrum specified i building codes and the value of T_y estimated in Step 1. Since torsion has been ignored, V_0 thu calculated is a conservative approximation.

Step 3: The first lateral vibration mode shape is approximated bilinearly, as proposed in Reference 6. Th bilinear first mode shape takes into account effects of the setback on the lateral response of th building and has been derived on the basis of the response of an equivalent 2DOF system representing the response of the base and tower, respectively. The bilinear first mode shape i obtained[6] from

$$\{v\}_1^{\mathrm{T}} = (\alpha_1 h_{1\mathrm{b}}, \alpha_1 h_{2\mathrm{b}}, \ldots, \alpha_1 H_{\mathrm{b}} = x_1, x_1 + \alpha_2 h_{1\mathrm{t}}, x_1 + \alpha_2 h_{2\mathrm{t}}, \ldots, x_1 + \alpha_2 H_{\mathrm{t}}) \quad (7$$

in which $h_{i\mathrm{b}}$ is the elevation of floor i of the base relative to the ground level; $h_{i\mathrm{t}}$ is the elevation o floor i of the tower relative to the setback level; and $\alpha_2 = \delta\alpha_1$.

The coefficient δ employed above is calculated from:

$$\delta = \frac{1 + 2\beta}{5\alpha} \geqslant 1{\cdot}0 \quad (8$$

where α and β are ratios of the generalised mass and stiffness of the 2DOF system, K_2^*/K_1^* anc M_2^*/M_1^*, respectively, and these can be evaluated straighforwardly, according to the formula proposed in Reference 6.

Step 4: A concentrated force $F_{\mathrm{t}} = 0{\cdot}07\, T_y V_0 \leqslant 0{\cdot}25\, V_0$ is applied at the top of the building. The remainder of the inelastic base shear force is then distributed over the height of the building according to the first mode shape:

$$F_j = \frac{M_j (v_j)_1 (V_0 - F_{\mathrm{t}})}{\sum_{j=1}^{N_{\mathrm{b}}+N_{\mathrm{t}}} M_j (v_j)_1}, \quad j = 1, 2, \ldots, N_{\mathrm{b}}, \ldots, N_{\mathrm{b}} + N_{\mathrm{t}} \quad (9)$$

Step 5: The locations of storey shear centres, $(x_s)_k$, $k = 1, 2, \ldots, N_{\mathrm{b}} + N_{\mathrm{t}}$, are determined. Also, given the vertical distribution of the lateral loads determined in Step 4, the location of the resultant of lateral loads acting above the kth storey, $(x^*)_k$, $k = 1, 2, \ldots, N_{\mathrm{b}} + N_{\mathrm{t}}$, is determined. Similarly, the generalized storey eccentricities, $(e_s)_k = (x^*)_k - (x_s)_k, k = 1, 2, \ldots, N_{\mathrm{b}} + N_{\mathrm{t}}$, are obtained by following the steps described in Reference 21.

Step 6: A horizontal section is made at each of the storey levels. The storey shear forces, V_k, $k = 1, 2, \ldots, N_{\mathrm{b}} + N_{\mathrm{t}}$, are calculated by summing the lateral loads acting at the floors above the storey under consideration.

Step 7: For the tower, the effect of torsional motion input due to torsional response of the base may be quantified by a design eccentricity, e_{D}, measured from the storey shear centre and expressed as a fraction of the dimension of the tower perpendicular to the direction of the ground motion:

$$e_D = \pm(\gamma R_a b + e_a) \tag{10}$$

where e_a is the accidental eccentricity, as previously defined. The value of the coefficient γ depends on the configuration of the setback building. Systematic research work is indeed to obtain an appropriate expression of this coefficient as a function of the configuration of setback frame buildings. For simplicity, this paper recommends that the value of γ be taken as 0·1.

