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Inelastic response of multistorey buildings under earthquake excitation

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Abstract. It is well recognized that structures designed to resist strong ground motions should be able to withstand substantial inelastic deformations. A simple procedure has been developed in this paper to monitor the dynamic earthquake response (time-history analysis) of both steel and concrete multistorey buildings in the inelastic range. The building is treated as a shear beam model with three degrees of freedom per floor. The entire analysis has been programmed to run on a microcomputer and can output time histories of displacements, velocities, accelerations and member internal forces at any desired location. A record of plastic hinge formation and restoration to elastic state is also provided. Such information can be used in aseismic analysis and design of multistorey buildings so as to control the damage and optimise their performance.

Key words: asymmetry; multi-storey building; hysteresis model; inelastic response; aseismic design.

1. Introduction

In the aseismic design of a building, it is important to know the behaviour of the building during a serious earthquake, when it could be subjected to forces and deflections whose magnitudes are several times of those predicted and provided for in the design. This introduces the need for inelastic (or nonlinear) dynamic analysis of the building. Cyclic motions of the building under earthquakes demand ductility characteristics which enable the building to absorb energy without collapse, even when some selective damage is allowed. At some locations of the building, occurrence of hinges must be predicted and/or controlled, and for this purpose inelastic time history analysis is very important. When this is possible the structural engineer can design the building to a certain minimum strength as well as consider its performance at greater overloads and large deformations. This results in optimum performance and considerable savings in comparison to designing for such loading on an elastic basis.

In earthquake resistant design of buildings dual design criteria are adopted. These are:
(a) the building must withstand a (likely) moderate earthquake without structural damage.
(b) the building must withstand the most severe earthquake likely to occur in the region with damage permitted in certain specified locations (i.e. selective damage) but without collapse or failure that might cause major property damage or loss of life.

Most multistorey buildings lack symmetry in either plan and/or evaluation, or in the distribution of mass and/or stiffness and are called asymmetric. A symmetric building is one in which the centres of mass and stiffness coincide on every floor and these lie on a common vertical axis. In practice, this requirement is impossible and most buildings are asymmetric to some extent. In an asymmetric building there is coupling between the lateral and torsional components of the response and this complicates the analysis. Asymmetry has been a major cause of the collapse or poor performance of buildings during earthquakes (Chandler 1991). Therefore it is important to account for the effects of asymmetry in the earthquake analysis of multistorey buildings, especially in the inelastic range, when they are subjected to large deformations. Analysis for criterion (a) is usually in the elastic range where the force–deflection relations are assumed to be linearly elastic. Elastic analysis of symmetric and asymmetric buildings has received extensive coverage in the literature; for example see Thambiratnam and Thevendran 1992, Maheri et al. 1991, Stafford Smith and Cruvellier 1990, Hejal and Chopra 1989, Thambiratnam and Irvine 1989, Cheung and Tso 1987, Chandler and Hutchinson 1987 & 1986, Tsicnias and Hutchinson 1981, Tso and Dempsey 1980 and Kan and Chopra 1977. Analysis pertaining to criterion (b) takes the structure into the inelastic range. In the last decade, most of the models proposed for inelastic analysis of asymmetric buildings were too simple and suitable only for single storey buildings. Models treated in various studies were different and therefore the few techniques available for inelastic earthquake analysis, unfortunately give conflicting results (Chandler and Duan 1990).

During the past several years the ability to analyse buildings for the effects of earthquake ground motion has increased considerably. In the preliminary design of a building structure, the primary concern had been the maximum response, as precise details of the response cost time and money. The concept of inelastic analysis of a structure using response spectra was first introduced by Newmark (1970). Later on other researchers used the response spectra for both elastic analysis (Tsicnias and Hutchinson 1981, Tso and Dempsey 1980 and Kan and Chopra 1977) and inelastic analysis (Shri Pal et al. 1987 and Balendra and Koh 1991), together with different hysteresis models and ground motions. These studies are helpful in developing design guidelines which indicate how peak response parameters vary with the dynamic, mechanical and damping characteristics. But designers also need to know the complete behaviour of a structure during an expected ground motion and therefore in an inelastic analysis, the focus of attention should not be only on the peak response. Realizing this, real time history analyses of asymmetric buildings subjected to earthquake ground motions have been carried out in the elastic range by Chandler and Hutchinson 1987 & 1986, and in the inelastic range by Kan and Chopra 1981 and by Corderoy and Thambiratnam 1993.

Studies on inelastic earthquake response of asymmetric building structures began at the commencement of the last decade, and most of them are based on single storey models. Kan and Chopra (1981) carried out parametric studies on the inelastic response of single–storey one way eccentric structural models subjected to the 1940 El Centro earthquake record, and used yield criteria on shear forces in two perpendicular directions as well as on torsion and shear force. Shri Pal et al. (1987) using different hysteresis models, analysed a viscously damped single degree of freedom system under five different ground motions. Balendra and Koh (1991) investigated the inelastic dynamic response of an asymmetric single–storey shear building supported on a linear elastic half–space for sinusoidal excitation. Unlike elastic response, the inelastic response is model dependent, and as mentioned earlier the conclusions drawn from existing inelastic studies are inconclusive and there are no general trends in the results (Chandler and Duan 1990).

