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Evaluation of seismic design criteria for asymmetric buildings

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ABSTRACT: The seismic performance of nonlinear asymmetric building structures with resisting elements in one and two orthogonal directions and the suitability of different design recommendations are evaluated. The building models are shear structures designed in accordance with a current code and with variations of it. Seismic performance is measured by the ratio of maximum ductility demand for asymmetric structures to the maximum ductility demand for the corresponding symmetric ones. Other performance indexes are also evaluated. The effect of the most relevant structural parameters and the uncertainties on the values of stiffnesses, strengths and on the location of the centre of mass (CM) is investigated. Results from the deterministic and probabilistic models used in this investigation show that the torsional response of building structures is significantly affected by the in-plan distribution of the total strength and that, for the investigated structural parameters, the currently used design coefficients for structures under torsion are not as effective as those of alternative designs.

INTRODUCTION

A significant proportion of the structural damage to buildings in Mexico City during the 1985 Michoacan earthquake has been attributed to ill-torsional behaviour. This was due to asymmetric distributions of stiffnesses, masses and/or strengths which induced eccentricities between the earthquake loading and the resisting force, producing ductility demands large enough to cause severe damage and even collapse of the structures.

Analysis of the damage statistics, Rosenblueth and Meli, 1986, has shown that many of these buildings were actually designed in accordance with a modern building code, where design eccentricities were explicitly considered. The rationale of this, as of many other codes, was to avoid the maximum ductility demand of the asymmetric structure exceeding that of the corresponding symmetric one.

The definition of the parameters involved in the design eccentricities was based on results from investigations of modal espectral analyses of linear models of buildings, Elourdy and Rosenblueth, 1968. This consideration contradicts the fact that the codes explicitly accept different levels of damage for high seismic intensities, which can only be reached if the structure incursions into the inelastic range. The effect of nonlinearity is incorporated, at a later stage, by reducing the seismic forces in accordance with an accepted design ductility demand.

Recent investigations have shown that the above assumptions are not valid, as inelastic responses are characterized not only by reduced forces but also by significant changes in behaviour when compared with the corresponding linear elastic ones. Furthermore, it has been shown that increased eccentricities, not accounted for in the design recommendations, are produced, even in nominally symmetric structures, by nonlinear effects present when some of the resisting elements incursion into the inelastic range and/or by uncertainties in the distribution of mass and in the nominal values of stiffnesses and strengths of the different resisting elements.

The problem of the inelastic behaviour of asymmetric building structures with deterministic and uncertain structural parameters has been the topic of research for the past few years; e.g. Ayala and Barrón, 1990; and Escobar & Ayala, 1991. Results show large discrepancies between the behaviour of nonlinear and linear structures. This paper summarizes the most relevant results of this investigations. It concentrates on the study of the influence of the most relevant structural parameters and the randomness of stiffnesses, strengths and location of the centre of mass upon structural performance emphasizing behavioural aspects and the rationale behind the conclusions and not on the methodology used.

ANALYSIS OF PREVIOUS WORK

The nonlinear behaviour of models of asymmetric structures has been frequently presented in the literature, e.g. Irvine and Kountoris 1980, Tso and Sadek 1985, Bozorgnia and Tso 1986, and Bruneau and Mahin 1987 among others. A common drawback found in most of most of these investigations is that the models studied were not designed in accordance with a current building code, situation that explains the frequent contradiction between their conclusions. For the same reason, the practical applicability of their results has been questioned, Ayala and Barrón 1990.

DESIGN ECCENTRICITY CONCEPT

In a building structure where the centres of torsion (CS) of the interstoreys do not coincide with the corresponding centres of mass, the translational vibration of the building couples with its torsional vibrations. This effect is implicitly considered in a three dimensional dynamic analysis. Seismic codes, however, allow only for the translation analysis of the building in the so-called static method with torsional effects introduced by considering a torsional moment in each interstorey given as the product of the interstorey shear force multiplied by the design eccentricity.

The design eccentricities for a given interstorey specified by the codes are given by

$$\begin{aligned} e_{d1} &= \alpha_1 e_s + \beta b \\ e_{d2} &= \alpha_2 e_s - \beta b \end{aligned} \qquad (1)$$

where e_s is the static eccentricity, α_1 and α_2 are the corresponding dynamic amplification and deamplification factors used to introduce the differences between the static and dynamic methods of analysis, b is the dimension of the floor in the direction normal to the excitation and β is the accidental eccentricity coefficient which takes into account the rotational component of the excitation and the uncertainties in the distribution of mass and stiffnesses.

