Bull. Ind. Soc. Earth. Tech., Paper No.365, Vol.34 No.2, June 1997, pp 47-74

A STATE OF ART:

SEISMIC ANALYSIS OF STEPBACK AND SETBACK BUILDINGS

Satish Kumar
Assistant Professor.
Civil Engg. Deptt.
College of Engg. & Tech.
Bathinda - 151001

D.K. Paul
Professor
Earthquake Engg. Deptt.
University of Roorkee
Roorkee - 247667

ABSTRACT

Hilly areas are under going rapid changes due to economic development, which have marked effects on the buildings in terms of style, material and method of construction. Stone, load bearing and wooden building structures are common. Most of the hilly regions of India are highly seismic. Buildings on hill slope differs in a way from other buildings. The various floors of such buildings steps back towards the hill slope and at the same time building may have setbacks also. Due to varied configurations of buildings in hill areas, these buildings become highly irregular and asymmetric. Buildings constructed in hill areas are much more vulnerable to seismic environment. In this paper the state of art on seismic analysis, behaviour and analytical modelling of components of hill buildings has been brought out and the conclusions has been drawn.

KEY WORDS:

Stepback, Setback, Torsional Coupling, Irregular Buildings, Stiffness Degradation, Analytical Models, Inelastic Analysis

INTRODUCTION

Multistoreyed R.C. framed buildings are getting popular in hilly areas and many of them are constructed on hill slopes. Setback multistoreyed buildings are unfrequent over level grounds whereas stepback buildings are quite common on hill slopes. A combination of stepback and setback buildings are also common on hill slopes. Setback and stepback buildings are shown in Fig.1. The buildings may have setbacks in one or both the principal directions located symmetrically or unsymmetrically about the vertical axis. At the location of setback stress concentration is expected when the building is subjected to earthquake excitation. These are generally not symmetrical due to setback and/or stepback and result into severe torsion under an earthquake excitation. Current building codes suggest detailed dynamic analysis for these types of buildings. For symmetrical multistoreyed setback buildings, the building may be decoupled whereas for unsymmetrical buildings, a coupled analysis is required. Buildings in hill areas are irregular and asymmetric and therefore are subjected to severe torsion in addition to lateral forces under the action of earthquakes. Many buildings on hill slopes are supported by columns of different heights. The shorter columns attract more forces as the stiffness of the short columns is more and undergo damage when subjected to earthquakes. Buildings in hill areas are subjected to lateral earth pressure at various levels in addition to other normal loads as specified on buildings on level grounds. Building loads transmitted at the foundation level to a slope create problem of slope instability and may result into total collapse of the building. The soil profile is non uniform on the hill slope and result into different properties of soils at different levels. The bearing capacity, cohesion, angle of internal friction, etc. may be different at different levels. It may result into unequal settlement of foundations and local failure of the slope. Climatic conditions and heavy rains are a big problem for buildings in hill areas requiring special attention for drainage, temperature control and lighting arrangements in the buildings. Literature on the seismic behaviour/analysis of stepback and setback type of buildings is scanty. Literature on dynamic behaviour of these types of buildings and its analysis procedures which take into account the asymmetry in the buildings, i.e. torsional coupling effects, various analytical models for R.C. beam/column elements and panel elements for inelastic behaviour, including stiffness degrading models have been reviewed in this paper and conclusions drawn therefrom.

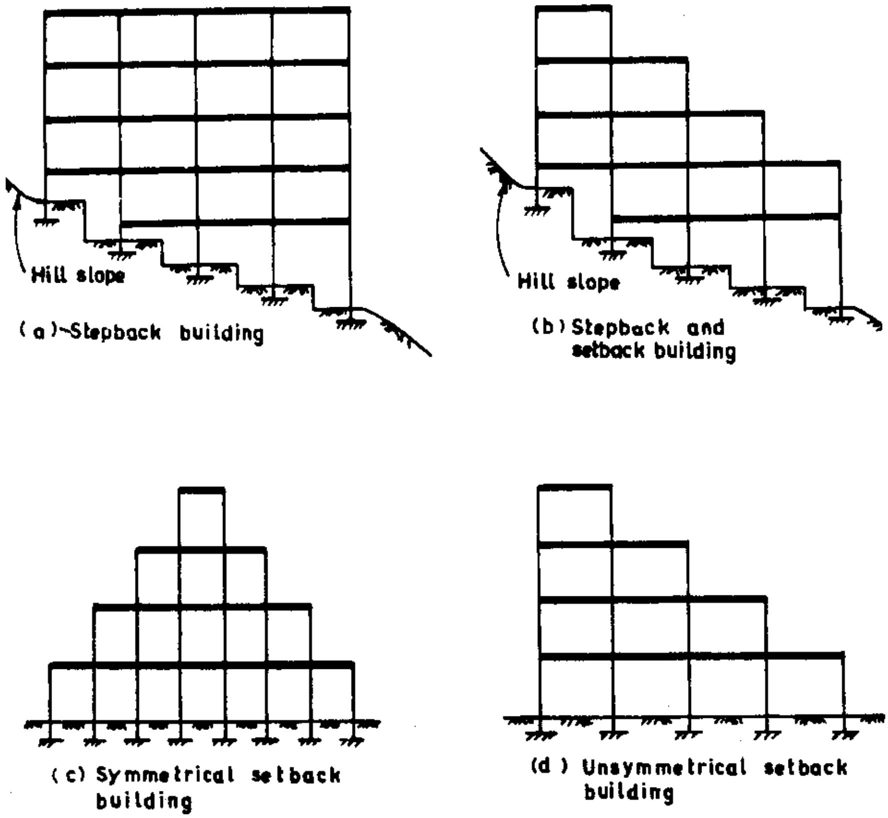


Fig. 1. - Stepback and Setback Building

SETBACK AND STEPBACK BUILDINGS

Berg(1962) studied the earthquake stresses in buildings with setbacks. In his study the building with setback has been represented by a rectangular cantilever shear beam with a setback at one location along its height. In the model the x-section is assumed uniform above and below the step. The rigidity per unit area and density are taken uniform throughout the height of the building but the rigidity is taken different in both the directions. The modal equations of motion are derived and computed modes are examined to show the effects of symmetric and unsymmetric setbacks upon the vibrational characteristics and upon the dynamic stresses induced by earthquakes.

Blume and Jhaveri(1969) investigated a highrise building with one or more setbacks involving special problems. The codes have dealt this problem by specifying the alternative to treat the tower portion of the building as a separate structure. The separate tower concept however does not take into account the fact that the ground motion is modified greatly by the base portion of the building before it affects the tower portion. The tower is subjected to an essentially harmonic forced vibration instead of the nearly random motion of the ground. The torsional and translational vibrations of a building with unsymmetrical setbacks is coupled in general. They carried out the analysis of a multistoreyed building with a setback.

The actual values of the code shear coefficients are found to be much smaller than the corresponding computed response values. The effect of setback on the base shear coefficient is strongly dependent on the characteristics of the ground motion and in turn these characteristics are significantly influenced by geologic conditions. It is essential in the case of setback structures to reconcile analysis and design procedures with the real earthquake problem and its probabilistic aspects.

Penzien(1969) Presented an approximate method of determining the peak seismic response of irregularly shaped buildings when subjected to base acceleration. Irregularly shaped buildings may have two contributing mode shapes with frequencies of nearly the same magnitudes. Two degrees of freedom system method has been applied to the lateral motion of buildings having large setbacks and to the coupled lateral motion of eccentric buildings. A comparative study of results has been carried out for the methods, i.e., two degrees of freedom system and single degree of freedom system. It is concluded that two degrees of freedom method accurately predicts the setback seismic coefficients C_{an} and torsion bending coefficients C_{tn} even when the period of the setback structure coincides with the fundamental period of the building and period of fundamental torsional mode of vibration equals the period of fundamental lateral mode of vibration. The single degree of freedom method is considerably in error when the period of the setback structure is near one of the lower periods of the building which supports it and when the period of the fundamental torsional mode is near the fundamental lateral vibration mode. A setback structure should be so designed that its fundamental period of vibration differs considerably from the first lateral vibration mode of the building and also does not coincide with the periods of other lower lateral vibration modes. The seismic forces developed in a setback structure and the seismic torsion forces developed in an eccentric building assuming elastic systems are much larger than standard code values. Therefore for determining seismic forces in setback structures and seismic torsion forces in eccentric buildings the desirable effects of inelastic deformations must be considered.

Pekau and Green(1974) investigated earthquake response of yielding frame structures with setbacks. Keeping in view the serious stress concentrations at the level of setback the effect of inelastic action is established. The base portion of the building consists of three equal bays for a fraction of overall height given by the level of setback. The tower portion is a single central bay where the storey sums of girder and column properties equal corresponding sums in the uniform three bay structures multiplied by degree of setback. The storey drift response is not sensitive to level of setback for relatively small towers. The tower and base shear coefficients both increase for decreasing size of setback. It is interesting to note that for high level of setback the base shear coefficient for setback and uniform structures tend to be the same. When the degree of setback is greater than 0.67 the presence of the setback has small effect.