This paper is one in a series which aim to present an acceptable simple method for inelastic dynamic analysis of multi–degree of freedom building systems. The procedure can monitor the inelastic time history responses to predict the location and extent of plasticity in a multi–

storey structure. In an earlier study, the plastic dynamic analysis of steel buildings was treated by Corderoy and Thambiratnam (1993) using an elastoplastic hysteresis model. In this paper, two hysteresis models: (1) elasto–plastic model and (ii) stiffness degrading model are used to monitor the inelastic behaviour of multistorey steel and concrete buildings respectively, under earthquake excitation. It is assumed that the elements of the structure remain uncracked. At this stage, the yield criterion pertains only to the lateral shear force in columns. This is acceptable in shear buildings where torsional coupling effects are not very significant. Later on, it is intended to include the torsional shear stress effects on the yield criterion.

In section two, the model used in the analysis together with the theory used are treated. Hysteresis models and relevant procedures used in this paper are described in the next section. Numerical examples are presented in section 4 to illustrate the procedure and the results discussed. The final section concludes the paper with relevant comments.

2. Structural model

The shear beam model described in Corderoy and Thambiratnam (1993) is used in the analysis of the multistorey buildings, which may be asymmetric and therefore subjected to the effects of torsional coupling. A reference column $R_0 \rightarrow R_n$ is located at the origin of the coordinate axis and intersects the floor levels at R_i $(i=1 \rightarrow n)$ (Figl. 1a). The important assumptions made in the model are: (1) floors are treated as rigid diaphragms each of which possess two horizontal translational and one rotational (about vertical axis) degrees of freedom (Fig. 1b); (2) kinetic energies of vertical members are neglected but these can be included by lumping column masses to the floors at their two ends and (3) ends of all vertical members are rigidly connected with the floors. Based on these assumptions, the transformation relation between the end displacements of column r at level i and those at a reference column R_i is given by (See Fig. 1a):

$$w_{ri} = D_r \, w_{Ri} \,, \qquad (i=1,2,3,\cdots,n) \tag{1}$$

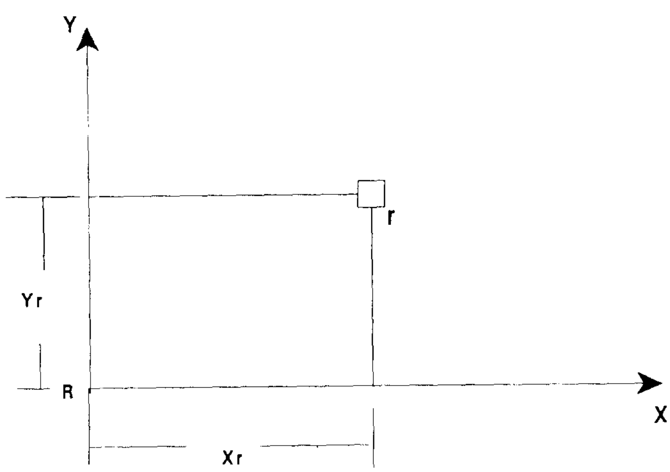


Fig. 1a Reference column and axes

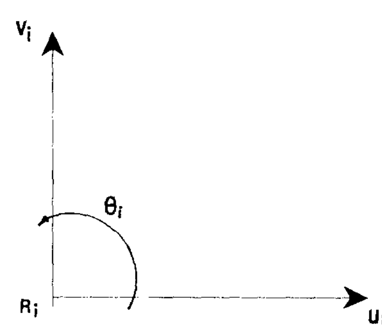


Fig. 1b Degrees of freedom at level i

Where $(w_{ri})^T = (u_{ri}, v_{ri}, \theta_{ri})$ are the displacements of column r at level i, and $(w_{Ri})^T = (u_i, v_i, \theta_i)$ are those at the reference column. The transformation matrix D_r is given by:

$$D_r = \begin{bmatrix} 1 & 0 & -y_r \\ 0 & 1 & x_r \\ 0 & 0 & 1 \end{bmatrix} \tag{2}$$

2.1. Potential Energy

For column r at level i, the potential energy (of this vertical member) is given by:

$$V_{ri} = \frac{1}{2} \Delta_{ri}^T K_{ri} \Delta_{ri} \tag{3}$$

Where $\Delta_{ri} = (w_{ri-1}, w_{ri})$ is the displacement vector of column r. If $\Delta_{Ri} = (w_{Ri-1}, w_{Ri})$ are the displacements of reference column, then Δ_{ri} and Δ_{Ri} are related according to

$$\Delta_{ri} = C_r \Delta_{Ri}; \quad C_r = \begin{bmatrix} D_r & 0 \\ 0 & D_r \end{bmatrix} \tag{4}$$

The potential energy of all of columns in level i is given:

$$V_i = \sum_{r=1} \frac{1}{2} \Delta_{Ri}^T C_r^T K_{ri} C_r \Delta_{Ri} = \frac{1}{2} \Delta_{Ri}^T K_{Ri} \Delta_{Ri} \tag{5}$$

The total potential energy of the building is obtained by summing up the contributions of V_i from level 1 to level n:

$$V = \sum_{i=1}^{n} V_i = \frac{1}{2} \sum_{i=1}^{n} \Delta_{Ri}^T K_{Ri} \Delta_{Ri} \tag{6}$$