Step 8: For the base, the design eccentricity is calculated according to one of the following two expressions (see the discussion after Step 9, below):

$$|(e_{D1})_k| = [2{\cdot}6 - 3{\cdot}6|(e_s)_k|/b]|(e_s)_k| + e_a$$
$$\geqslant 1{\cdot}4|(e_s)_k| + e_a \tag{11}$$

$$|(e_{D2})_k| = |0.5|(e_s)_k| - e_a| \tag{12}$$

In equations (11) and (12), e_{D1} always has the same sign as that of $(e_s)_k$, whilst e_{D2} has the same sign as that of $(e_s)_k$ if $0.5|(e_s)_k| - e_a \geqslant 0$ and has the opposite sign to $(e_s)_k$ if $0.5|(e_s)_k| - e_a \leqslant 0$.

Step 9: The storey torques, $T_k = V_k(e_D)_k$, $k = 1, 2, \ldots, N_b + N_t$, acting about the storey shear centres are calculated for each storey. The design lateral loads for the resisting frame elements are calculated at each storey under the action of the storey shear force V_k and storey torque T_k. It should be noted that to be consistent with existing design codes, the design eccentricity expression which induces the more severe design loading for the resisting element being considered should be employed in determining the storey torques.

The design eccentricity expressions (11) and (12), have been proposed by the authors for regularly stiffness eccentric frame buildings[22] and have recently been adopted by the new Australian earthquake resistant design code.[28] Since the base may be considered as a 'mass' eccentric system, the above design eccentricity expressions do not necessarily lead to satisfactory inelastic performance of the base in irregular setback buildings. Employing a single storey mass eccentric system, Gomez *et al.*[26] have concluded that the storey centre of strength (defined as the point through which the resultant of all element shear forces in a storey acts when all elements in that storey are loaded to their yielding strengths) should be close enough to the storey shear centre, in order to achieve satisfactory inelastic performance of the perimeter element located at the same side as the storey shear centre (measured from the centre of mass). A study by Rutenberg and Eisenberger[29] and a recent study by De Stefano *et al.*[30] have concluded that the optimum location of the centre of strength which minimises the maximum inelastic response in plan-asymmetric systems is close to the midpoint between the storey shear centre and the centre of mass. Therefore, the strength distribution in the base determined in Step 9 above should be adjusted, if necessary, to satisfy the above guidelines in order to achieve satisfactory inelastic performance of the base. These adjustments are outlined in Step 10.