Humar and Wright(1977) carried out analytical study of the dynamic behaviour of selected series of multistorey steel frames with symmetric setbacks. The models of the structure were having finite number of degrees of freedom with masses lumped at the floor levels. Conclusions derived are that the relative contributions of the higher modes to the base shear in general increase with decreasing tower base plan area for setback type buildings. The maximum interstorey drifts are substantially greater than the comparable responses for comparable uniform buildings in the inelastic range. The maximum shear coefficients are substantially greater than the comparable responses for comparable uniform buildings. Codes underestimate the distribution of base shear throughout the building height for setback buildings. In the setback buildings, the shear coefficients show a sudden and marked increase in the transition region between the base and the tower. For a setback building with the mass and stiffness substantially proportional to the plan area, the seismic response depends upon the ratio of the tower plan area to the base plan area, rather than upon the ratio of the plan dimensions of the tower and base in the direction of vibration as specified in 1973 SEAOC Code. There appears to be a strong correlation between general nature and distribution of the elastic and inelastic seismic responses of setback buildings. Thus a less expensive elastic analysis in most practical design applications can be employed.

Cheung and Tso(1987) presented a simple method for lateral load analysis of buildings with setback for preliminary design purposes. The concept of compatibility has been employed in this method. The lateral load acting on the structure is divided into two sub components. The sub component consists of applied load acting on the tower portion of the structure together with a set of compatible loads acting at and below the setback level. This part of the load is resisted by tower portion only. The compatible loads are to offset the effect of loadings above the setback ensuring compatibility between the tower and the base portion of the structure. The second component will then consist of applied loads at and below the setback level less the compatible loads. This second set of loads will be resisted by base portion only. The final response of the structure will be the sum of responses under each of the two loading sub components as discussed above. For eccentric setback structures, additional computation is necessary to take into account the torsional effect. The lateral loading is first subdivided into translational and torsional loadings. Then effect of the translational loading is worked out as in the symmetric setback structures after which the effect of the torsional loading is worked out. To accomplish this, locations of the centres of rigidity are to be worked out. Then distribution of torsional shear can be carried out by using the compatible concept again. The total response will be the sum of translational response and torsional response. This method can be used as design tool as well as it provides an insight into the load transfer mechanism involved in such type of structures especially in the region where setback occurs.

Sobaih, Hindi and Al-Noury(1988) studied the effect of different parameters on the nonlinear dynamic analysis of setback reinforced concrete frames. The parameters are setback level ratio 'L_S '; variation of beam properties; earthquake intensity and the type of nonlinear model. The tower portion exhibits larger displacements as 'L_S ' decreases compared with uniform frame. The response of such frames is affected by setback ratio, beam properties, earthquake intensity and nonlinear model used in the analysis.

Satake and Shibata(1988) carried out dynamic inelastic analysis for torsional behaviour of a setback type building using three dimensional frame model. First, the original designed building model is analyzed to see the torsional behaviour of a setback type building subjected to strong earthquake. Secondly, the modified model whose strength is modified as per the torsional response properties is analyzed and found that if the torsional response is not considered in the seismic design, it affects the response and damage distribution. The strength of each frame must be determined according to its torsional response properties to control the damage level. The distribution of elastic response shear can be used to determine adequate strength distribution. By taking into account the torsional response properties, the requirement of total strength can be reduced about 20% as compared with current design value.

Shahrooz and Moehle(1990) carried out the experimental and analytical studies of seismic response of setback structures. Only two dimensional response parallel to the setback was considered. The influence of setback on dynamic response, the adequacy of current static and dynamic design requirements for setback buildings and design methods to improve the response of setback buildings were the main points under consideration in the study. Only responses parallel to setback buildings are analyzed so that torsional effects do not arise. The results of the test structure were similar to those for a structure with regular configuration except torsion. The resulted behaviour of the structure using modal spectral analysis & static analysis design method did not differ notably. Both the methods were found inadequate to prevent concentration of damage in the members near the setback. For the setback structures, it was concluded that the design should be such which will impose increased strength on the tower relative to the base. A static analysis is proposed by the author which amplifies the forces. The ductility demand according to the proposed method is reduced considerably.

Wood(1992) investigated influence of setbacks on nonlinear response of R/C framed buildings. The displacement and shear responses of the setback frames were governed by effective first mode. Maximum top storey displacement and maximum interstorey drift for all frames increased with increasing ground motion intensity. However, the magnitude of displacement response and interstorey drift did not depend upon the frame profile as observed. Maximum storey displacement and interstorey drift were well represented by linear first mode shapes, The linear mode shapes for setback structures exhibit kinks, that were not present in case of uniform frames. But kinks did not influence the dynamic behaviour of setback frames. Maximum storey inertial forces and maximum storey shear were similar to the equivalent lateral force distribution used for design. Differences between nonlinear behaviour of regular and setback frames do not warrant the different design procedures to be adopted in the current building codes. There was no indication of concentration of forces or displacements in different stories with different mass or stiffness.

Jain and Mandal(1992) studied multistoried buildings with V-shaped plan by modelling each wing as a vertically oriented anisotropic plate for the motion in the transverse direction. All modes exhibit floor flexibility in case of unequal stiffness in transeverse and longitudinal directions. Both rigid floor as well as flexible floor modes existed in case of equal stiffness in transverse and longitudinal directions. In this study torsional stiffness of floors and frames is neglected. The modes involving floor deformations are not excited by a spatially uniform ground motion. Problem of stress concentration can be taken care of by designing the structure in such a way that longitudinal & transverse stiffness of the structure are equal. Various parameters like relative values of stiffness in the longitudinal & transverse direction of each wing, angle, aspect ratio and height to length ratio have very significant effects on the relative floor flexibility. Floor flexibility affects the shear distribution among transverse frames thus leading to unsafe design for some frames. If the total transverse stiffness is more than the longitudinal stiffness the first floor mode involves more deflection at the junction than that at the free end and vice-versa. As the angle between the wings increases(decreases), floor flexibility effects decreases (increases), These effects increase significantly with an increase in aspect ratio and with decrease in building height. Depending upon the configuration of the structure, floor flexibility may overload some of the transverse frames.

Paul(1993) suggested a simplified method for analysis of stepback and setback buildings by taking one d.o.f. per floor(i.e. translational either in x or y directions) and studied the hill buildings using this method. Results obtained by this method have been compared with 6 d.o.f. per node analysis.

Kumar and Paul(1994, 1996) developed a simplified method of dynamic analysis for irregular buildings such as on slopes, having stepback and setback configurations characterised by centre of mass of various floors of the building lying on different vertical axes and so is its stiffness, with 3 d.o.f. per floor assuming floor diaphragm as rigid This simplified method is based on the concept of transformation of mass and stiffness about a common vertical reference axis located anywhere in the space. The overall size of the problem is reduced tremendously. Accidental eccentricity can be taken into account by simply shifting the centre of mass in x and y directions by a distance equal to accidental eccentricity. The result obtained from present formulation are almost the same as obtained from 6 d.o.f. per node analysis with rigid floor diaphragm.

TORSIONAL COUPLING AND DYNAMIC BEHAVIOUR

Kan & Chopra(1977) have studied the torsionally coupled buildings in which centre of mass of all the floors lies on one vertical axis with 3 d.o.f. per floor with rigid floor diaphragm assumption and found that any lower mode of vibration of torsionally coupled building can be approximated as a linear combination of three vibration modes of the corresponding uncoupled system, i. e., the jth mode in translational vibration in x-direction, the jth mode in torsional vibration and jth mode in translational vibration in y-direction. This has facilitated the procedure to be simpler as compared to the standard procedure for analysing the response of torsionally coupled multistorey buildings to earthquake ground motion. Numerical examples have been solved and it is found that the approximate procedure is sufficiently accurate for purposes of design of most multistorey buildings. Idealized system has been shown in Fig.2. The effect of torsional coupling depends strongly on the ratio of natural frequencies for uncoupled torsional and lateral motions of the corresponding uncoupled system.

Humar & Wright(1977) studied dynamic behaviour of multistorey steel rigid frame building with setbacks. The steel frame was modelled as dynamic system having finite number of degrees of freedom with the masses lumped at the floor levels. In the setback type buildings maximum utilized girder ductility ratios are substantially greater than the comparable uniform building. It is evident that setback buildings with slender towers designed according to such codes may undergo serious distress in the tower portion when subjected to severe earthquakes.

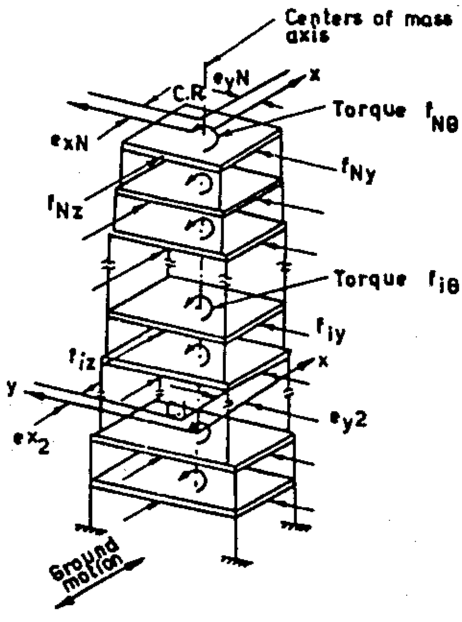


Fig. 2. - Idealized System for Setback Building

Rutenberg *et al.*(1978) proposed a scheme to calculate the effect of torsion on each lateral load resisting element of asymmetrical buildings in the context of the response spectrum techniques. The scheme consists of (i) obtaining the modal shear and torque on the building by the response spectrum technique and (ii) computing the total modal shear forces on each frame by resolving the modal shear and torque on the building according to principles of structural mechanics. Then the total shear force on each frame is obtained by combining the total modal shears on that frame in a root sum square manner.