2.2. Kinetic energy

Since the kinetic energies of vertical members are neglected, the kinetic energy at level i is solely contributed by the floors and is given by:

$$T_i = \frac{1}{2} (\dot{u}_{ci} \quad \dot{v}_{ci} \quad \dot{\theta}_i) \begin{bmatrix} m_i & 0 & 0 \\ 0 & m_i & 0 \\ 0 & 0 & J_{ci} \end{bmatrix} \begin{Bmatrix} \dot{u}_{ci} \\ \dot{v}_{ci} \\ \dot{\theta}_{ci} \end{Bmatrix} = \frac{1}{2} \dot{w}_{ci}^T M_{ci} \dot{w}_{ci} \tag{7}$$

Where $\dot{u}_{ci}$, $\dot{v}_{ci}$ are the velocity components of the centroid C_i in the x and y directions respectively. $\dot{\theta}_{ci}$ is the angular velocity of the floor slab about the vertical axis, while m_i and J_{ci} denote the slab's mass and polar moment of inertia about the centroid respectively. To unify the variables in the floor, $\dot{w}_{ci}$ is transformed to $\dot{w}_{Ri}$ which is the velocity vector of the reference column:

$$\dot{w}_{ci} = D_{ci}\,\dot{w}_{Ri}\,; \qquad D_{ci} = \begin{bmatrix} 1 & 0 & -y_{ci} \\ 0 & 1 & x_{ci} \\ 0 & 0 & 1 \end{bmatrix} \tag{8}$$

The kinetic energy is obtained as:

$$T_i = \frac{1}{2}\,\dot{w}^T_{Ri} D^T_{ci} M_c D_{ci}\,\dot{w}_{Ri} = \frac{1}{2}\,\dot{w}^T_{Ri} M_{Ri}\,\dot{w}_{Ri} \tag{9}$$

Where M_{Ri} is the mass matrix of the floor about reference point R_i. Kinetic energy for the whole building T is obtained by summing up T_i to give:

$$T = \sum_{i=1}^{n} T_i = \sum_{i=1}^{n} \dot{w}^T_{Ri} M_{Ri}\,\dot{w}_{Ri} \tag{10}$$

2.3. Equation of motion

In the absence of damping, Lagrange's equation are:

$$\frac{d}{dt}\left(\frac{\partial T}{\partial \dot{w}_{Ri}}\right) - \frac{\partial T}{\partial w_{Ri}} + \frac{\partial V}{\partial w_{Ri}} = P_{Ri}\,; \quad (i=1,2,3,\cdots,n) \tag{11}$$

Where $P_{Ri} = (F_{xi}, F_{yi}, F_{Ri})^T$ is the load vector at node i on the reference column. Using equation (11) together with equations (6) and (10), the equation of motion is obtained as:

$$M_R \ddot{W}_R + K_R W_R = P_R \tag{12}$$

In the above equation, M_R and K_R are the mass and stiffness matrices, $W = (w_{R1}, w_{R2}, w_{R3}, \cdots, w_{Rn})$ is the displacement vector with $3n$ elements and n is the number of storeys. The term on the right hand side is the inertia force vector due to ground motion.

In this study ground motion is simplified to a ground acceleration depicted by a sine function, which is given by:

$$\ddot{v}_g = A \cdot \sin\frac{2\pi t}{T} \tag{13}$$

where A is its amplitude taken as a fraction of the acceleration due to gravity g and T is its period.

3. Hysteresis Models and Procedure

The hysteresis models used in this study are: (1) the elasto–plastic model for steel buildings and (2) the stiffness degrading model for reinforced concrete buildings (Shri Pal et al. 1987). These two hysteresis models are best suited for their corresponding structures, and their loading and unloading rules are shown in Fig. 2a and Fig. 2b, respectively. For the elasto–plastic model, the times of yield and restoration of each column in every floor are recorded. For the stiffness degrading model, the coordinates of control points (C1–C10) are monitored and slopes of loading and unloading segments are determined. The yield codes (0–8) are used to depict the different states of loading and unloading. The loading/unloading