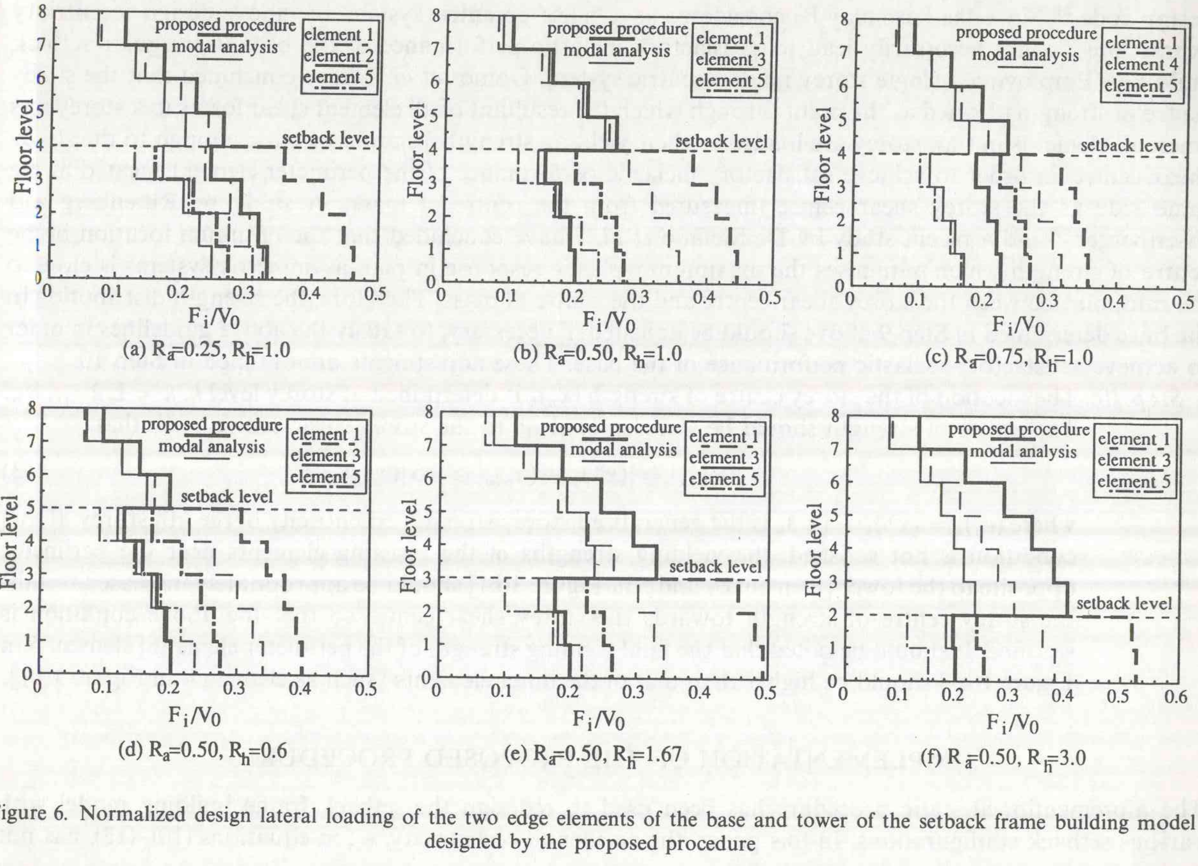
Step 10: The location of the storey centre of strength $(x_p)_k$ is determined at storey level k, $k = 1, 2, \ldots, N_b$. The centre of strength should be sufficiently close to the storey shear centre, such that:

$$|(e_p)_k| = |(x^*)_k - (x_p)_k| \geqslant 0{\cdot}5\,|(e_s)_k| \tag{13}$$

where $(e_p)_k = (x^*)_k - (x_p)_k$ is the generalized storey strength eccentricity of the kth storey. If this condition is not satisfied, the yielding strengths of the resisting elements near the perimeter opposite to the tower [elements 4 and 5 in Figure 1(b)] should be appropriately increased to shift the storey centre of strength towards the storey shear centre so that the above condition is satisfied. It should be noted that the final yielding strength of the perimeter element [element 5 in Figure 1(b)] should be higher than that of the inner elements [such as element 4 in Figure 1(b)].

IMPLEMENTATION OF THE PROPOSED PROCEDURE

The aforementioned static procedure has been used to redesign the setback frame building model with various setback configurations. In this paper, the accidental eccentricity, e_a, in equations (10)–(12), has not been included in determining the design lateral loading of resisting elements for the reasons given earlier. Th overstrength factor associated with the proposed static procedure is shown in Figure 4, in comparison wit the overstrength factor associated with the modal analysis method. Also shown in Figure 4 (dotted lines) ar the ratios of the former to the latter. Figure 4 suggests that the proposed static procedure always leads t a larger overstrength factor (with values up to 1·55) and an additional 10–30 per cent increase in the tota strength of the first storey, compared with the lateral strength resulting from the modal analysis procedure. I view of the fact that the strengths of elements 4 and 5 need to be increased in order to achieve satisfactor inelastic response of these elements and at the same time, the strengths of elements 1 and 2 should not b reduced in order to achieve satisfactory elastic response of these elements, the additional strength-relate costs resulting from the proposed procedure are moderate and justifiable. The design lateral loading F_i normalized to the inelastic static base shear force V_0, of the two edge elements of the base (elements 1 and 5 and that of the two edge elements of the tower (namely elements 1 and 2 if $R_a = 0{\cdot}25$, elements 1 and 3 i $R_a = 0{\cdot}5$, and elements 1 and 4 if $R_a = 0{\cdot}75$) determined by both the modal analysis and the proposed stati procedures, are shown in Figure 6 for purposes of comparison. It is evident from Figure 6 that the propose procedure has resulted in a re-distribution of element strengths, compared with results obtained using the modal analysis procedure. In the tower, the proposed procedure always leads to a larger design strength fo the two edge elements. It is noted that these elements have identical strengths when implementing the proposed procedure [Figure 6, above setback level, shows results only for the tower edge element closest to the notch, with strength determined according to equation (10)]. This increase in strength is particularly important for columns near the notch. In the base, the strength of the element at the centre, element 3, is little changed. However, the strengths of the two edge elements (1 and 5) have been significantly increased by the proposed static procedure (compared with modal analysis), except when $R_a = 0{\cdot}25$ [Figure 6(a)]. This