Tso and Dempsey(1980) studied the dynamic torsional response of a single mass partially symmetric system to ground excitation. The torsional response and dynamic eccentricity are shown as functions of the eccentricity of the system and its uncoupled frequency ratio. It is shown that the dynamic eccentricity can best be expressed as a bilinear function of eccetricity for the critical ratio as unity. A comparison with the torsional provisions of five seismic Codes(Canada, Mexico, Newzealand, ATC3 and Germany) shows that the torsional moment and edge displacement of the system is underestimated by the first four Codes when the eccentricity is small and the uncoupled torsional and lateral frequencies are close.

Kan & Chopra(1981) studied the coupling of lateral and torsional motion under earthquake excitations for buildings where the centre of mass does not coincide with centre of resistance. It is found that effect of torsional coupling depend significantly on ratio of uncoupled torsional to lateral frequency. For relatively large value of this ratio these effects are simple and can easily be generalized. For systems with ratio equal or greater than 2 these lateral deformations are unaffected. The response is primarily in translation and for most buildings which are strong in torsion, yielding of the system is controlled primarily by the yield shear. After initial yielding the system has a tendency to yield further primarily in translation and behaves more and more like an inelastic single degree of freedom system, responding primarily in translation. The torsional coupling generally affects maximum deformation in inelastic system to a lesser degree compared to corresponding linearly elastic system.

Volcano(1982) studied the influence of the structural properties and earthquake features on differently defined ductility requirements and it is observed that damage level was similar for the structures with differently defined ductility factors. Weak or stiff structures have greater ductility requirements. Duration of earthquake ground motions causes an increasing effect on the ductility factors which account for hysteretic energy. Hardening gives generally a more uniform distribution of ductility requirements but in some systems, total damage can increase inspite of an increase of hardening ratio. Softening produces a detrimental effect on the structures. The viscous damping produces a reduction of the mean and maximum ductility requirements.

Tso and Sadek(1985) studied the ductility demand and the edge displacement of a simple eccentric model in the inelastic range. It is found that unlike elastic response the coincidence of uncoupled torsional and lateral frequencies does not lead to exceptionally high inelastic response. It was also found that the system eccentricity has a large effect on ductility demand than earlier studies indicated. Eccentricity has the effect of increasing the edge displacement of the structure by a factor upto three when compared with that of a symmetrical system. Increase in torsional stiffness of the structure tends to reduce this factor.

Bozergnia and Tso(1986) studied the inelastic earthquake response of a one way torsionally coupled system subjected to two types of ground motion excitations. The effects of eccentricity, yield strength, uncoupled torsional to lateral frequency ratio and uncoupled lateral period on the response of the system were examined. The ductility demand on the critical element in the eccentric model can be upto about three times that for the corresponding symmetrical systems. Asymmetry affects the right edge displacement more than it affects the element ductility demand, especially for torsionally flexible structures. The ductility demand does not depend much on uncoupled torsional to lateral frequency but edge displacement is more sensitive to this ratio especially for stiff structures with low yield levels. It is shown that the stiff eccentric structures are vulnerable to such high ductility demand. Therefore the design strength of stiff eccentric buildings should not be reduced from the elastic strength demand.

Costa, Oliviera and Duarte(1988) studied the buildings exhibiting the vertical irregularities. The building was idealized as a set of plane moment resisting frames connected to shear walls by rigid diaphragms. Nonlinear behaviour for both the frames and walls were considered. It is found that ductility demand distributions are irregular in shear walls but fairly regular in the frames except for storeys immediately above a discontinuity, where there is a significant increase in the frame ductility demand. The ductility demands in the frame and shear wall are almost the same for regular building. For irregular buildings the ductility demand can be nearly twice the ductility demand for regular buildings. In general if irregularity occurs in frame then the ductility demand is increased in shear wall and if irregularity occurs in shear wall than the increase in ductility demand is observed in frame.

Sobaih, Hindi and Al-Noury(1988) studied the effect of different parameters on the nonlinear dynamic analysis of setback reinforced concrete frames. The parameters are setback level ratio`L_S' defined as L/L' where L is total height of the frame and L' is the height of the base portion of the frame; variation of beam properties; earthquake intensity and the type of nonlinear model. Maximum interstorey drifts occur at the intermediate floors for L_S =0.375. At upper floors ductility demand for beams increase as L decreases. Also ductility demand for external columns may exhibit larger values as L_S decreases, as shown in Fig. 3. The response of such frames is affected by setback ratio, beam properties, earthquake intensity and nonlinear model used in the analysis.

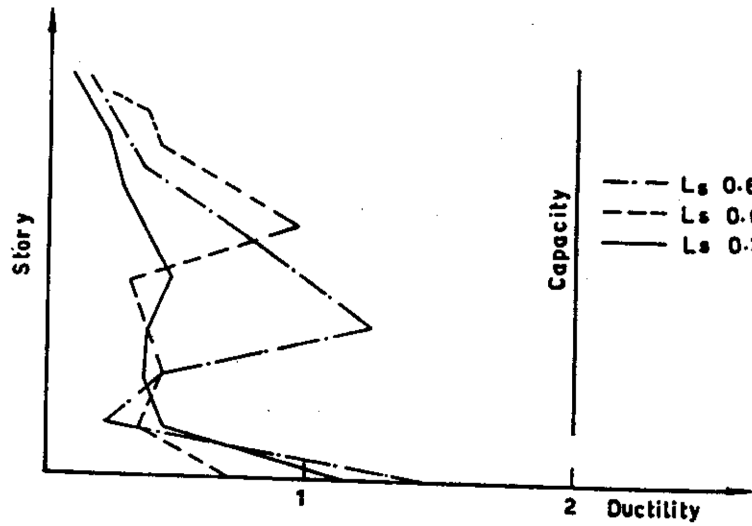


Fig. 3. - Column Ductility Demand for Unsymmetrical Frame

Hejal and Chopra(1989) studied the effects of lateral torsional coupling on the earthquake response of multistorey buildings. The effects of lateral torsional coupling on the responses of multistorey building and its associated one storey system are similar. It causes a decrease in the base shear, base overturning moment and top floor lateral displacement at the centre of rigidity, but an increase in the base torque. These effects are directly dependent on e/r ratio. Torsional coupling effects in the response of multistorey buildings depend on ρ (i.e. beam to column stiffness ratio). The effects of lateral torsional coupling on the height wise variations of forces is not very significant. It is more pronounced for storey shears and storey torques than storey overturning moments. Lateral torsional coupling affects the response spectra also for torsionally stiff systems the effect is very small but for torsionally flexible system the effect is significant. For torsionally stiff systems with closely spaced uncoupled frequencies and larger e/r values, the base torque at the centre of rigidity is approximated by the quantity $V_{BO}e_1w_1$. The product of base shear V_{BO} in the corresponding torsionally uncoupled multistorey system, e_1 is the effective eccentricity and $w_1$is the effective wt. in the fundamental vibration mode of the associated one storey system normalized by its total wt.

Prasad and Jagdish(1989) presented the inelastic response of single storey structure, square in plan supported on four columns subjected to earthquake assuming that the model is having three degrees of freedom per floor. The responses of the structure to simultaneous action of the two orthogonal horizontal components of the ground motion and to one of the two components have been compared.The torsion does not seem to significantly influence the result for small eccentricities, but for the large eccentricities like e/a=0.3, the maximum ductility demand of some columns reduced due to torsion, the largest of maximum ductilities is increased. This increase ranges from 5% to 50% when compared with zero eccentricity case. Disparity between largest of the maximum ductilities to the smallest of the maximum ductilities increases with eccentricity for the two components while it decreases with eccentricity for one component input. For zero eccentricity case torsional response was noticed for two components input and not for one component case. The response spectra has showed that the columns in short period structures experience larger ductilities.

Hamzeh *et al.*(1990) investigated inelastic response of torsionally coupled systems to an ensemble of real earthquake records in terms of system parameters such as lateral frequency, uncoupled torsional to lateral frequency ratio and eccentricity of the system. Angle of incidence of earthquake has significant effect on both ductility ratio and torque, especially in the low period range. For the smaller period range the system ductility ratio, the system torque and ductility ratios for the weakest column are significantly influenced by ratio of torsional and lateral frequency for the two component earthquake.

Goel and Chopra(1991) presented the influence of system parameters, uncoupled lateral vibration period, uncoupled torsional to lateral frequency ratio, stiffness eccentricity, relative values of strength and stiffness eccentricity, yield factor on the inelastic response of one storey asymmetric plan systems to two excitations. It is found that the torsional deformation of elastic as well as inelastic systems tends to increase with increasing stiffness eccentricity e_s /r and decreasing frequency ratio Ω_θ over a wide range of structural vibration periods. For very long period, displacement sensitive elastic systems, the torsional deformations tends to zero regradless of e_s /r and Ω_θ values. The lateral deformation of elastic as well as inelastic systems generally decreases with increasing e_s /r and decreasing Ω_θ. The element deformation of elastic systems is affected more by e_s /r and Ω_θ compared to the inelastic systems. It is also concluded that the torsional deformation of the system decreases if it is excited well into the inelastic range. Inelastic action influences the largest of peak deformations among all resisting elements of systems in a manner similar to the way it influences the lateral deformation. The ratio of the lateral deformations at the C_S of inelastic and elastic asymmetric plan systems is significantly different than symmetric plan systems, then the effects of plan asymmetry are significant. The ratio of element deformations for inelastic and elastic systems is affected by plan asymmetry to a greater degree compared to the ratio for deformation at C_S and is smaller for asymmetric plan systems.