Examples of codes based on the format given by eq 1 are shown in Table 1.

The determination of the torsion centres of the interstoreys of a building, function of the applied loading, requires a three dimensional analysis, e.g. Damy and Alcocer, 1987. Recently, however, the existence of a torsion centre and the validity of the design format involved in eq 1 have been questioned as the definition of the design eccentricities given by eq 1 assume that the response of the building occurs in the elastic range. This assumption is not longer valid if inelastic incursions are accepted, i.e. a ductility reduction factor is considered for the design forces.

To understand the effect of the value of the coefficients upon structural performance, a parametric study with simplified building models, designed in accordance with the current Mexico City Code and two variations of it was carried out. The selected design coefficients are given in Table 2.

Table 1. Design eccentricity coefficients for different building codes; DDF 1987, CEB 1987, NBCC 1977 and ATC 1978.

CODE	α_1	α_2	β
MEXICO	1.5	1.0	0.10
C.E.B.	0.5	0.0	0.05
CANADA	1.5	0.5	0.05
A.T.C.	1.0	0.0	0.05

Table 2. Considered design eccentricity coefficients

DESIGN	α_1	α_2	β
1	1.50	1.00	0.1
2	1.25	0.75	0.1
3	1.00	0.50	0.1

The purpose of this strategy was to evaluate the applicability and the limitations of the current design criteria and, if applicable, to suggest alternatives.

For a given interstorey, the application of the design eccentricities given in eq 1 produces a torsional overstrength which for the design criteria proposed above is constant, e.g. for a single storey building with two resisting elements, the torsional overstrength is equal to $(0.5e_s/b + .2)*F$, where F is the design shear force. For this case, the first part of the overstrength accounts for the dynamical amplification effects and the second, for the accidental eccentricity effects.

To illustrate the implications of the above design criteria for the single storey building, in the first design criterium all the dynamical amplification overstrength is assigned to the element furthest from the centre of torsion, in the second, 50% of this is assigned to each element and, in the third all of the overstrength is assigned to the element nearest to the centre of torsion.

STRUCTURAL MODELS

In this investigation, building structures were idealized as shear single storey structures with two to four resisting elements in one and two directions and attached to an infinite rigid rectangular floor, fig 1.

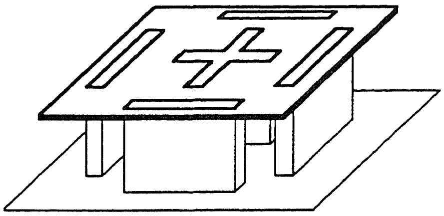


Fig. 1. Structural model

The resisting elements followed a bilinear, non-degrading, hysteretic behaviour with a second branch with a slope of 1% of the value of the slope of the initial branch. The behaviour characteristics of the models allowed for a simple computational model as that shown in fig 2.

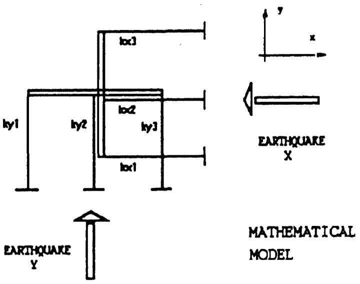


Fig. 2. Computational model

For the models presented in this paper the base shear coefficient considered a reduction corresponding to a prescribed ductility demand.

Contrary to the behaviour of asymmetric structures in the elastic range, structures incursioning into the inelastic range behave differently if the eccentricity is produced by movements of the centre of mass or by movements of the centre of torsion. In this paper both situations are considered. For the stiffness eccentric models, the centre of mass was located at the centre of the floor, and for the mass eccentric buildings, the centre of torsion was located at the centre of the floor, fig 3. Mixed situations where neither of them was located at the centre of the floor were also considered but not presented in this paper.

The seismic excitation of the models was the E-W and N-S components of the 1985 Michoacan earthquake recorded at the SCT site. Due to the computational effort involved in the simulations, the original duration of the record was reduced using a criterium based on Arias intensity, Ayala and Barrón, 1990.