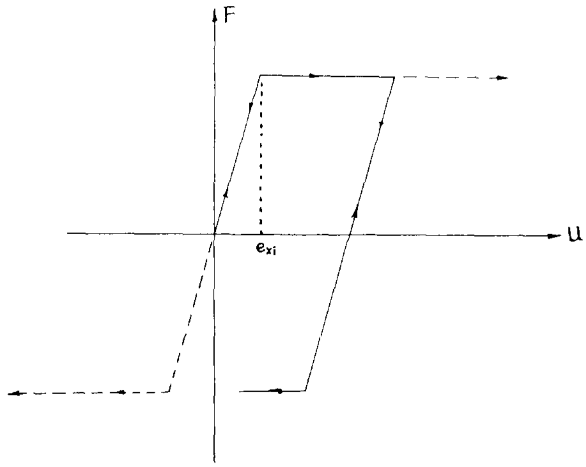
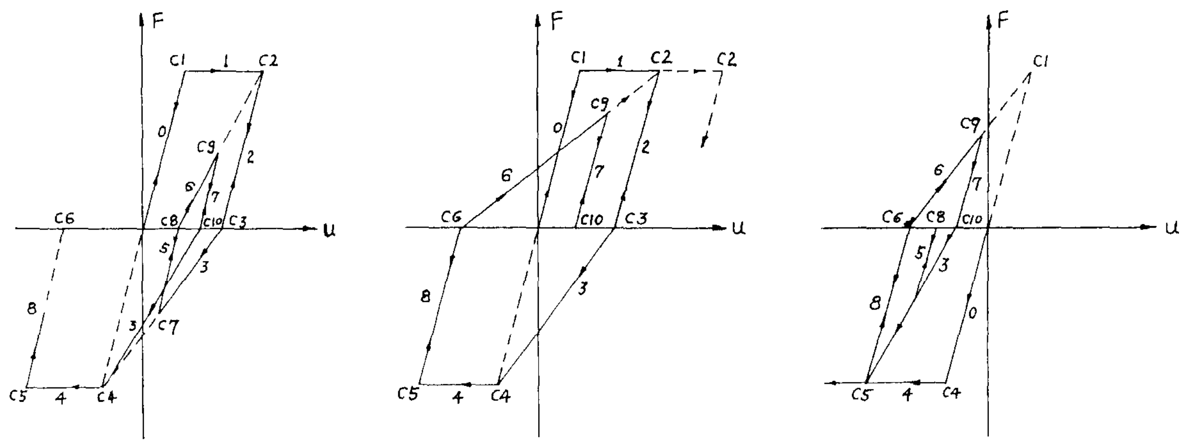


Fig. 2a Elasto-plastic hysteresis model



(a) (b) (c)

Fig. 2b Stiffness degrading hysteresis model

slopes of segments 0, 2, 5, 7, 8 remain equal to the initial stiffness, while the stiffness at segments 1 and 4 remain zero. The slopes of segments 3 and 6 which depend on the coordinates of control point of last zero crossing and last unloading point are computed within the program. In this study it is assumed that the members in the building possess sufficient ductility so that they could be restored to the elastic state from any location on the lines of zero slopes of the hysteresis models.

The entire analysis has been programmed to run on a microcomputer and is capable of carrying out quasi static or time history analysis of multistorey buildings subjected to earthquakes. The present microcomputer program uses direct integration method on equation (12) and gives the displacements u_i, v_i, θ_i of every floor (with respect to the reference column), and the lateral forces at the ends of all vertical members. Numerical integration in carried out using Newmark's Method with $\alpha = 1/6$, $\beta = 0.5$. For monitoring inelastic behaviour, the program checks the columns one by one in a storey and storey by storey, for formation of plastic hinges and/or restriction.

Considering the behaviour of the rth column at level i in x and/or y direction, the maximum shearing force in this column will be:

in x direction

$$FM_{xri} = \frac{12EI_{yr}}{L_i^3} e_{xi} \tag{14a}$$

in y direction

$$FM_{yri} = \frac{12EI_{xr}}{L_i^3} e_{yi} \tag{14b}$$

where E is Young's modulus, I_{xr} and I_{yr} are the appropriate second moments of area about x axis and y axis respectively, L_i is the height of the columns in level i, e_{xi} and e_{yi} are the elastic limits at level i in x direction and y directions respectively. This yield criterion will be approximate for members in a building subjected to significant torsional motions. In such cases torsional shear stresses and the interaction between the shear forces in the x and y directions must be considered in predicting the yield. This could be done in the present procedure and work is presently underway towards this. These elastic limits can be controlled by the designer depending on the design requirements. This is a useful feature in the approach presented herein and enables a certain degree of control on the number and order of plastic hinges to be formed during a severe ground motion so as to give the designer a feasible way to minimise the earthquake damage to a building.

4. Examples and discussion

Two multistorey buildings are treated in this study. Both structures are subjected to ground acceleration given by

$$\ddot{v}_g = \begin{cases} 0.3g \sin \pi t \cdots\cdots t \leq 2 \\ 0.15g \sin \frac{\pi t}{2} \cdots\cdots t > 2 \end{cases} \tag{15}$$

In all the buildings, weight densities of the floor slabs are taken as 5 *kPa*, while those of the roof slabs are taken as 2.5 *kPa*.

4.1. Example 1–3 storey building with setbacks

In the first example a three storey building having uniform and symmetrical columns and with two set backs as shown in Fig. 3 is treated to illustrate the procedure. Here, two structures, one of steel and the other of reinforced concrete are considered.

Case 1. Steel building

In the steel building, Young's modulus and Poisson's ratio are taken to be $20 \times 10^7 kN/m^2$ and 0.3 respectively, and all other columns have second moments of area : $I_x = I_y = 42 \times 10^6$