Maheri, Chandler and Bassett(1991) tested models designed with variable ratios of torsional to lateral stiffness and with both symmetric and asymmetric mass distributions under earthquake base loading and it was concluded that the analysis of dynamic structural properties leads to very accurate predictions of frequencies and mode shapes. The analysis showed that earthquake response in the asymmetric cases is dominated by first mode but the experimental results showed that the second mode is much more significant than the theory predicts. In torsionally coupled structures, the theory overestimates the contribution of the first mode. The difference between theoretical and experimental responses identified are particularly significant for structures with low uncoupled frequency ratio R_f. The experimental results and conclusions drawn in comparison with theoretical predictions are considered to be widely applicable to seismic analysis and design of asymmetric multi-storey frame structures.

Chandler and Duan(1991) evaluated the factors which affects the inelastic seismic performance of torsionally asymmetric buildings. It is shown that Mexico 76 Code torsional provisions are inadequate and on other hand Mexico 87 Code torsional provisions are over conservative. It was found that the element at the stiff edge is the critical element which suffers severe damage than the corresponding symmetric structures. The peak ductility demand of the element at the flexible edge is always lower than that of corresponding symmetric ones. It has been recommended that the design eccentricities expressions of Mexico 87 Code be changed to $1.5e_s + 0.1b$ and $0.5e_s - 0.1b$ It will lead to minimum strength design in resisting elements.

Rutenberg, Benbenishti and Pekau(1992) presented a parametric study of earthquake response of single storey asymmetric structures designed by the static provisions of various codes. It is shown that SEAOC/UBC and NBCC designs lead to lower ductility demand than the ATC/NEHRP and CEB designs. The presence of elements normal to the direction of excitation usually moderates peak ductility demand displacement and rotation but the effect is not appreciable. In the asymmetric systems design results in larger ductility demand than in symmetric systems. Ductility demand response is affected by the type of model chosen. Increase in torsional to lateral frequency ratio tends to lower peak ductility demand. The maximum displacement of asymmetric systems is larger than that for the similar symmetric system and the factor of 2 is possible.

Nassar, Osterass and Krawinkler(1992) presented seismic design based on strength and ductility demand. It highlights the significance of ductility in seismic performance. Serviceability and collapse limit states has been expressed explicitely. Ductility capacity is the basic seismic design parameters for collapse limit state. Statistically inelastic response of SDOF and MDOF systems provides a means of developing strength design criteria based on ductility capacity. This approach is more complicated than the current code approach.

Goel and Chopra(1992) evaluated the effects of plan asymmetry on earthquake response of code designed one storey systems to know that how these effects are well represented by various building codes. It was concluded that the stiff side element with design force smaller than its symmetric plan value experienced increased ductility demand because of plan asymmetry. The ductility demand on the flexible side element is significantly smaller than in the symmetric plan system. Asymmetric plan systems with reduction factor R=1 may experience structural damage due to yielding and non structural damage resulting from increased deformations. Present building codes does not ensure that deformation and ductility demands for symmetric and asymmetric plan systems are similar. It is also concluded that additional deformations due to plan asymmetry cannot be reduced by modifying the design eccentricity in the codes.

Zhou and Minoru(1992) presented pulse response analysis to evaluate the maximum responses of asymmetric structures and is applied to an idealized monosymmetric system. Results were compared with those given by time history analysis and it was concluded that proposed procedure gives reasonable estimate of responses.

Boroschek and Mahin(1992) presented dynamic torsional behaviour of an existing building that responded severely during service level earthquake. Parametric studies are carried out on linear and nonlinear models of the building. The torsional behaviour in regular structures increases stress and ductility demands in element located away from the centre of rotation and translational displacement are also affected. These effects are influenced by the characteristics of the input ground motion and these effects are more severe for elastic structures than inelastic structures and are highly dependent on the characteristics of the input ground motion.

Fukada, Kobayashi, Adachi, Nagata and Hayashi(1992) presented the results of vibration tests performed on the building after its completion and result of simulation analysis. It was found that the plane frames with different heights in a setback building are to be given the same lateral rigidities to their own weights. Then the natural vibration periods of such structures are shorter than those of design analysis due to the fact that additional rigidities due to non structural elements give their own contribution. When the vibration amplitudes are larger, then these additional rigidities disappear.

Ayala, Garcia and Escobar(1992) evaluated the seismic performance of nonlinear asymmetric building structures with resisting elements in one and two orthogonal directions and the suitability of different design recommendations. Seismic performance is measured by the ratio of maximum ductility demand for asymmetric structures to the maximum ductility demand for the corresponding symmetric ones. The torsional response of the building structures is significantly affected by the in-plan distribution of the strength. The coefficients involved in the design eccentricities recommended in the current code for Mexico city which follows the distribution of mass, it may lead to the values of performance indexes in excess of those considered adequate. To keep the values within the acceptable limits, the design coefficients are to be modified in such a way that the torsional overstrength is kept constant, the interstorey resisting force is moved toward a position between C_m and C_s.

Corderoy and Thambiratnam(1993) presented a simple method for earthquake analysis of torsionally coupled setback buildings on flat grounds. The analysis uses the shear beam model in which floors are assumed to be rigid, each with three degrees of freedom. The system can be analysed elastically or elasto-plastically. The whole procedure has been programmed in such a manner that any degree of asymmetry can be taken care of. The sequence of columns yielding demonstrated the effect of asymmetry. The columns closest to the floor centroids yield first. The time step has been selected 1/10th of the structure's fundamental period for elastic analysis, For elasto-plastic range the time step must be smaller than this. In the design situations the column stiffness and strength are to be chosen such that behaviour of the building is consistent with the function to be performed by the building under consideration.

Cruvellier and Smith(1993) presented a method for static and dynamic analysis of 3 dimensional asymmetric buildings composed of intersecting bents of any structural type by modelling it in two dimensions. The two dimensional model is simple which neither requires pre nor requires post analysis transformations. In this method, the engineer is forced to understand structural action in order to translate it from three to two dimensions. It is more relevant to educational considerations rather than to the practical purposes.

Correnza *et al.* **(1994)** analysed a series of models subjected to both uni and bi directional ground motion input and found that for the flexible edge element, accurate estimates of additional ductility demand arising from torsional effects may be obtained from uni-directional models only for medium range to long period systems. These estimates may be over conservative for short period systems, which constitute a large proportion of system for which Code static torsional provisions are utilized. It is further concluded that models incorporating the transverse elements but analysed under uni-directional lateral loading may under estimate by up to 100% the torsional effects in such systems, but are reasonably accurate for medium and long period structures.

Llera ***et al.*****(1994)** studied the accidental torsion effects in buildings due to stiffness uncertainty. Symmetric plan buildings can be asymmetric due to the discrepancies between the computed and actual values of the structural element stiffness and undergo torsional vibrations under the effect of purely translational ground motion. Such accidental torsion leads to increase in structural element deformations which is shown essentially insensitive to the uncoupled lateral vibration period of the system but is strongly affected by the ratio of uncoupled lateral and torsional vibration periods. It has been found that the structural deformations due to stiffness uncertainty is shown to be much smaller than implied by the accidental torsional provisions in the building code and most other building codes.

Wong and Tso(1994) studied the inelastic seismic response of the torsionally coupled unbalanced structural systems with strength distributed using elastic response spectrum analysis. It has been shown that inelastic responses depend strongly on the torsional stiffness of the system. For torsionally stiff system, the torsional response leads to decrease in the stiff edge displacement but for torsionally flexible systems, the torsional response leads to increase in the stiff edge displacement by taking accidental torsion effects into account, the response spectrum analysis gives strength distribution such that there is no excessive ductility demands on the lateral load resisting elements.

Llera ***et al.*****(1994)** studied the differences between the increase in building response due to accidental eccentricity predicted by Code specified static and dynamic analysis for symmetric and unsymmetric single and multistorey buildings. Upper and lower bounds for differences in response computed from static and dynamic analysis are obtained for general multistorey systems. These differences in response primarily depend upon the ratio of the fundamental torsional and lateral frequency of the building. These are larger for small values of the frequency ratio and decrease to zero as the frequency ratio becomes large. The discrepancies between the increase in response due to accidental eccentricity predicted by dynamic and static analyses is in many cases of the same order of magnitude as the response increase itself. This suggests that the Code specified static and dynamic analysis to account for accidental torsion should be modified to be mutually consistent.

Llera *et al.*(1995) Suggested the simplified model for analysis and design of multistorey buildings based on single super element per building storey by matching the stiffness matrices and ultimate yield surface of the storey with that of the element. The errors in peak responses are expected to be less than 20% for most practical structures. The model uses an accurate representation of storey shear and torque surfaces, which capture the fundamental features controlling the inelastic behaviour of the building.

CODAL PROVISIONS OF VARIOUS COUNTRIES

Australian Standard 2121-1979 recommends that where a regular building or framing system has one setback in which the plan dimension of the tower in each direction is at least 0.75 times the corresponding plan dimension of the lower part, such a building may be considered as being without a setback for the purposes of determining and distributing earthquake forces. Buildings with other conditions of setback in either zone A or 1, the tower shall be designed as a separate building using the larger of values of the seismic response factor C at the base of the tower determined by considering the tower as a separate building for its own height or as part of the overall structure. The resulting shear from the tower shall be applied at the top of the lower part of the building which shall be otherwise considered separately for its own height. For buildings with other conditions of setback shall be analysed by considering the dynamic characteristics of such buildings. Horizontal torsion can be accounted for by taking the design eccentricity e_d as $1.7e_s-e_s^2/b+0.1b$ or $e_s-0.1b$ where b is the maximum lateral dimension of the building perpendicular to the horizontal loading direction under consideration and e_s is the static eccentricity.