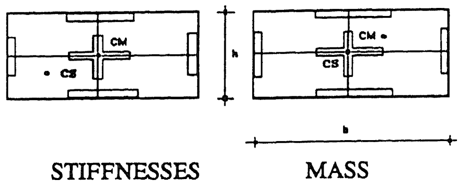


Fig. 3. Types of eccentricity.

The stiffnesses and mass of the models were assigned to produce prescribed translational fundamental periods. Different in-plan distibutions of stiffnesses were analyzed for structures with more than two resisting elements in each direction. In this paper only one distribution is presented, with 75% of the total stiffness in each direction assigned to the outermost elements and the remaining 25% to the intermediate element(s).

The strengths of each resisting element were defined using the lateral force method with design eccentricities given in eq 1. Their value was always affected by an overstrength factor to take into account the fact that real strengths are systematically larger than the nominal strengths given by the design formulae Meli and Avila, 1989. The in-plan distribution of strengths was defined by the parameters X_R and Y_R which measure the distances from the centre of the floor to the corresponding components of the total resisting force.

The deterministic analyses were carried out for all possible locations of the centre of mass considered in the accidental eccentricity, i.e. three locations for one direction and nine for two.

For the probabilistic models with random structural parameters, the values of stiffnesses and strengths for the resisting elements were considered lognormal distributed with mean values corresponding to the deterministic models and an assumed coefficient of variation of 20%, Sues, Wen and Ang, 1983. Coefficients of correlation of 0. and 0.5 were considered between stiffnesses and strengths. The location of the centre of mass was a random variable with uniform distribution between the limits given in the definition of the design eccentricities. The probabilistic analyses were carried out using MonteCarlo simulations with selected sampling given by multipoint estimations, Rosenblueth, 1983.

CONSIDERED VARIABLES

To cover cases of practical interest within the limitations of the selected models, the following variables were considered:

1. Structural eccentricity (mass and stiffness)

$0. \leq e_s \leq 0.3b$

2. Translational periods

   T=0.5, 1.0 and 1.5 sec

3. Uncoupled frequency ratio Ω, function of the aspect ratio of the floor dimensions, fig 1.

   b/h=0.5, 1.0, and 2.0

4. In-plan strength distribution.

   $-0.3b \leq X_R \leq 0.3b$, and

   $-0.3b \leq Y_R \leq 0.3b$

5. Overstrength of the resisting elements.

   R_r/R_n=1.0, 1.25 and 1.5

6. Design criterium, (Table 2).
7. Number and orientation of resisting elements.
8. Design ductility

   Q=2, 3 and 4

9. Uncertainty in the location of the centre of mass.

Additionally for the probabilistic models:

10. Uncertainty in stiffnesses
11. Uncertainty in strengths
12. Correlation between stiffnesses and strengths.

STRUCTURAL PERFORMANCE

The purpose of considering design eccentricities in the seismic design of buildings is to obtain a structural performance for the asymmetric structures which does not exceed that of the corresponding symmetric one. Most design codes implicitly consider as a measurement or index of structural performance the interstorey ductility demand. In this paper, results for this index are presented, nevertheless, others were also evaluated, i.e. number of inelastic incursions, number of yield deformation reversals, dissipated hysteretic energy and accumulated ductility, and are presented elsewhere, Escobar and Ayala, 1991.

ANALYSIS OF RESULTS

The computational effort involved in the calculation of the results of this parametric study was enormous. It was impossible to present in a single paper all the results derived from this investigation. Thus, this only presents some representative results which show general tendencies in the seismic performance of the studied models.

Figures 4 and 5 show the distribution of the performance index versus the parameters X_R and e_s for stiffness and mass eccentric models respectively with elements in one direction. The illustrated behaviour was characteristic of most of the studied models and basically shows that better indexes are attained if the distribution of strengths moves the location of the resisting force toward the location of the centre of torsion. The particular design for the current Mexico City Code is shown by the broken lines.