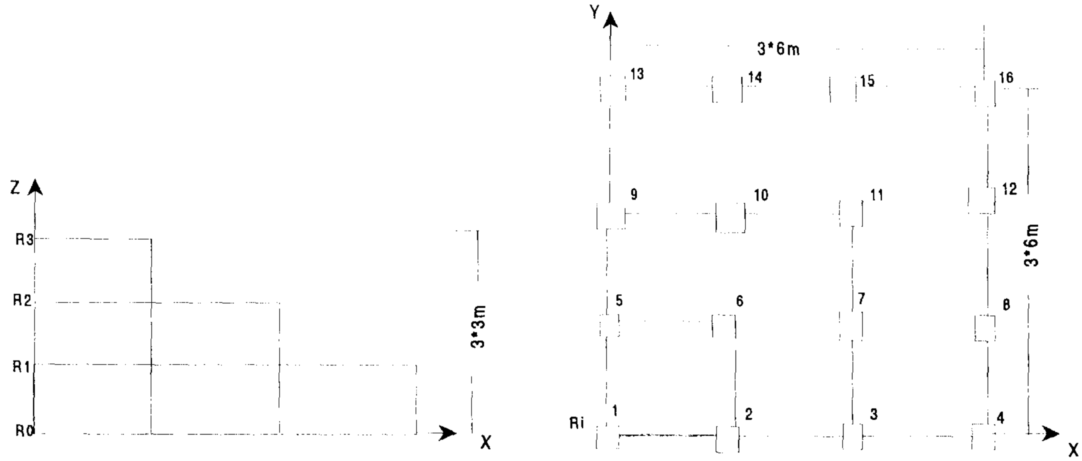


Fig. 3 3 storey building

mm^4. The elastic limits are set to $15mm$, and the elasto–plastic hysteresis model is adopted. The horizontal deflections in the y direction at first storey level on the reference axis are shown in Fig. 4a, and the shear force just below the first floor in the y direction is shown in Fig. 4b. In these figures, ep and e pertain to the elasto–plastic and elastic responses respectively. It can be seen that there are no significant differences between the elasto–plastic and elastic responses. The elastic response is obtained when no elastic limits are set on e_{xi} or e_{yi} in equation (14).

This is because when the elastic limits is set to $15mm$, only a few of the columns at the first storey level yield while most of the other columns at this level remain elastic. There is no occurrence of a complete yield or collapse situation. It can be seen that for this case the overall displacement ductility factor is very small at approximately 1.17. Table 1 records the information on the yielding and restoration of first storey columns. From this table it is evident that the columns which yield are quickly restored to their elastic state.

Case 2. Reinforced concrete building

In the reinforced concrete building, Young's modulus and Poisson's ratio are assumed to be $30\text{x}10^6 kN/mm^2$ and 0.2 respectively and all the columns have second moments of area: $I_x = I_y = 325.5\text{x}10^6 mm^4$ with the elastic limits set to $10mm$. When the elastic limits was set to $14.1mm$ or larger, there was no occurrence of yielding. Fig. 5a and Fig. 5b give the horizontal deflections in the y direction at the first storey level on the reference axis and the shear forces in the y direction just below the first floor. The hysteresis model used in this analysis is the stiffness degrading model. In these figures e and sd pertain to the elastic and stiffness degrading responses respectively. The figures show that the emerging of hinges in the columns brings larger deflections of the floors. The maximum deflection of elastic line is $14.1mm$ (obtained when no elastic limit was set) while the maximum deflection of the stiffness degrading line is $32mm$ and this deflection occurs at 1.79 seconds after the ground motion commenced. At this instant, all the sixteen columns at first storey have yielded for more than 0.44 seconds. How-

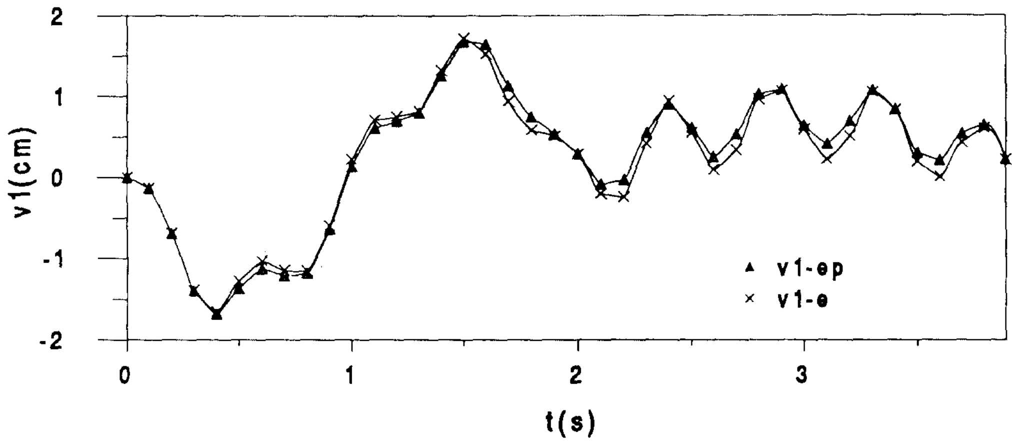


Fig. 4a Horizontal deflections of level 1 in 3 storey building(steel)

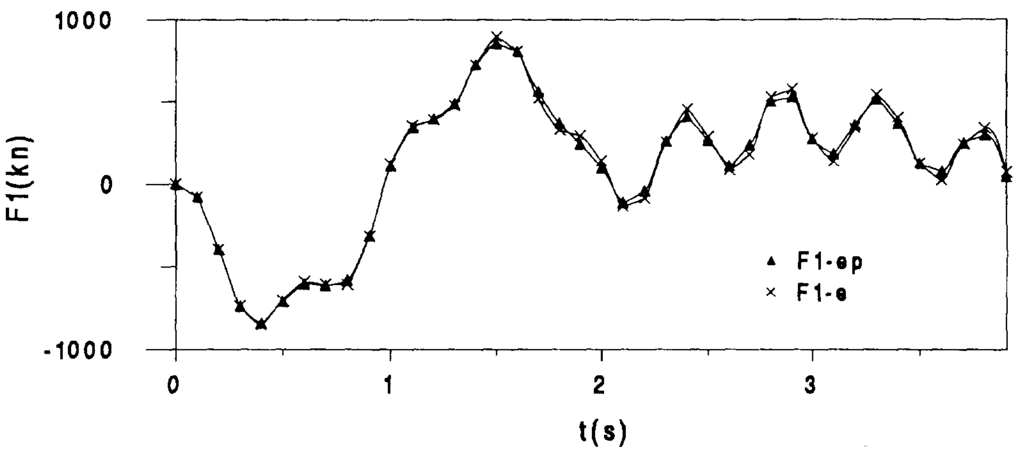


Fig. 4b Y direction shear forces below level 1 in 3 storey building(steel)