National Building Code of Canada(1990) specifies that where the centroids of mass and the centres of stiffness of the different floors do not lie approximately on vertical lines, a dynamic analysis shall be carried out to determine the torsional effects. A setback is a sudden change in plan dimension or a sudden change in stiffness along the height of a building. The effects of major changes in stiffness and geometry are best investigated by dynamic methods. The design eccetricity for regular asymmetric structures has been specified as $1.5e+0.1D_n$ or $0.5e-0.1D_n$ where D_n is the plan dimension of the building in the direction of computed eccentricity, e is the distance between the location of the resultant of all the forces at and above the level being considered and the centre of rigidity at the level being considered.

National Standard of People's Republic of China Aseismic Building Design Code GBJ 11-89 says the effect of structural torsion can be taken into account by assigning 3 d.o.f. per floor; i.e. mutully orthogonal two components of translation and one component of rotation.

Seismic loads and actions are correspondingly evaluated by means of the spectral and modal methods.

Japan Earthquake Resistant Design Method for Buildings specifies that a coupled system consisting of appendage and the main structure must be analyzed according to modal analysis procedure which includes evaluation of natural periods and associated oscillating modes for a structural model.

New Zealand Standard NZS 4203:1992 says that a three dimensional modal analysis or a three dimensional numerical integration time history analysis shall be used for structures having horizontal and vertical irregularity.

Swiss Standard SIA 160 (1989) says that if the plan layouts of the structure and the distribution of mass are not approximately symmetric and with significant discontinuities throughout the height of the structure, then a more detailed method of calculation shall be used, such as the response spectrum method.

Regulations for Earthquake Resistant Design of Buildings in Egypt - 1988 Code of Practice for Loading, Ethiopia ESCPI-1983, Indonesian Earthquake Code 1983, National Structural Code for Buildings Philippines recommends that for buildings with setback where the plan dimension of the tower portion in each direction is at least 75 percent of the corresponding plan dimension of the lower part, the effect of the setback may be neglected for the purposes of determining seismic forces by the equivalent static force method. For other conditions of setback in buildings the detailed dynamic analysis is to be carried out.

I.S.1893-1984 recommends that buildings having irregular shape and or irregular distribution of mass and stiffness in horizontal and/or vertical plane shall be analysed by modal analysis and torsional shears are to be accounted for separately by taking eccentricity equal to 1.5 times the static eccentricity between the centre of mass and centre of stiffness. Negative torsional shears are to be neglected.

Earthquake Resistant Standards National Regulations of Construction Peru recommends that if the reduced dimension in plan is not less than 3/4 parts of the dimension of the immediate lower story in the direction in which the earthquake is considered, the force H shall be calculated and shall be distributed in height according to the usual practice. Similarly if the base of the building with reduction has the height less or equal to 30% of the total height of the building, it shall be considered that the reduction will not modify the distribution of H force. If the reduced dimension in plan is less than the 3/4 parts of the dimension of the immediate lower story in the direction considered, the reduced part shall be determined at its base according to the following criteria:

(a) In the case when the reduction is between 50% and 75%, the reduced part will be treated like one independent tower and the base shear force will be determined according to its base multiplied by an amplification factor of 1.25.

(b) In the case when the reduction is more than 50%, the reduced part will be treated like one independent tower and the base shear force will be determined according to its base multiplied by an amplification factor of 1.5.

(c) To the base of a building considering as a whole with the reduction referred above shall be applied the shear force calculated according to the indication of the same paragraph adding the forces that will be determined for this lower portion, as indicated above.

Uniform Building Code of U.S.A.(1991) recommend that structures having irregularity of the type as (1) a story in which the lateral stiffness is less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above (2) where the effective mass of any story is more than 150 percent of the effective mass of an adjacent story, a roof which is lighter than the floor below need not be considered.(3) Where the horizontal dimension of the lateral resisting system in any story is more than 130 percent of that in an adjacent story, one story penthouses need not be considered, are to analysed with dynamic analysis. A three dimensional model shall be used for the dynamic analysis of structures with highly irregular plan configurations such as those having a plan irregularity and having a rigid or semi rigid diaphragms. Accidental eccentricity is defined as 5% of the plan dimension in the direction of stiffness eccentricity.

ANALYTICAL MODELS

3D R.C. Frame Members

Buildings with irregular shapes are highly torsionally coupled under the action of lateral loads such as earthquake loads. Therefore analysis of a structure is to be carried out in the 3D space. This requires the modelling of the members such as(i.e. beams/columns) should be three dimensional in nature. Therefore r.c. frame members should be modelled as three dimensional frame members with 6 d.o.f. per node of the structure. Different models of 3D r.c. members are presented by various authors for non linear analysis of building frames under earthquake loading.

Nigam(1967) presented nonlinear analysis of frame structure under dynamic loading. The yield condition of the member is governed by the interaction of different stress resultants. The plastic deformation after yielding of the section are assumed to be concentrated on single section that is a plastic hinge of zero length is assumed to exist. The plastic flow is assumed to be along the gradient of the yield function evaluated at the point representing the current state of the stress resultants.

Tseng and Penzien(1975) presented a model consisting of two parts in series, linear and nonlinear. The nonlinear part is assumed to be concentrated at the beam ends in the form of 3D plastic hinges. The elastic-plastic tangent stiffness matrix has been derived using the generalized yield function $F(P_u, M_{yu}, M_{zu})$ and the associated flow rules for elasto-plastic solid section.

Takizawa and Aoyama(1976) introduced a model which takes the interaction of biaxial bending moment and stiffness degrading effects. Prager-Ziegler kinematic hardening theory was used to shift from the cracking stage to the yield stage.

Gillies and Shepherd(1981) presented a three dimensional elasto-plastic model. The inelastic actions were confined to the ends of the element. Rigid end blocks were specified to simulate the joint core zones. Two rotational springs were provided at each end of the beam elements to model the flexural behaviour along the local y and z axes. This also includes axial spring which has the capability to represent the axial yield under combined bending and axial tension or compression.

Lai *et al.*(1984) presented a model consisting of two inelastic elements at the two ends of a reinforced concrete member sandwiching a linear elastic line element as shown in Fig. 4. For each inelastic element there are four inelastic springs at each of the four corner regions with a fifth spring at centre of the section. Each of the four exterior springs represents the stiffness of the effective reinforcing steel and effective compression concrete. The fifth spring has only one component which is from the effective concrete in centre region. Inelastic behaviour is fully concentrated within the inelastic elements located at two ends of a member.

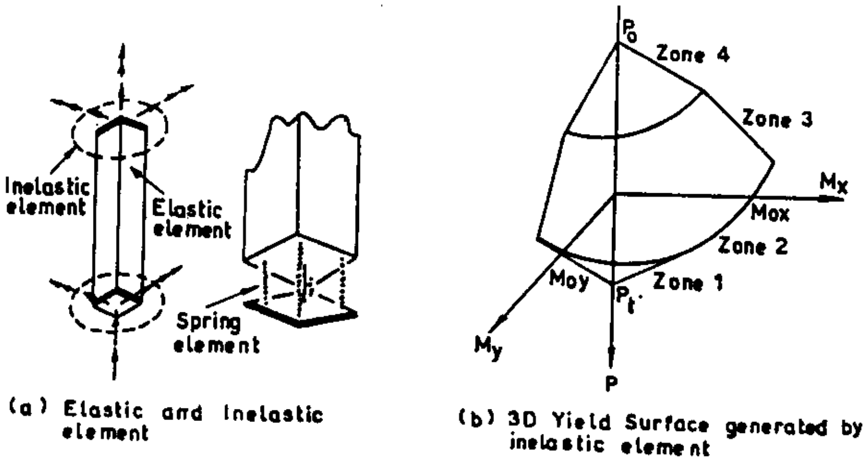
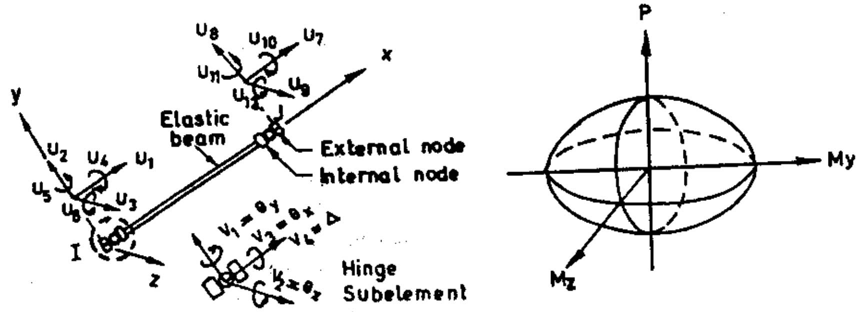


Fig. 4. - Inelastic Model for 3D R/C Member: Lai etal(1984)

Powell and Chen(1986) presented a 3D beam column element with a generalized plastic hinge as shown in Fig.5. This model takes into account the interaction of axial forces, biaxial bending moments and torsion. This model also takes into account the stiffness degradation and the rate dependent material properties.

Sfakianakis and Fardis(1991) presented a new model for r.c. columns subjected to a cyclic biaxial bending with axial force. Its basic constituent is the tangent flexibility matrix of a section which relates the set of increments of the 3 normal stress resultants P, M_y, M_z to that of the corresponding section deformation. This incremental flexibility relation is based on the bounding surface concept which is constituted as the locus of points (P, M_y, M_z) at ultimate strength of the r.c. cross section.