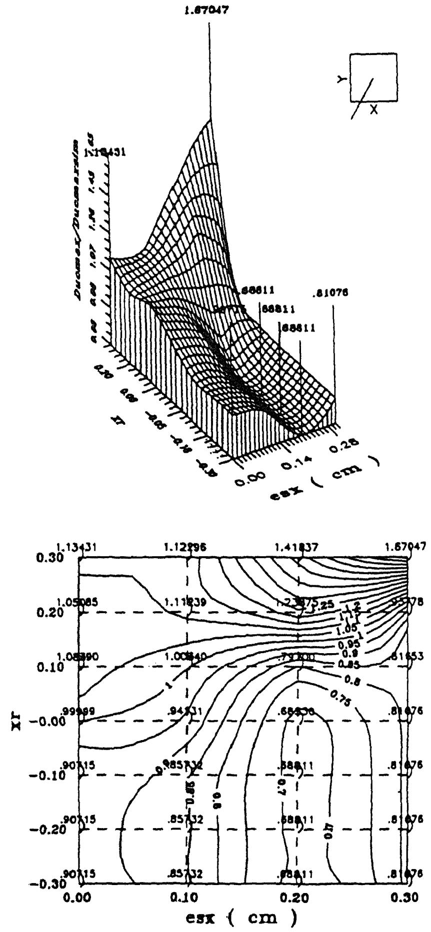
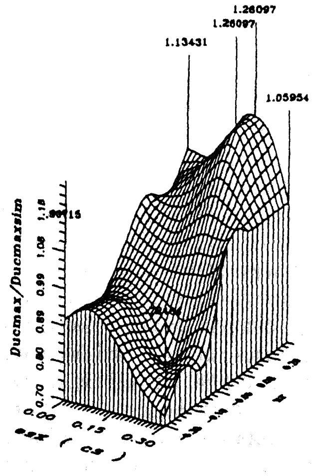


Fig. 4. Envelope of maximum ductility ratio versus e_s and X_R. Design 1, $T_x=T_y$=1.5 sec, Q=4, h/b=1.0.



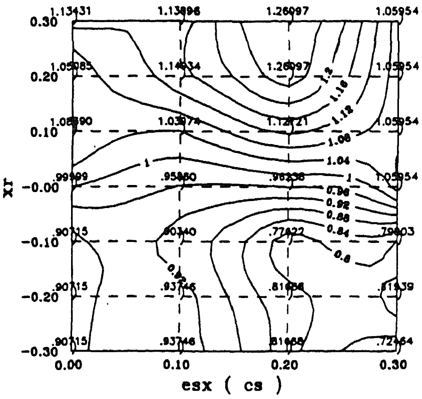


Fig. 5. Envelope of maximum ductility ratio versus e_s and X_R. Design 1, $T_x=T_y=1.5$ sec, Q=4, h/b=1.

Quantitatively, the mass or stiffness eccentricity, models exhibit different performance indexes. However, their distibutions were similar, with higher indexes associated with mass eccentric structures.

The ratio of total to minimum design strength versus X_R and e_s parameters is shown in fig 6. Any change in the distribution of strengths implies an increment in the total and, consequently, an increase in the cost of the structure.

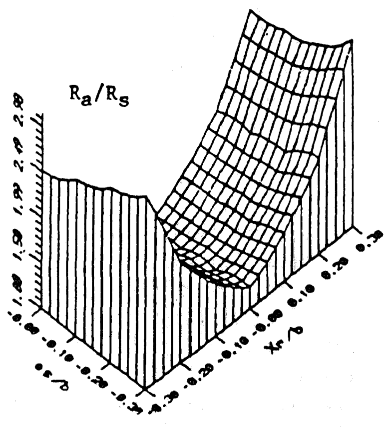


Fig. 6. Total strength ratio versus e_s and X_R. Design 1, T=1.0 sec, Q=4, h/b=1.0, one direction.

The alternative designs 2 and 3, produced distributions of strengths with locations of the minimum design resisting force closer to the centre of torsion and in general exhibited better performance indexes than design 1. Fig 7 show the performance index distribution versus X_R and Y_R for a model with resisting elements in both directions. It may be observed that for the two alternative designs, smaller indexes are attained even without increasing the total strength. On numerous occasions, the minimum performance indexes were obtained for design 3.

