INELASTIC DYNAMIC RESPONSE OF CONCRETE FRAMES
DESIGNED IN ACCORDANCE WITH THE MEXICAN SEISMIC CODE

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SUMMARY

This paper presents an investigation on the behavior of concrete frames designed in accordance with the Mexican Seismic Code. The behavior of the code-designed frames was studied using step-by-step nonlinear dynamic analysis. Four different properly scaled earthquakes recorded in Mexico City were used as exitation. It is shown that the behavior of the frames may be inadequate due to excessive column inelastic demands. A rational method to protect the columns which directly considers design ductility, axial load level and confinement is proposed.

INTRODUCTION

In the current Mexican Seismic Code (Ref. 1), structures made up of reinforced concrete frames may be designed using a ductility factor (Q) of either 4 or 6 depending essencially on detailing requirements. Although it may be stated that the requirements and limitations when designing with Q=6 are rather stringent, the same does not hold true for the much more common case when Q=4. The designs using a ductility factor of 4 comprise the large majority of cases and in these, little attention towards ductility oriented detailing is required. The primary purpose of the research from which this paper is extracted was to find out if the inelastic demands that may be expected for concrete frames designed in accordance with the Mexican Code are compatible with the corresponding detailing practice required by the same document. The investigation also focused on the effect of allowable story drift on inelastic demands. Due to space limitations many of the results of the overall research are not presented herein. A complete report may be found as Reference 2.

DESIGN AND NONLINEAR ANALYSIS OF THE SELECTED FRAMES

Ground Motions Utilized

The structures selected were analized with the following ground motions: a) GM1: 14-March 79/SAHOP/COMP N90W; b) GM2: 24-October 80/ Texcoco Centro Lago/Comp N90W; c)GM3: 2-August 68/Foundation Atizapan Building/Comp E-W and d) GM4: 6-July 64/Patio Hidalgo Building/Comp E-W.

All four ground motions were recorded on soft soil in Mexico Federal District. The maximum accelerations were scaled to 0.06g to match the spectral acceleration for zero period in the code. The elastic and inelastic spectra for all the scaled records are shown in figure 1. All spectral ordinates correspond to 5% critical damping.

Design of the Frames

The frames selected for the investigation are shown in figure 2. The

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designs were done in accordance with the principles of strength design. The load combinations used in proportioning all frame members were: 1.4 (DL + LL) and 1.1 (DL + LL ± E).

The second order effects were evaluated and included in the design through the use of the "p-Δ method". The analysis to obtain internal forces was carried out using the program TABS, all translational modes were included in the modal combinations. The sections for both frames were obtained by trial and error to make the maximum story drift in Frame I be very close to 0.016h (maximum permitted by the code if there are no nonstructural elements which may suffer damage from deformations) and 0.008h in Frame II which is the usual limit.

Mathematical Models for the Nonlinear Dynamic Analyses

Once the frames were designed a set of nonlinear mathematical models were defined for the purpose of the step-by-step nonlinear dynamic analyses. The program utilized for the step-by-step analyses was DRAIN-2D. The beams were modeled with a TAKEDA hysteretic behavior while the columns were assumed elasto-plastic with a yield criterion based on idealized tri-linear interaction diagrams. The second order effects were included through the use of the first-order geometric stiffness of the columns. Rayleigh type damping was included with 5% of critical in the first two modes of the original elastic structures. An integration time step of 0.02 secs. was found to me more than adequate for the structures and frecuency content of the ground motions utilized.

Hinge Rotation Capacity

It is apparent that without reasonable values of the hinge rotation capacities, the demands, no matter how "precisely" calculated loose much of their significance. The hinge rotation capacity was defined, for the purposes of this investigation, as the plastic rotation which monotonically may take place with a strength degradation of no more than 15%. The equivalent length of plastic hinge used was one half of the effective depth. In order to account for all important parameters such as axial load level, confinement, steel strain hardening, spalling of shell concrete at large strains etc., a computer program titled COSECOR (behavior of reinforced concrete sections) was developed. In the case of columns the mean value of the axial load during the earthquake was used in the evaluation of the hinge rotation capacity (i.e., axial load from dead plus live loading). The details of the procedure used to calculate hinge rotation capacity may be found in Reference 2.