Table 1 Yield and restore times of first storey columns in 3 storey steel building

Time(s)	Cols. Yielding	Cols. Restored
0.33	1,5,9,13	
0.38	2,6,10,14	
0.40		1,2,5,6,9,10,13,14
1.43	1,5,9,13	
1.49	2,6,10,14	
1.55		2,6,10,14
1.56		1,5,9,13

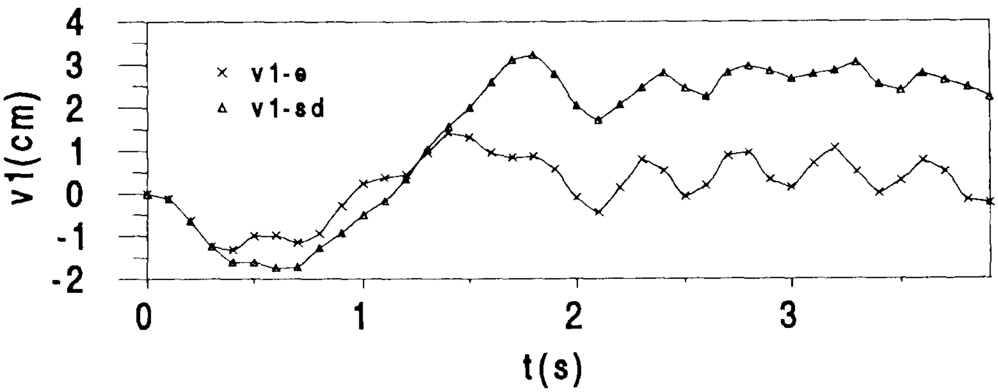


Fig. 5a Horizontal deflections of level 1 in 3 storey building (concrete)

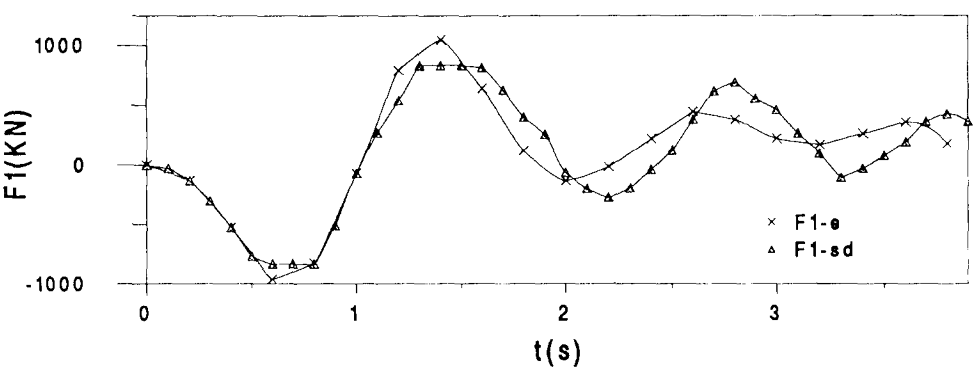


Fig. 5b Y direction shear forces below level 1 in 3 storey building (concrete)

ever the building did not collapse because the ground motion and the upper storeys exert some restoring forces on the first floor to bring it back. This response gives an overall displacement ductility factor of approximately 3.2. Table 2 gives detail information on hinge formation.

Both Table 1 and Table 2 show that columns 1, 5, 9, 13 yield first. These are on a line parallel to the y axis (along which the ground motion acts) and most affected by the initial anti-clockwise twisting of this asymmetric building, which enhances their lateral displacements. Columns 4, 8, 12, and 16 yield last as the effects of the twisting reduce their lateral displacements. In this three storey setback building, both the centres of mass and stiffness lie on the axis of symmetry at 45° to the x axis. However the centre of mass is closer to the y axis causing initial anti-clockwise twisting when the ground motion is in the positive y direction.