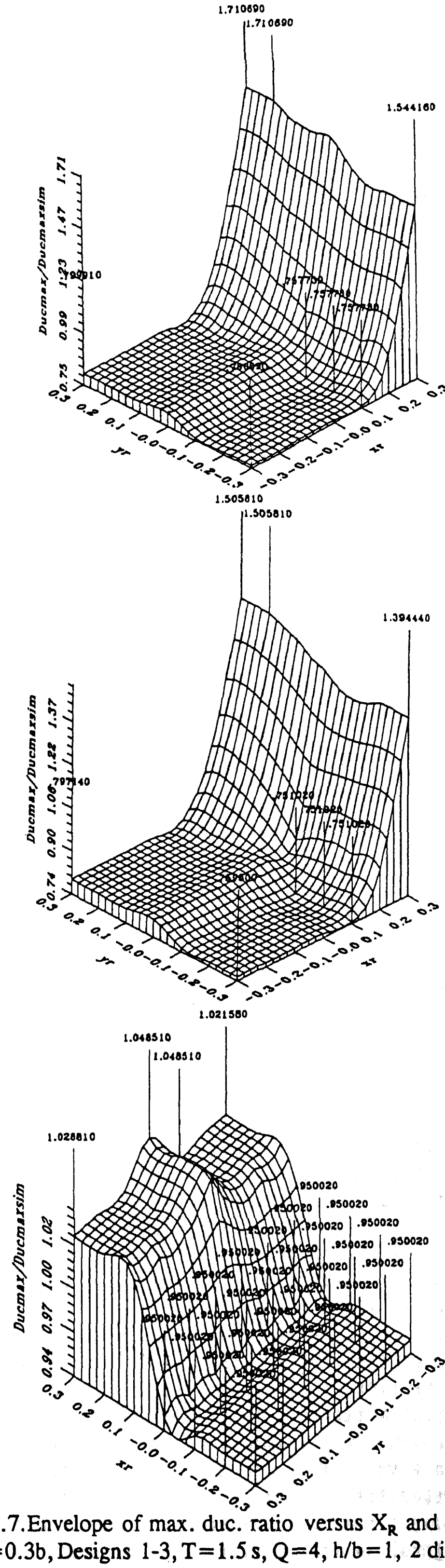


Fig.7.Envelope of max. duc. ratio versus X_R and Y_R. e_s=0.3b, Designs 1-3, T=1.5 s, Q=4, h/b=1, 2 direc.

For all designs, the characteristic behaviour was preserved, i.e. better performance indexes were attained when the distribution of strengths leads to a location of the resisting force nearest to the CS.

For probabilistic models, fig 8 shows the probability that the performance index exceeds that corresponding to the deterministic symmetric structure versus the strength distribution for a model with three resisting elements in one direction. As for the deterministic models, lower probabilities of exceedence are obtained for locations of the total resisting force close to the CS. This figure also shows the high probabilities of exceedence associated with nominally symmetric structures.

The effect of the structural parameters not discussed in this paper may be found elsewhere, e. g. Ayala y Barrón 1990 and Escobar and Ayala 1991.

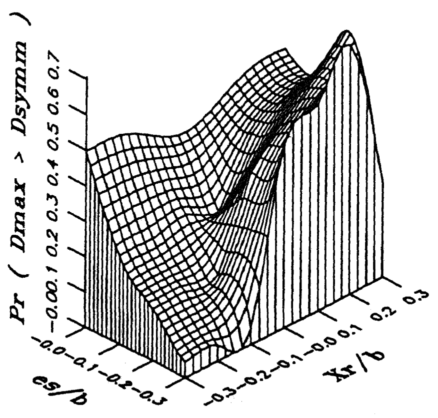


Fig. 8. Probability of exceedence of the maximum ductility demand versus e_s and X_R. Design 1, T=1.0 sec, Q=4, h/b=1, one direction.

CONCLUSIONS

From the analysis of the above results, it is concluded that the coefficients involved in the design eccentricities recommended in the current code for Mexico City, lead to designs for the elements which follow the distribution of mass, and that this distribution may lead to values of the performance indexes in excess of those considered adequate. To keep these values within acceptable levels, i.e. less or equal than one and low probability of exceedence, it is required either to further increase the total strength of the structure by moving the position of the resisting force to a location between the CM and the CS or, in a better way, to modify the design coefficients in such a way that the torsional overstrength is kept constant but the interstorey resisting force is moved toward a position between the CM and the CS. A pragmatic design recommendation would be to keep the strengths of the resisting elements as proportional as possible to the corresponding stiffnesses.

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