RESULTS FROM THE NONLINEAR ANALYSIS

Beam Behavior

Figure 3 shows some selected representative results obtained for the beams of Frame I (the results for Frame II were similar but with much less inelastic action due to the fact that minimum steel governed the design of a large portion of the Frame). As may be seen in figure 3 only ground motion 3 at level 2 induced a plastic hinge rotation which slightly exceeded the theoretically calculated capacity, considering the nature of the problem this is of no real significance. In most levels the capacity far exceeds the demand. (The positive moment hinge capacities are not plotted because they are much larger than the negatives and therefore not critical by inspection.)

Column Behavior

The maximum plastic hinge rotations in the columns of Frame I, as derived from a model based on "design strength" (nominal strength times ϕ factors) are shown in Figure 5. As may be seen from the figure, the computed inelastic demands at some levels far exceed the corresponding capacities. At the third floor, the ratio of demand over capacity is greater than three.

An investigation of the results shown ilustrates the formation of a "story column mechanism" at the fourth level which clearly does not have enough rotation capacity to survive the ground motions (with the exception perhaps of ground motion 2). It is of interest to note that even though the degree of inelastic action induced by the various ground motions differed, the pattern of hinging is essentially the same for all four of them. The tendency for inelastic action to concentrate at the 4th floor is totally undetectable from a study of the linear modal analysis used in the design.

It is apparent that the overall behavior of the columns in Frame I is deficient. The results for Frame II are not shown since just about all the columns in this structure were governed by minimum steel (which resulted in such a large overstrength that only minimal inelastic action was detected).

Allowable Story Drift

Figures 4 and 6 show the effects of allowable or "design story drift" on the plastic hinge rotations experienced by the beams and the columns respectively. As may be seen from the figures, in both cases the inelastic demands are smaller for the more rigid design i.e., Δ=0.008h; the only exception occurs at the foundation level in the case of the columns. The clear tendency to obtain smaller plastic hinge rotations in the stiffer designs comes from the very basic definition of the term ductility. If one designs a structure with a ductility factor of say 4 and an allowable story drift of 0.016h, the implication is that 3/4 of the total story sway occurs after inelastic action has ensued, that is 0.012h. If the same ductility factor of 4 is used but the story drift is limited to 0.008h (stiffer design) then the inelastic portion of the total sway is only 0.006h. The local inelastic demands, which are largely dependent on inelastic story sway, will be significantly less in the latter case, even though in both designs a ductility factor of 4 is utilized. It follows then that ductility factor alone is not an absolute measure of the inelasticity implied in a given design, the initial stiffness of the structure plays also an important role (an extensive treatment of the effect of initial stiffness on inelastic demands may be found in Ref. 3). At present the Mexican Seismic Code limits the use of the higher allowable story drift of 0.016h only on the basis of nonstructural elements being prone to suffer heavy damage from structural distortions with no attention given to the much larger inelastic demands implied; this criterion seems to need some reviewing.

IMPROVING THE BEHAVIOR: PROPOSED CHANGES IN COLUMN DESIGN

The observed behavior of the columns in Frame I (Fig. 5) is totally contrary to the well known Strong column-Weak beam philosophy of seismic resistant concrete frame design. Although the Mexican Seismic Code in its commentary refers to a Strong column-Weak beam desirable behavior it does not, at present, contain any clauses designed to enforce it. Some other codes such as Appendix A of ACI and ATC-3 require that at beam column joints the sum of the flexural strength of the columns exceed that of the beams in the plane of the frame considered; this requirement is obviously in the right direction but has been shown not to be sufficient to prevent extensive column hinging (Ref. 4).

Columns must be made significantly stronger than beams at a common joint in order to be protected from probable extensive yielding. The additional flexural strength needed is a function of how much the column inflexion points drift from their "elastic positions" thus changing the way the "beam moment input" is divided between the columns at the joint. The column inflexion points depart from their otherwise "elastic positions" during the response as a consequence of the drastic change in relative rigidities induced by the beams inelastic behavior. Since the severity of inelastic action in the beams is dependent on the ductility factor Q it follows that the additional flexural strength needed in the columns is also a function of Q (notice that in an elastic design, that is Q=1, no additional column strength beyond that dictated by the structural analysis is required to prevent column hinging). The quantitative relationship between additional strength and ductility factor Q, has been studied extensively elsewhere (Refs. 2 and 3).

On the basis of the previous discussion it is suggested that the load combination to obtain the column moment for designing be changed from:

$1.1\ (M_{grav.} \pm M_{earth.})$ to $1.1\ (M_{grav.} \pm \frac{\omega}{\lambda} M_{earth.})$.

The axial load in the column is not significantly affected by the movement of the inflexion points and may therefore remain unchanged as $1.1\ (P_{grav.} \pm P_{earth.})$.