(a) Elastic and inelastic (hinge) subelement

(b) Force moment yield surface

Fig. 5. - Inelastic Model for 3D R/C Member: Powell and Chen

Thanoon(1993) proposed yield criteria for 3 D r.c. frame members taking interaction of axial forces, bending moments M_y and M_z and torsional moment T. Shear forces effects have been neglected. The yield function is described as

20

$$(m_y/m_{yp})^2 + (m_z/m_{zp})^2 + (t/t_{uo})^2 = 1.0 \tag{1}$$

$$\text{where } m_{zp}/m_{zo} = a_1+a_2(p_u/p_o)+a_3(p_u/p_o)^2+a_4(p_u/p_o)^3 \tag{2}$$

$$\text{and } m_{yp}/m_{yo} = b_1+b_2(p_u/p_o)+b_3(p_u/p_o)^2+b_4(p_u/p_o)^3 \tag{3}$$

where m_{zo} and m_{yo} are the dimensionless values of the ultimate bending moment capacities of a section about z and y axes respectively, when the axial force is equal to zero. m_{zp} and m_{yp} are the dimension less values of the ultimate flexural strengths about the z and y axes respectively, for a fixed value of P_u. The constants a_1 to a_4 and b_1 to b_4 are the polynomial constants.

Kumar(1996) proposed yield criteria for 3D r.c. frame members taking interaction of axial forces, bending moments m_y and m_z and torsional moments T_x and shear forces V_y and V_z. The following equations are used in simulating the interaction of moments, torsion, axial forces and shear forces.

$$(m_y/m_{yp})^2 + (m_z/m_{zp})^2 + (T_x/T_{xv})^2 = 1.0 \tag{4}$$

$$\text{where } m_{zp}/m_{zo} = a_1 + a_2(P_x/P_o) + a_3(P_x/P_o)^2 + a_4(P_x/P_o)^3 \tag{5}$$

$$m_{yp}/m_{yo} = b_1 + b_2(P_x/P_o) + b_3(P_x/P_o)^2 + b_4(P_x/P_o)^3 \tag{6}$$

$$(T_{xv}/T_{xo})^2 + (V_z/V_{zo})^2 + (V_y/V_{yo})^2 = 1.0 \tag{7}$$

Here m_{zo} and m_{yo} are the dimensionless ultimate bending moment capacities of the section about z and y axes respectively when the axial force is zero. T_{xo} is the dimensionless ultimate torsional moment capacity of the section about x - axis when the shear forces V_z and V_y are zero. m_{yp} and m_{zp} are the dimensionless values of the ultimate flexural strengths about z and y axes respectively for a fixed value of axial load P_u. T_{xv} is the dimensionless values of the ultimate torsional strength under the action of fixed value of V_z and V_y, shear forces along z and y axes respectively. The a_1, a_2, a_3, a_4 and b_1, b_2, b_3, b_4 are the polynomial constants combining all the above four equations gives following yield surface.

$$f(P_x, m_y, m_z, T_x, V_z, V_y, P_o, M_{zo}, M_{yo}, T_{xo}, V_{yo}, V_{zo}, ai, bi) = 1 \quad (8)$$

It is observed that the various elements of the unsymmetrical buildings are subjected to all the six components of forces/moments. Therefore the yield criteria should take interaction of all the six components of forces to get the realistic results.

Stiffness Degrading Models

The inelastic analysis of reinforced concrete buildings subjected to strong earthquake motions requires a realistic model which takes into account the continually varying stiffness and energy absorbing characteristics of the structure due to the inelastic behaviour of the structural members under strong motions. Various models available in the literature are described briefly here.

Takeda *et al.*(1970) presented a model by defining a primary curve for initial loading and a set of rules are described for reversal loading. The primary curve is characterised by three linear segments, such as cracking point, yielding point and point beyond yielding. depending upon the loading state the set of rules are prescribed for reversal loading to take its paths. Seven condition hysteretic model has been formulated.

Imbeaut and Nielson(1973) suggested the use of degrading bilinear model. This model uses a deteriorating elastic stiffness that represents the average value of the unloading and reversed loading stiffness. The reduced elastic stiffness is a function of the maximum displacement ductility and is given as $K=K_g(1/\mu_d)^{\alpha}$ in which μ_d = displacement ductility = D_{max}/D_{yield}, α=constant $0.5<\alpha<0.6$,K_g=initial stiffness based on gross section.

Anderson and Townsend(1977) presented two degrading stiffness models. The first referred to as the degrading trilinear model (DTL) has the following properties. The initial loading branch and all unloading branches have stiffness K_g based on the gross section properties. The strain hardening branch has a stiffness = 0.03 K_g and reversed loading branches have stiffness = $K_g(1/\mu_c)^{1.5}$ in which μ_c is the maximum curvature ductility of the member. The second model is referred to as a degrading trilinear connection model. In this the initial loading branch stiffness and subsequent unloading branch stiffnesses of the hinge element=$K_g/4$. This takes into account the loss of stiffness produced by concrete cracking in the beam and joint rotation generated by bond failure around the longitudinal beam steel anchored in the joint core.

Chen and Powell(1982) took stiffness degradation into account when reversed loading is applied. It is assumed that the stiffness degrades independently for each force component of each subhinge in inverse proportion to the largest previous hinge deformations. The unloading stiffness K_1' for cracking hinge and K_2' for yielding subhinge depend on the previous maximum positive and negative hinge deformation and are controlled by input coefficients α_1 and α_2 for

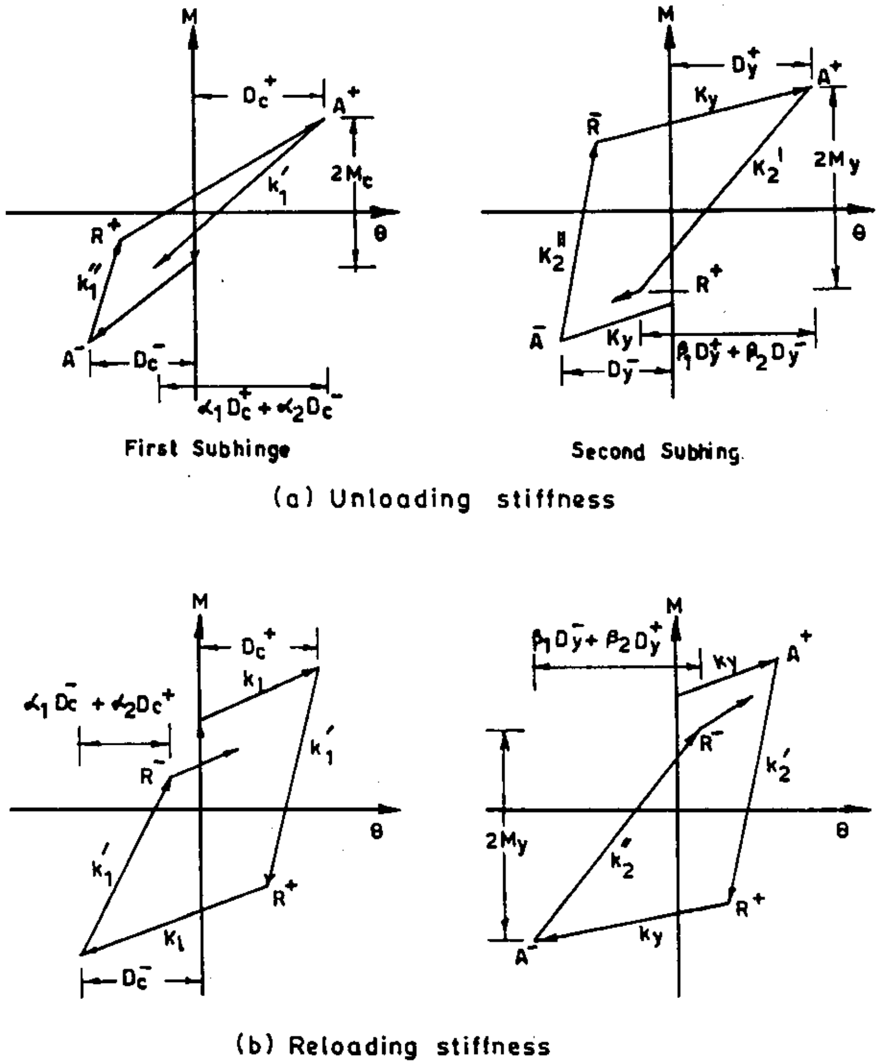
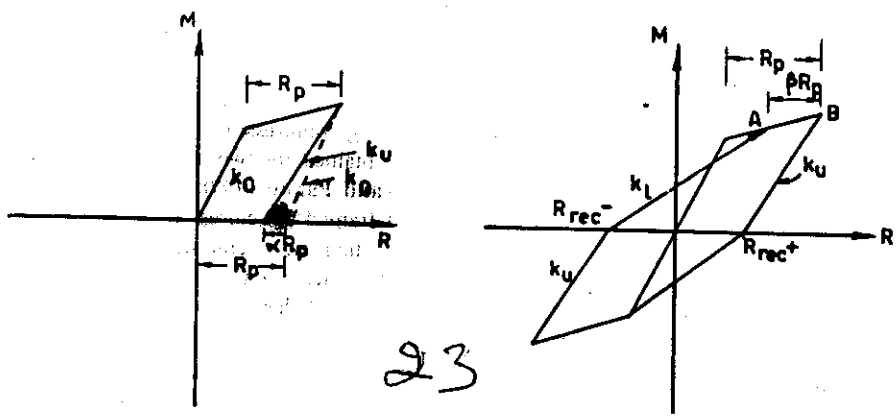


Fig. 6 - Stiffness Degrading Model

cracking subhinge and β_1 and β_2 for yielding subhinge. These coefficients control the reloading stiffnesses $K_1^{''}$ and $K_2^{''}$ also as shown in Fig. 6.