4.2. Example 2-10 storey reinforced concrete building

In the second example a 10 storey reinforced concrete building haveing a prismatic cross-section with uniform and symmetrical columns as shown in Fig. 6 is considered. Young's

Table 2 Hinge formation in first storey level of 3 storey concrete building

Time(s)	Hinge No.	Cols.No.	Time(s)	Hinge No.	Cols. No.
0.27	C4	1, 5, 9, 13	2.12	C7	1, 5, 9, 13
0.28	C4	2, 6, 10, 14	2.13	C7	2, 6, 10, 14
0.31	C4	3, 7, 11, 15	2.14	C7	3, 7, 11, 15
0.37	C4	4, 8, 12, 16	2.15	C7	4, 8, 12, 16
0.5	C5	4, 8, 12, 16	2.18	C8	1, 5, 9, 13
0.51	C5	3, 7, 11, 15	2.19	C8	2, 6, 10, 14
0.67	C5	2, 6, 10, 14	2.20	C8	3, 7, 11, 15
0.68	C5	1, 5, 9, 13	2.21	C8	4, 8, 12, 16
0.94	C6	1, 5, 9, 13	2.80	C9	2, 6, 10, 14
0.96	C6	2, 6, 10, 14	3.18	C9	3, 4, 7, 8, 11, 12, 15, 16
0.99	C6	3, 7, 11, 15	3.29	C9	1, 5, 9, 13
1.00	C6	4, 8, 12, 16	3.59	C10	4, 8, 12, 16
1.31	C1	1, 2, 5, 6, 9, 10, 13, 14	3.61	C7	4, 8, 12, 16
1.32	C1	3, 7, 11, 15	3.63	C8	4, 8, 12, 16
1.34	C1	4, 8, 12, 16	3.73	C9	4, 8, 12, 16
1.78	C2	4, 8, 12, 16	3.81	C10	4, 8, 12, 16
1.79	C2	1, 2, 3, 5, 6, 7, 9, 10	3.84	C10	3, 7, 11, 15
1.79	C2	11, 13, 14, 15	3.91	C7	3, 4, 7, 8, 11, 12, 15, 16
1.94	C3	4, 8, 12, 16	3.93	C7	2, 6, 10, 14
1.97	C3	3, 7, 11, 15	3.95	C10	1, 5, 9, 13
1.98	C3	2, 6, 12, 14	3.97	C8	4, 8, 12, 16
1.99	C3	1, 5, 9, 13	3.98	C8	3, 7, 11, 15

modulus and Poisson's ratio are taken as $30\times 10^6 kN/m^2$ and 0.2 respectively. All the columns have the same second moment of area: $I_x = I_y = 1250\times 10^6/mm^4$. The elastic limits is set to 25 mm and the stiffness degrading hysteresis model is used. During the first 4 seconds of excitation, the displacements of the first and second storeys exceeded the elastic limit while all the other storeys remained in the elastic state. The horizontal deflections in the y direction at the first floor level on the reference axis and shear forces just beneath the first floor in the y direction are shown in Fig. 7a and Fig. 7b. The overall displacement ductility factor for this case is approximately 2.2. Table 3 gives detailed information of hinge formation at the first storey level. It can be seen that all the columns in this storey have simultaneous hinge formation. This is expected as the building is symmetrical and there is no torsional coupling. At time 0. 56 seconds and time 1.28 seconds, all 20 columns in this level yielded and the stiffness became zero. At time 0.6 seconds and time 1.3 seconds, all 40 columns in the first two levels yielded and the stiffnesses of these columns became zero. But the building did not collapse and all the columns were restored at 0.8 seconds and 1.56 seconds respectively. This is because the ground motion and the upper storeys exert restoring forces on these floors to bring them back. This behaviour is only possible if the building possess sufficient ductility as assumed.