The value of ω proposed is a function of the ductility used in the design and also of the amount of axial load acting on the column. This is because it is well known that the rotation capacity of critical regions decreases rapidly with increasing axial load and therefore heavily loaded columns must be protected more than lightly loaded ones.

The specific values of ω recommended in Reference 2 are:

Q=1	$\omega = 1.0$		
Q=2	$\omega = 1.0$ for $\alpha \leq 0.1$	$\omega = 0.9 + \alpha$	for $\alpha > 0.1$
Q=4 & 6	$\omega = 1 + \frac{Q}{20}$ for $\alpha \leq 0.1$	$\omega = 0.9 + \alpha + \frac{Q}{20}$	for $\alpha > 0.1$

Where $\alpha = \frac{P_u}{f_c A_g}$ and ω need not be larger than the value obtained when $\alpha = .5$

The value of ω for axial loads equal to or greater than $0.5 f_c A_g$ are such that a totally elastic response of the column is assured (the ductility for such high axial loads is essentially nil).

The parameter λ which appears in the denominator of the proposed design equation is intended to account for the benefitial effect of special confinement. When special confinement is utilized in the potential plastic hinge zone of a column the design moment should be somewhat smaller than for a non-confined case because the amount of inelastic action that may be withstood without significant strength degradation is larger in the confined colum. On the basis of limited studies using the program COSECOR (Ref. 2) it is tentatively suggested that λ be taken as follows:

For standard unconfined designs $\lambda = 1$

For confined columns (as specified for example in Appendix A of ACI318-77)

For $\alpha \leq 0.1$ $\lambda = 1$

For $\alpha > 0.1$ $\lambda = 1 + \frac{2}{3} (\alpha - 0.1) \leq 1.2$

For very low axial loads i.e., $P_u \leq 0.1 f_c A_g$, the value of λ for the confined case is kept as one because in this range the ductility is ample with the standard detailing.

The results obtained for Frame I (Δ=0.016h) with the columns redesigned according to the proposed procedure indicated an acceptable behavior with plastic hinge rotations drastically reduced. The concentration of column yielding at the third floor level dissapeared and so did the formation of a story-column mechanism. The behavior of the beams in the redesigned frame showed larger plastic hinge rotations in levels 3 through 6 (larger than those in Figure 3). The increased participation of the beams in these levels is in this case highly desirable since it reflects the desired Strong column-Weak beam behavior; the demands remained, of course, well within the rotation capacities.

ACKNOWLEDGMENT

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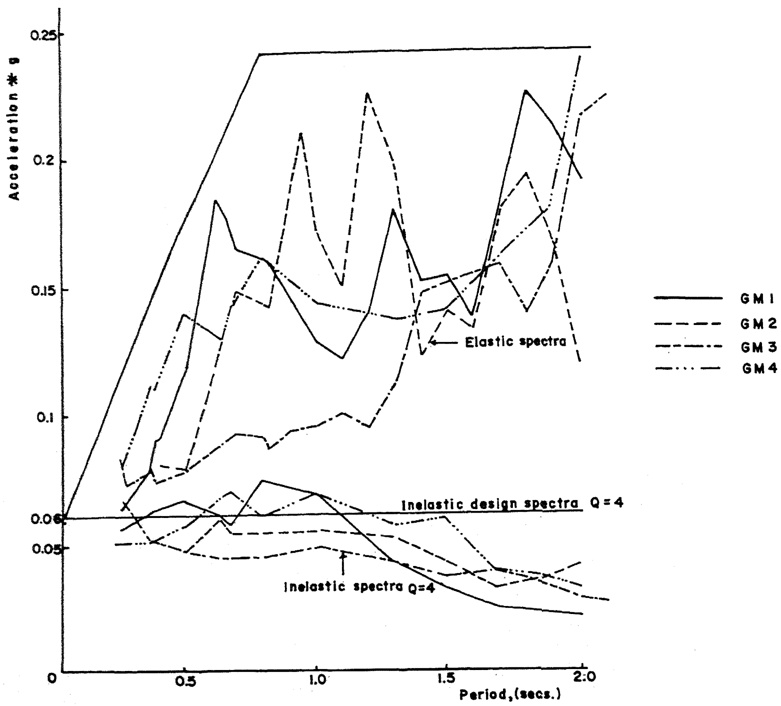

FIGURE 1. Elastic and inelastic spectra for the ground motions utilized.