(a) R_a=0.25, R_h=1.0 (b) R_a=0.50, R_h=1.0 (c) R_a=0.75, R_h=1.0

(d) R_a=0.50, R_h=0.6 (e) R_a=0.50, R_h=1.67 (f) R_a=0.50, R_h=3.0

Figure 6. Normalized design lateral loading of the two edge elements of the base and tower of the setback frame building model designed by the proposed procedure

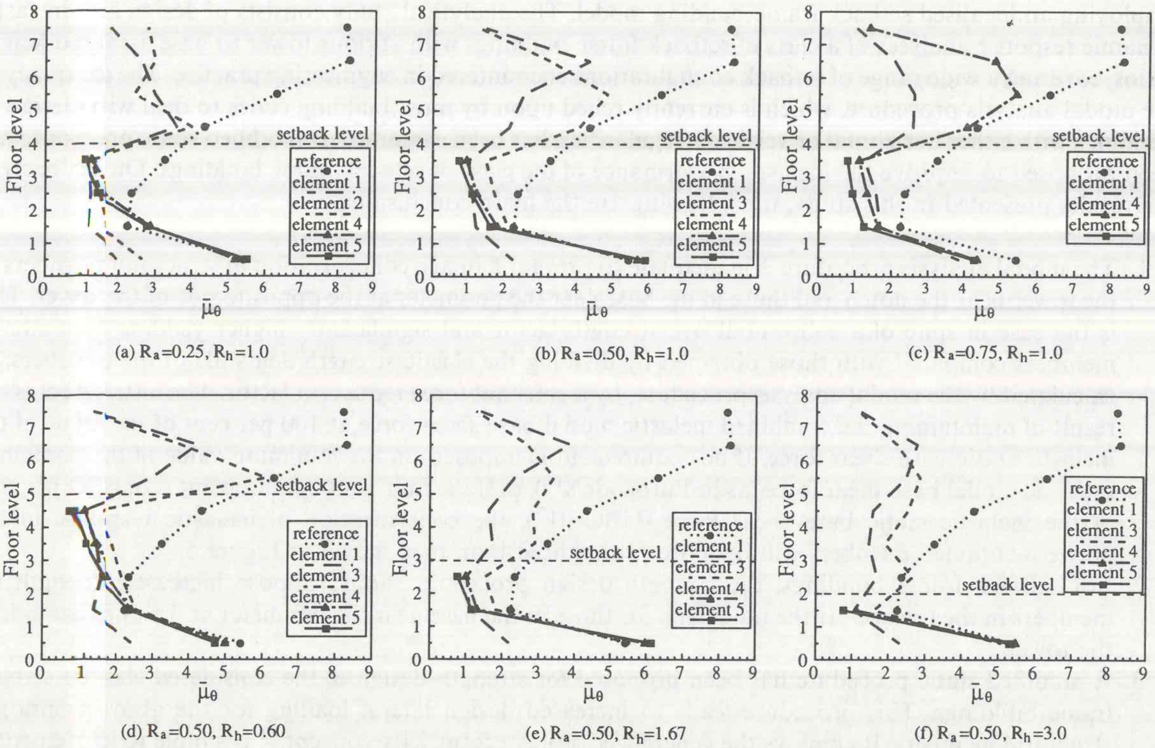


Figure 7. Average ductility demands of selected resisting elements of the setback frame building model designed by the proposed procedure

increase is necessary for element 5. However, in general, element 1 is overdesigned to some extent. This is the result of employing a single simplified design eccentricity expression for the strength design of element 1. Further study is needed, though beyond the scope of this paper, to improve the proposed design procedure in this respect.