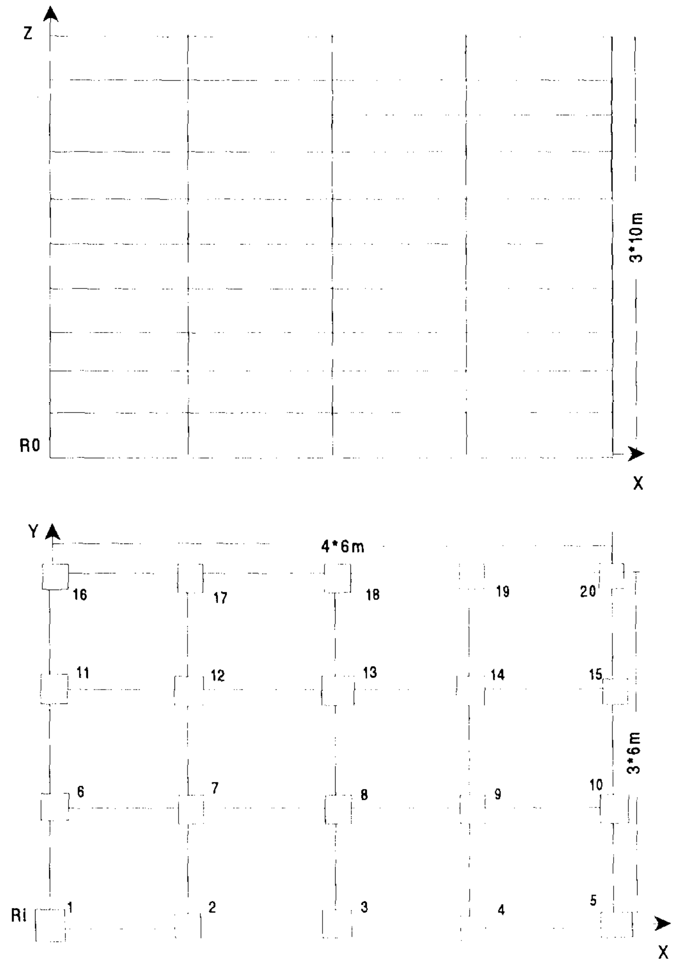


Fig. 6 10 storey building

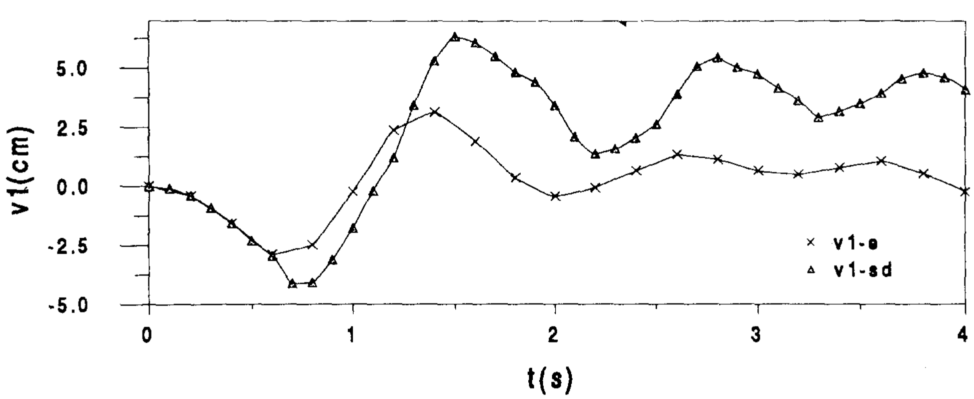


Fig. 7a Horizontal deflections of level 1 in 10 storey building

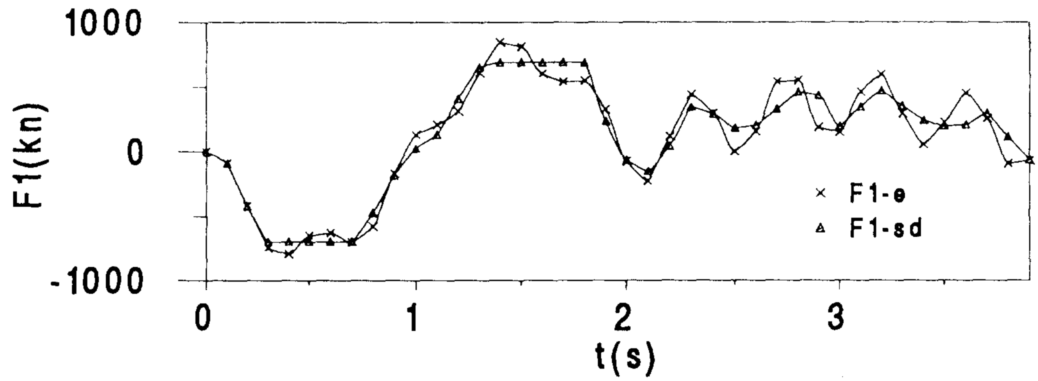


Fig. 7b Y direction shear forces below level 1 in 10 storey building

Table 3 Hinge formation in first and second storey levels of 10 storey building

First Storey			Second Storey		
Time(s)	Hinge No.	Cols. No.	Time(s)	Hinge No.	Cols. No.
0.56	C4	1, 2, ···········, 20	0.60	C4	1, 2, ···········, 20
0.80	C5	1, 2, ···········, 20	0.66	C5	1, 2, ···········, 20
1.02	C6	1, 2, ···········, 20	1.04	C6	1, 2, ···········, 20
1.28	C1	1, 2, ···········, 20	1.30	C1	1, 2, ···········, 20
1.56	C2	1, 2, ···········, 20	1.36	C2	1, 2, ···········, 20
2.00	C3	1, 2, ···········, 20	2.02	C3	1, 2, ···········, 20
2.26	C7	1, 2, ···········, 20	2.22	C7	1, 2, ···········, 20
2.48	C8	1, 2, ···········, 20	2.48	C8	1, 2, ···········, 20
2.84	C9	1, 2, ···········, 20	2.80	C9	1, 2, ···········, 20
3.26	C10	1, 2, ···········, 20	3.24	C10	1, 2, ···········, 20
3.32	C7	1, 2, ···········, 20	3.36	C7	1, 2, ···········, 20
3.46	C8	1, 2, ···········, 20	3.46	C8	1, 2, ···········, 20
3.86	C9	1, 2, ···········, 20	3.82	C9	1, 2, ···········, 20

5. Conclusions

A simple procedure for the inelastic dynamic analysis of multistorey buildings (symmetric or asymmetric), when subjected to earthquake loads, has been presented in this paper. It can also perform elastic analysis if required. When performing inelastic analysis, depending on whether the building is steel or concrete, it is possible to choose the elasto-plastic hysteresis model or the stiffness degrading model respectively. The failure criteria used in both hysteresis models pertain only to lateral shear effects. The procedure gives time histories of displacements and internal forces at all locations together with the information on plastic hinge formation and restoration column by column and storey by storey. Numerical examples of asymmetric and symmetric buildings were treated. The results for the cases considered, indicate that even with some plastic hinges forming at certain locations in the lower storeys, the deflections are not excessive and that the buildings did not collapse. This is partly due to the restoring action of the other storeys and partly due to the reversal of the earthquake forces. The simple method presented in this paper will give the designer a useful tool for the aseismic design of buildings in which it is possible to control the damage (plastic hinge formation). This will enable the buildings to withstand significant inelastic deformation without collapse, when resisting strong ground motion.

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