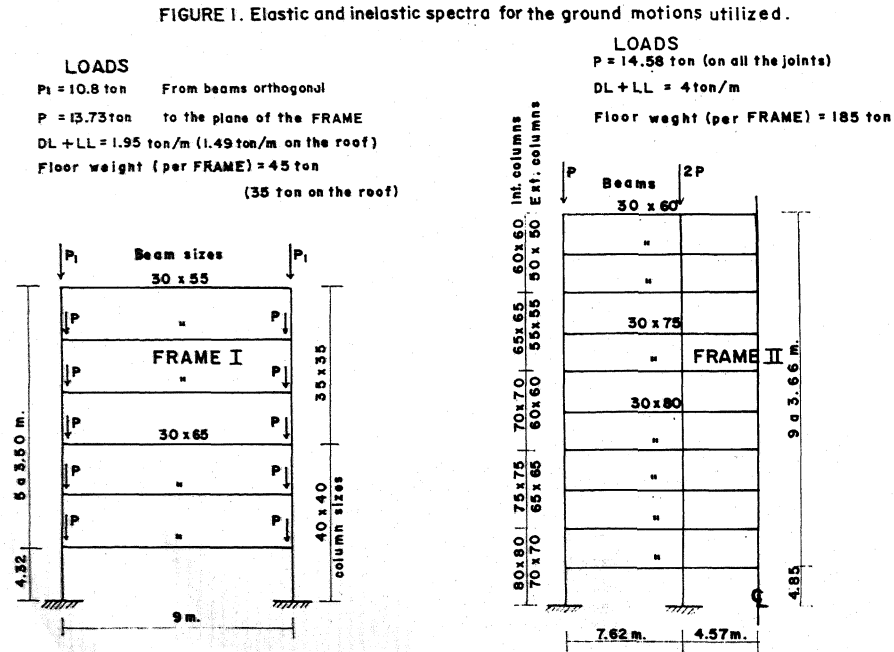

FIGURE 2. Frame elevations and design data.