Allahabadi and Powell(1988) introduced stiffness degrading model for r.c. beams under cyclic loads. Strain hardening and degrading flexural stiffness are approximated by assuming that the element consists of a linear elastic beam element with nonlinear rotational springs at each end. All plastic deformation effects including the effects of degrading stiffness are introduced by means of the moment rotation relationships for the hinge rotations. The moment rotation relationship for each hinge is an extended version of Takeda's model which has the behaviour as shown in Fig.7. The extension to Takeda's model are (i) a reduction of unloading stiffness by an amount which depends on the largest previous hinge rotation. (ii) Incorporation of variable reloading stiffness which is larger than that of the Takeda's model and also depends upon the past rotation history.



(a) Unloading Stiffness

(b) Reloading Stiffness

Fig. 7. - Extended Takeda's Model

The unloading stiffness K_u depends on the maximum hinge rotation and is controlled by input parameter α as shown in Fig. 7(a) and its value varies from 0.0 to 0.4. The reloading stiffness as controlled by input parameter β as shown in Fig.7(b) and its value varies from 0.0 to 0.6.

Kunneth *et al.*(1990) presented the hysteretic model which uses three parameters α, β, γ in conjuction with nonsymmetric trilinear curve. Stiffness degradation represented by α is introduced by setting a common point on the extrapolated initial stiffness line and assumes that unloading lines target the point until they reach the x-axis, after which they aim the previous maximum or minimum points. Pinching behaviour is introduced by lowering the target maximum or minimum point to a straight level γP along the previous unloading line. Reloading lines now aim this new point untill they reach the crack closing point after which they target the previous maximum or minimum point. Strength degradation is introduced by parameter β.

Panel Elements

Reinforced cement concrete framed building structures consists of r.c. space frame with brickwork or concrete block masonry infills and stiffened by floor slabs acting as rigid diaphragms. Slabs and infill panel increases the load carrying capacity of the r.c. framed structures. The analysis of the above system may be carried out either on approximate basis or on finite element basis. To analyse the structure into elastic range the approximate methods are sufficient and to extend the analysis into inelastic range the finite element method is to be employed. The various modelling techniques available in the literature for panel elements is briefly described here.

Holmes(1961) proposed the concept of panel element as an equivalent compression strut of thickness equal to that of the panel and a width equal to one third of the length of diagonal of the panel. Effective elastic modulus for the equivalent strut were computed on the basis of various tests. **Smith(1962,66,68)** proposed that the width of equivalent strut depends upon the loading applied, relative stiffness of the frame and the infill.

Mallick and Severn(1967) used finite element analysis with rectangular finite element for the panel and a number of link elements capable of taking compression and shear for interface element between the frame and the infill. The results obtained for two storeyed infilled frame were found to match with experimental results. **King and Pandey(1978)** described procedure based on finite element method for analysing infilled framed structures. It is shown that the fairly coarse finite element meshes can be used for finding the lateral stiffness of the single frame. The infill is idealised as four noded rectangular elements with 2 degrees of freedom at each node. **Liauw and Kwan(1984)** examined the nonlinear behaviour of non integral infilled frames using finite element method. It was shown that the stress redistribution towards collapse were significant and the strength of non integral infilled frames was very much dependent on the flexural strength of the frame. **May and Naji(1991)** carried out nonlinear analysis of infilled frames. The frame members has been modelled with 3-noded frame elements, and panel elements has been modelled as 8-noded element. A 6-noded interface element has been used to model the interface between the frame and the panels. The analysis provided good results up to the failure load.

Liauw(1970,73) analysed the frame with infill by using the eight term stress function to satisfy the boundary condition of continuous compatibility between frame and the infill.

Kost *et al.*(1974) described a method for dynamic analysis of frames with shear walls with pre existing gaps at the interface of walls and frames. All parts of the structure are assumed to be linear but the response of the structure is nonlinear due to opening and closing of gaps. Each panel is modelled as four noded elements with 16 generalised displacements.

Rao and Seetharamulu(1983) used elements which included inplane rotation to study the behaviour of staggered shear panels in tall buildings. A good agreement between experimental and analytical results has been reported.

Papia(1988) used boundary element to model the behaviour at the infill interface. A comparison of results with those of the equivalent strut model has been made.

CONCLUSIONS

Based on the literature review the following points emerge

- Setback type and regular asymmetric buildings on flat grounds have been analysed extensively with approximate and rigorous methods of analyses. The buildings on sloping ground have also been studied.
- It has been observed that torsional deformations of systems tend to increase with increasing eccentricity.
- Building Codes suggest, a detailed dynamic analysis is to be carried out for irregular asymmetric buildings. But for regular asymmetric buildings static analysis procedure is recommended by taking the design eccentricity as suggested in various Codes of Practice.
- Simplified dynamic analysis procedures for seismic analysis of regular and irregular asymmetric buildings have been presented and can be used for practical design purposes.
- For inelastic dynamic analysis of buildings r.c. member modelling based on plastic hinge concept has been used. These model take into account the interaction of axial forces and moments in three directions. The effect of shear forces has also been considered.
- Stiffness degrading models based on Takeda's model for 2D r.c. members has been used extensively.
- Various models based on finite element approach for panel elements has been used. It is very important to account for stiffness due to panel elements while analysing frames under seismic loading.
- Case studies of reinforced concrete framed buildings on hill slope during the past earthquakes need to be carried out to throw light on the causes of damages/failures.
- Experimental studies are needed to study the seismic behaviour of buildings on sloping ground so as to validate the theoretical results.
- Detailed soil structure interaction studies are needed for buildings on sloping ground in hill areas.
- Stability of rock slopes needs to be investigated under the action of building loads.

REFERENCES

1. **Allahabadi, R. and G.H. Powell(1988)**, Drain-2Dx user guide, report no. UCB/EERC-88/06, University of California, Berkeley.

2. **Anderson, V.C. and W.H. Townsend(1977)**, Models for r.c. frames with degrading stiffness, Proceedings of the Structural Division, ASCE, Vol.103, No. ST12, pp(2361-2376).

3. **Ayala, A.G., O.Garcia and.A.Escobar(1992)**,Evalution of seismic design criteria for asymmetric buildings, Proc.of 10th WCEE, Vol.X, pp(5693-5698).

4. **Berg, G.V.(1962)**, Earthquake stresses in tall buildings with setbacks, Proc. 2nd symposium on Earthquake Engg., University. of Roorkee, India, pp(267-284).

5. **Blume, J.A. and D.P.Jhaveri(1969)**, Time history response of buildings with unusual configurations, Proc. IVth WCEE, Vol.III, pp(155-170).

6. **Boroschek, R.L. and S.A.Mahin(1992)**, Investigation of coupled lateral-torsional response in multistorey buildings, Proc.of 10th WCEE, Vol.VII, pp(3881-3886).

7. **Bozorgnia, Y. and W.K.Tso(1986)**, Inelastic earthquake response of asymmetric structures, J. of ASCE, Vol. 112, No.2 pp(383-399).

8. **Chandler, A.M. and X.N.Duan(1991)**, Evaluation of factors influencing the inelastic seismic performance of torsionally asymmetric buildings, J. of EESD, Vol.20, pp(87-95).

9 **Chen, P.F.S. and G.H. Powell(1982)**, Generalized plastic hinge concepts for 3 D beam column elements, Report No. UCB/EERC/82-20, University of California, Berkeley.

10. **Cheung, V.W.T and W.K.Tso(1987)**, Lateral load analysis for buildings with setback, .J. of Structural Engg.ASCE, Vol.113, No. 2, pp(209-227).

11. **Chopra, A.K. and R.K.Goel(1991)**, Evaluation of torsional provisions in seismic codes, J. of Structural Engg.ASCE, Vol.117, No. 12, Dec.pp(3762-3782).

12. **Corderoy, H.J.B. and D.P.Thambiratnam(1993)**, Microcomputer analysis of torsionally coupled multistorey buildings for earthquakes, Int. J. of Computers & Structures, Vol.46, No.4, pp(593-602).

13. **Correnza, J.C. and G.L. Hutchinson(1994)**, Effect of transverse load resisting elements on inelastic earthquake response of eccentric plan buildings, J. of EESD, Vol. 23, pp(75-89).

14. **Costa, A.G., C.S. liveira and R.T. Duarte(1988)**, Influence of vertical irregularities on seismic response of buildings, Proc. of 9th WCEE, Vol. V, Tokyo-Japan, pp(491-496).

15. **Cruvellier, M. and B. Stafford Smith(1993)**, A planar model for the static and dynamic analysis of asymmetric building structures, J. of Computers and Structures, Vol. 48, No. 5, pp(951-956)

16. **Earthquake Resistant regulations A World list - 1992** Compiled by International Association for Earthquake Engg.

17. **Fukada, Y., T.Kobayashi, M.Adachi, M.Nagata and Y.Hayashi(1992)**, Torsional vibration of a building with stepped heights, Proceedings of 10th WCEE, Vol.7, pp(3939-3944).

18. **Gillies, A.G. and R. Shephered(1984),** Inelastic dynamic modelling of reinforced concrete frame elements, Proc. of International Conference on Computer aided analysis and design of concrete structures, Yugoslavia, pp(1271-1283).

19. **Goel, R.K. and A.K.Chopra(1992),** Evaluation of seismic code provisions for asymmetric plans, Proc.of 10th WCEE, Vol.X, pp(5735-5740).

20. **Goel, R.K. and A.K.Chopra(1991),** Inelastic seismic response of one storey asymmetric-plan systems, J. of EESD, Vol.20, pp(201).