A series of inelastic dynamic response analyses of the redesigned setback frame building model have been carried out. The ground motions are identical to those used in the previous section dealing with the modal analysis procedure. The average ductility demands have been computed as described previously, and are shown in Figure 7. The standard deviations of computed ductility demands corresponding to the six records range from 5 to 50 per cent of the mean values, being consistent with the scatter of the spectral accelerations at one second period. Results in Figure 7 should be compared with the corresponding responses shown in Figure 5 (modal analysis procedure). Figure 7 indicates that the proposed procedure leads to a significant reduction in the ductility demands compared with those associated with the modal analysis procedure. In the base, the ductility demands of the perimeter elements at the opposite side of the tower (elements 4 and 5) are lower than, or around that of the reference system for all the setback configurations considered. In the tower, the degree of inelastic action (and hence damage concentration) at columns near the notch has been greatly reduced. The ductility demands of columns in the two edge elements are lower or around those of the reference system. Hence, the proposed procedure can be regarded as successful in reducing damage concentration in critical regions of the class of setback frame buildings considered herein.

CONCLUSIONS

As a first step towards understanding the inelastic earthquake torsional response of setback frame buildings in which both horizontal and vertical irregularities arise, this paper has carried out an analytical study employing an idealised setback frame building model. The analytical study consists of design and inelast dynamic response analyses of a class of setback frame buildings with various tower to base area and heigh ratios, covering a wide range of setback configurations encountered in engineering practice. The adequacy c the modal analysis procedure, which is currently relied upon by most building codes to deal with design c buildings with either horizontal or vertical irregularities, has been examined. A modified static procedure ha been proposed to improve the inelastic performance of the class of setback frame buildings. On the basis c the results presented in this study, the following are the main conclusions:

1. The modal analysis procedure is inadequate to prevent damage concentration in structural members i the tower near the notch and those in the base near the perimeter at the opposite side of the tower. Th is the case in spite of a substantial overstrength factor and significantly higher yielding strengths c members compared with those obtained by dividing the elastic strength demands of the members, a calculated by the modal analysis procedure, by a constant force reduction factor. The latter effect is th result of maintaining the combined inelastic modal base shear force at 100 per cent of the value of th inelastic static base shear force. If no requirement is imposed on the minimum value of the combine inelastic modal base shear force (as in Eurocode 8[10]), or if the minimum requirement is only 90 per cer of the inelastic static base shear force (UBC 91[7]), the concentration of inelastic response in th above-mentioned members will be more pronounced than that shown in Figure 5.
2. For setback frame buildings, the strength design procedure should impose increased strength fc members in the tower near the notch and for those in the base near the perimeter at the opposite side c the tower.
3. A modified static procedure has been proposed for strength design of the considered class of setbac frame buildings. This procedure leads to increased design lateral loading for the above-mentione structural members. It employs the generalized storey eccentricity concept,[21] is simple to implement i design practice and has been shown to result in improved and satisfactory inelastic performance c setback frame buildings having a wide range of setback configurations. Compared with the moda analysis procedure, the proposed procedure always leads to more conservative design lateral loads fc the two edge elements in the tower and those in the base near the perimeter at the same side as th tower. Since these elements are the ones affected most unfavourably by torsion when setback building respond elastically, the proposed procedure is also expected to result in satisfactory elastic performanc of the class of setback frame buildings considered in this study.

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