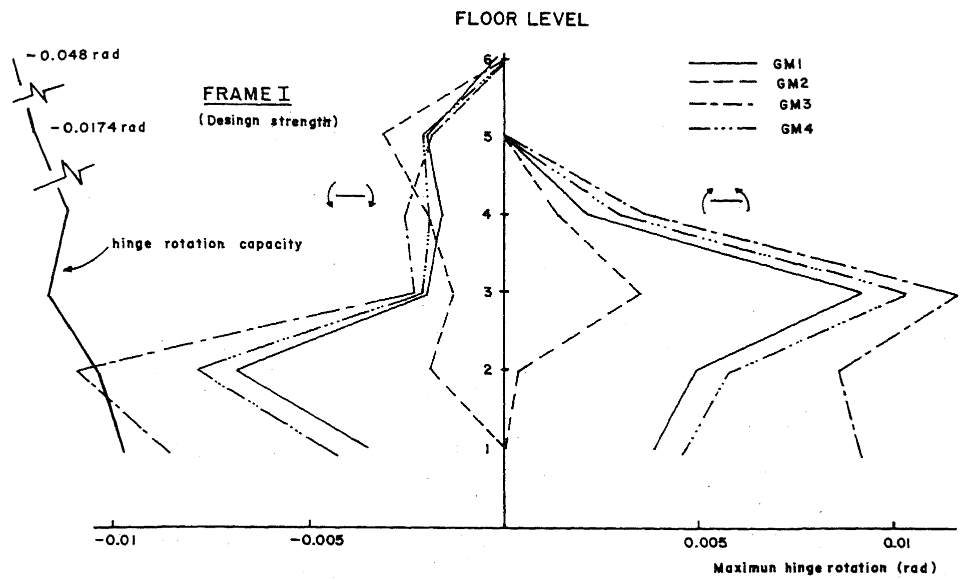

FIGURE 3. Plastic hinge rotations in the beams: $\frac{\Delta}{h} = 0.016$

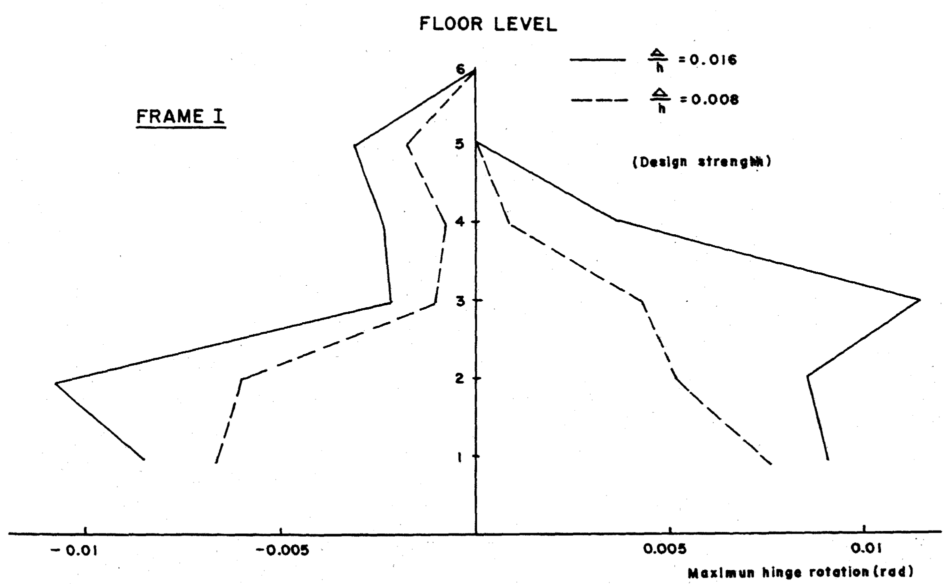

FIGURE 4. Envelopes (maxima of the 4 ground motions) of plastic hinge rotations in the beams for designs with allowable story drifts of 0.008 h and 0.016 h.

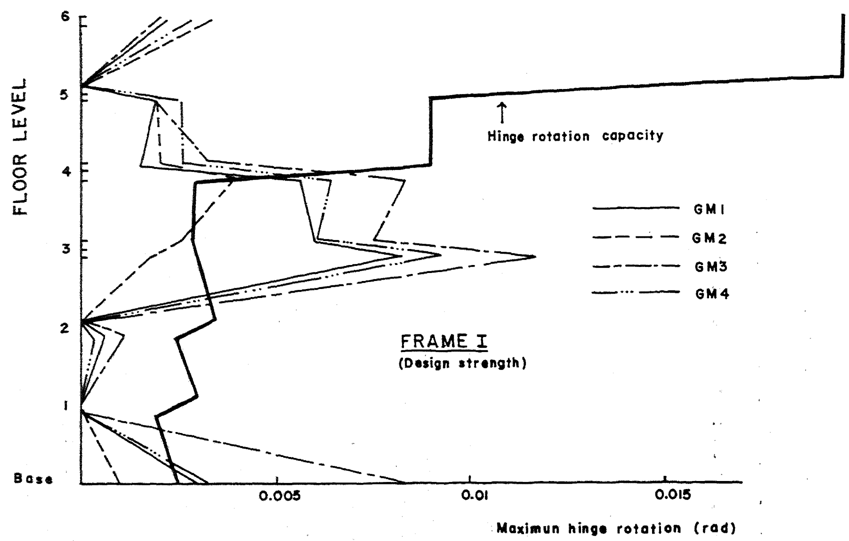

FIGURE 5. Plastic hinge rotations in the columns : $\frac{\Delta}{h} = 0.016$

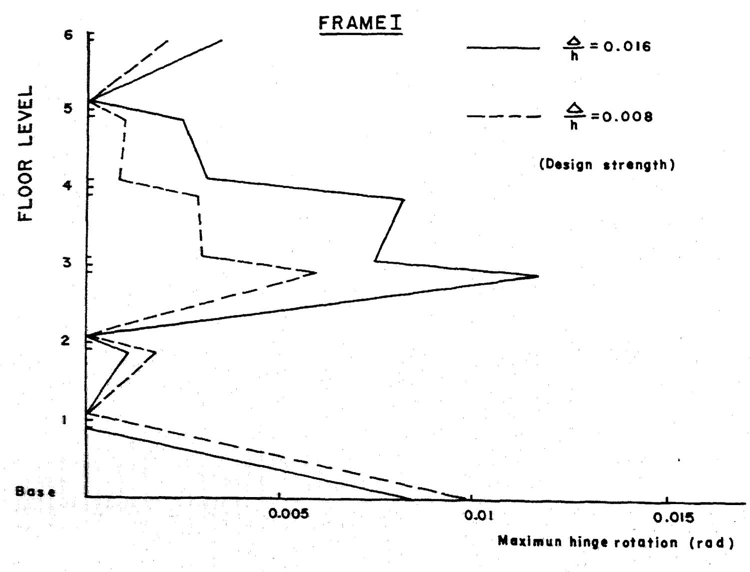

FIGURE 6. Envelopes (maxima of 4 ground motions) of plastic hinge rotations in the columns for designs with allowable story drift of 0.008 h and 0.016 h.