21. **Hamzeh, S., T.K. Datta and S.M.A. Kazimi(1990),** Inelastic response of asymmetric system to an ensemble of earthquake records, Proc. of Ninth symposium on Earthquake Engg., Vol. 1, pp(8-17).

22. **Hejal, R. and A.K.Chopra(1989),** Lateral torsional coupling in earthquake response of frame buildings, J. of Structural Engg.ASCE, Vol.115, No.4, pp(852-867).

23. **Holmes, M.(1961),** Steel frames with brick work and concrete in filling, Inst. of Civil Engrs(London), 19, pp(473-498).

24. **Humar, J.L. and E.W.Wright(1977),** Earthquake response of steel framed multistorey buildings with setbacks, J. of EESD, Vol.5(1), pp(15-39)

25. **Imbeault, F.A. and N.N. Neilson(1973),** Effect of degrading stiffness on the response of multistory frames subjected to earthquakes, Proceedings of fifth WCEE Rome Italy.

26. **Jain, S.K. and U.K.Mandal(1992),** Dynamics of buildings with V-shaped plan, J. of structural engg.ASCE, Vol.118, pp(1093-1112).

27. **Jain, S.K. and U.K.Mandal(1992),** Seismic response of horizontal setback buildings, Proc. of 10th WCEE, Vol.VII, pp(3853-3858).

28. **Kan, C.L. and A.K.Chopra(1977),** Elastic earthquake analysis of torsionally coupled multistorey buildings, J. of EESD, Vol.V, pp(395-412).

29. **Kan, C.L. and A.K.Chopra(1981),** Torsional coupling and earthquake response of simple elastic and inelastic systems, J. of ASCE, Vol. 107, ST8, pp(1569-1588).

30. **King, G.J.W. and P.C. Pandey(1978),** The analysis of infilled frames using finite elements, Proc. of Instn Civ. Engrs, Part 2, 65, pp(749-760).

31. **Kost, G. and R.B. Barber(1974),** Non linear dynamic analysis of frames with filler panels, J. of Structural Division ASCE, Vol. 100, No. ST4, pp(743-757)..

32. **Kumar, S.(1996),** Seismic Analysis of Stepback and Setback Buildings, Ph.D. Thesis, University of Roorkee, Roorkee, INDIA.

33. **Kumar, S. and D.K. Paul(1994),** Dynamic analysis of Stepback and Setback buildings, Proc. Tenth Symposium on Earthquake Engg., Vol. 1, pp(341-350).

34. **Kumar, S. and D.K. Paul(1994)**, 3-D analysis of irregular buildings with rigid floor diaphragms, Bull. of Indian Society of Earthquake Technology, Vol. 31, No. 3, pp(141-154).

35. **Kunneth, S.K., A.M. Reinhorn and Y.J. Park(1990)**, Analytical modelling of inelastic seismic response of r.c. structures, J. of Structural Engg ASCE, Vol. 116, No. 4, pp(996-1017).

36. **Lai, S., G.T. Will and S. Otani(1984)**, Model for inelastic biaxial bending of concrete structures, J. Structural Engg. Div. ASCE, Vol. 110, No. 11, pp(2563-2584).

37. **Liauw, T.C.(1970)**, Elastic behaviour of infilled frames, ICE, 46, pp(343-349).

38. **Liauw, T.C. and K.H. Kwan(1984)**, Non linear behaviour of non integral infilled frames, Computers and Structures, 18(3), pp(551-560).

39. **Llera, J.C. and A.K. Chopra(1994)**, Accidental torsion in buildings due to stiffness uncertainty, J. of EESD, Vol.23, pp(117-136).

40. **Llera, J.C. and A.K.Chopra(1994)**, A simplified model for analysis and design of asymmetric plan systems, J. of EESD, Vol.24, pp(573-594).

41. **Llera, J.C. and A.K. Chopra(1995)**, A simplified model for analysis and design of asymmetric plan buildings, J. of EESD, Vol. 24, pp(573-594).

42. **Maheri, M.R., A.M. Chandler and R.H. Bassett (1991)**, coupled lateral torsional behaviour of frame structures under earthquake loading, J. of EESD, Vol.20, No.1, pp(61-85).

43. **Mallick, D.V. and R.T. Severn(1967)**, The behaviour of infilled frames under static loading, Inst. Civil Engrs(London), 38, pp(639-656).

44. **May, I.M. and J.H. Naji(1991)**, Non linear analysis of infilled frames under monotonic and cyclic loadings, Computers and Structures, 38(2), pp(149-160).

45. **Nassar, A.A., H.Krawinkler and J.D.Osteraas(1992)**, Seismic design based on strength & ductility demands, Proc. of 10th WCEE, Vol.X, pp(5861-5866).

46. **Nigam, N.C.(1967)**, Inelastic interactions in the dynamic response of structures, Ph.D Thesis, California institute of Technology, Pasadena.

47. **Paul, D.K.(1993)**, Simplified seismic analysis of buildings on hill slopes, Bull of ISET, Vol. 30, No. 4, pp(113-124).

48. **Pachau(1992)**, Seismic response of r.c. framed buildings on hill slopes, M.E. thesis, University of Roorkee, Roorkee.

49. **Papia, M.(1988)**, Analysis of infilled frames using a coupled finite element and boundary element solution scheme, Int. J. Num. Methods in Engg., 26, pp(731-742).

50. **Penzien, J.(1969)**, Earthquake response of irregularly shaped buildings, Proc. 4th WCEE, Vol.2, pp(75-89).

51. **Pekau, O.A. and R.Green(1974)** Inelastic structures with setbacks, Proc. 5th WCEE, Vol.2, Rome, Italy, pp(1744-1747).

52. **Powell, G.H. and P.Fu. Song Chen(1986)**, 3-D beam Column element with generalized plastic hinge, J. of Engg. Mech. Div., ASCE, Vol. 112, No. 7, pp(627-641).

53. **Prasad, R.B.K. and K.S.Jagdish(1989)**, Inelastic torsional response of a single storey framed structures, J. of Engg. Mechanics, ASCE, Vol.115, No.8, pp(1782-1797).

54. **Rao, K.N.V.P. and K. Setharamulu(1983)**, Staggered shear panels in tall buildings, J. Struct. Engg. Div., ASCE, 109(5), pp(1174-1193).

55. **Rutenberg, A., A.Benbenisht and O.A.Pekau(1992)**, Nonlinear seismic behaviour of code designed eccentric systems, Proc.of 10th WCEE, Vol.X, pp(5751-5756).

56. **Rutenberg, A., T. Hsu and W.K. Tso(1978)**, Response spectrum techniques for asymmetric buildings, J. of EESD, Vol. 6, No. 5.

57. **Satake, N. and A.Shibata(1988)**, Dynamic inelastic analysis for torsional behaviour of setback type building, Proceedings of the 9th WCEE, Vol.V, pp(49-54).

58. **Sfakianakis, M.G. and M.N. Fardis(1991)**, Biaxial column element for non linear dynamic analysis of space frame reinforced concrete structure, Proc. of European Conf. on Structural Dynamics, Bochum, FR Germany pp(557-564).

59. **Shahrooz, B.M. and J.P.Moehle(1990)**, Seismic response and design of setback buildings, J. of Structural Engg.ASCE, 116(5), pp(1423-1439).

60. **Smith, S.B.(1966)**, Behaviour of square infilled frames, J. Strct. Div. ASCE, 92, (ST1), pp(381-403).

61. **Sobaih, M., A.Hindi and S.AL-NOURY(1988)**, Nonlinear seismic analysis of setback reinforced conc.frames, Proceedings of 9th WCEE, Vol V, pp(485-490).

62. **Takeda, T., M.A. Sozen and N.Noeby Nielsen(1970)**, Reinforced concrete response to simulated earthquakes, Proceedings of the Structural Division, ASCE, Vol. 96, No. St12, Dec. pp(2557-2573).

63. **Takizawa, H. and H. Aoyama(1976)**, Biaxial effects in modelling earthquake response of r.c. structures, J. of EESD, Vol. 4, pp(523-552).

64. **Terzaghi, K. and R.B. Peck(1967)**, Soil Mechanics in Engineering practice, John Wiley & Sons, New York.

65. **Thanoon, W.A.M.(1993)**, Inelastic dynamic analysis of Concrete frames under Non Nuclear Blast loading, Ph.D. Thesis, University of Roorkee, Roorkee.

66. **Tseng, W.S. and J. Penzien(1975),** Seismic analysis of long multiple span highway bridges, J. of EESD, Vol. 4, pp(3-24).

67. **Tso, W.K. and K.M. Dempsey(1980),** Seismic torsional provisions for dynamic eccentricity, J. of EESD, Vol. 8.

68. **Tso, W.K. and A.W.Sadek(1985),** Inelastic seismic response of simple eccentric structures, J. of EESD, Vol. 13, pp(255-269).

69. **Vulcano, A.(1982),** Comparison between ductility requirements based on different criteria, Proc. of VII symposium on Earthquake Engg., University of Roorkee, pp(209-214).

70. **Wong, C.M. and W.K.Tso(1994),** Inelastic seismic response of torsionally unbalanced systems designed using elastic dynamic analysis, J. of EESD, Vol. 23, pp(777-798).

71. **Wood, S.L.(1992),** Seismic response of R/C frames with irregular profiles, J. of Structural Engg.ASCE, Vol.118, No.2, pp(545-566).

72. **Zhou, Y. and Y.Minoru(1992),** Pulse response analysis of asymmetric structures, Proc. of 10th WCEE, Vol.VII, pp(3957-3960).