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EVALUATION OF THE INELASTIC SEISMIC BEHAVIOUR OF R/C BUILDINGS DESIGNED BY THE CEB MODEL CODE

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SUMMARY

The seismic behaviour of a 9-storey reinforced concrete building, with a dual (frame+wall) lateral load resisting system, designed to the 1985 CEB Model Code for Seismic Design, is evaluated with the aid of inelastic time-history analysis, combined with comparisons of required and available ductility in critical members. It is concluded that the behaviour of the structure is satisfactory and meets the specified design criteria. Some areas where the Code provisions appear to be over-conservative are also pointed out.

INTRODUCTION

Current design practice recognises that, during a strong earthquake, a structure will enter the inelastic range, to a degree depending on the magnitude of the seismic excitation. Design forces are typically derived by reducing the appropriate elastic response spectra by a factor (usually called "behaviour factor") depending on the ability of the structure to dissipate the input seismic energy through inelastic deformation mechanisms. In this perspective the investigation of the inelastic seismic response of structures, such as reinforced concrete (R/C) buildings, assumes a particular importance.

The present study concerns a typical R/C building with a dual lateral load resisting system, designed according to the provisions of the 1985 Model Code for Seismic Design (MC/SD)(Ref.1). This Code has been prepared by an international group of specialists within the framework of the Euro-international Committee of Concrete (C.E.B.) and reflects the current trends in the earthquake-resistant design of concrete structures.

DESIGN OF THE STRUCTURE ANALYSED

The structure under consideration is a symmetric nine-storey R/C building, with a lateral load resisting system, in the direction considered, consisting of a central wall connected to the end columns through beams with a span of 6.0 m. Because of the symmetric structural layout, only the plane structure shown in Fig. 1 has to be analysed.

The building is assumed to be constructed in an area of moderate seismicity, for which an effective peak ground acceleration of 0.15g is specified. Among the three design ductility levels suggested by the CEB MC/SD, the intermediate one (Ductility Level II) was chosen. The behaviour factor specified by the Code for dual systems consisting of frames and isolated walls is K=2.1. Assuming firm ground conditions, the final base shear coefficient is found to be C_d=0.12, while the corresponding effective masses include 100% of the permanent loads plus 20% of the variable loads (as specified by the MC/SD for appartment buildings).

The capacity design procedures concerning design shear forces and relative strength of beams and columns at joints were applied, as specified by the MC/SD for D.L.II structures. The wall was designed for a linear envelope of the moments calculated from the lateral load analysis, vertically displaced by a distance equal to the horizontal length of the wall (l_w). In addition a dynamic magnification factor

$$\omega = 0.6T + 0.85 \nless 1.3$$

was applied to the column moments and shears above the ground level.

METHODOLOGY OF ANALYSIS

The beams and columns of the structure shown in Fig.1 were idealised as two-node line elements and inelastic behaviour was assumed to take place in concentrated plastic hinges at the element ends only ("point hinge modelling"). The nonlinear moment(M)-rotation(Θ) relation for the point hinges of beam and wall elements was specified to follow the bilinear version of the Takeda model, as incorporated in the DRAIN-2D program by Powell et al.(Ref.2), while for column hinges a simple bilinear model combined with an axial load-moment interaction diagram was used, in order to take into account the effect of fluctuations in axial loading on the member strength. The wall was modelled as a deep beam, using rigid zones to connect the centerline of the wall to the adjacent beams.

The structure was subjected to a number of base acceleration time-histories and the Newmark β=1/4 method was used to solve the dynamic equilibrium matrix equations. The computer program used in the present analysis was DRAIN-2D/85(Ref.3), an extended version of the well-known DRAIN-2D program(Ref.2).

A critical factor for the evaluation of the seismic behaviour of the structure analysed was the estimation of whether the R/C members were capable to develop the plastic rotations indicated by the analysis. The plastic rotation capacity of a member end region was calculated as

$$\Theta_p = (\phi_u - \phi_y)\, l_p$$

where ϕ_u and ϕ_y are the curvatures of the critical section at ultimate and at yield respectively, calculated from the section geometry and reinforcement, using appropriate stress-strain models for concrete and steel, and l_p is the equivalent plastic hinge length, calculated according to standard empirical relations(Ref.3). It should be pointed out that in the case of columns the conservative approach was adopted to calculate ϕ_u and ϕ_y as those corresponding to the largest axial load (N_{max}) recorded during the dynamic analysis, although, in general, $(\Theta_p)_{max}$ and N_{max} do not occur simultaneously.

The earthquake accelerograms used in the present study included the well-known El Centro, 1940 S00E and Pacoima Dam, 1971 S16E, as well as a greek record, namely the Thessaloniki, 1978 N30E. All records were normalised according to their spectrum indensities (SI) to 0.56, 1.00 and 1.50 times the intensity of the El Centro motion ($SI=SI_o$). If these intensities are correlated to the SI of the design spectrum given by the MC/SD(Ref.1), it is found that they correspond to effective peak accelerations (A_{ef}) of 0.15, 0.27 and 0.40g, respectively (note that the corresponding nominal peak accelerations are 0.20, 0.35 and 0.52g). The last value was considered as representative of the maximum credible earthquake in the area of the structure under consideration.

DISCUSSION OF RESULTS

The main response quantities for the three records used in the study, normalised to the intensity of El Centro S00E, are shown in Fig.2. As expected (Ref.3), the response varies with each motion and different motions produce critical response at different parts of the structure. Overall it appears that the Thessaloniki record produces maximum response quantities, at least for the intensity considered. It is worth noting that, as shown in Fig.2(c), the wall is the main seismic load resisting element in the ground storey, while, at the top storey, the columns dissipate most of the input seismic energy (typical behaviour for a dual structural system).

The distribution of plastic hinges along the height of the building, for various levels of earthquake intensity, is shown in Fig.3. Full circles indicate yielding at both faces of a member, while open circles indicate that yielding took place at one face only. It is seen that for the design intensity, corresponding to A_{ef}=0.15g, only one column in the top storey yields and so does the wall at ground level only, while the rest of the vertical elements remain elastic. It appears that the capacity design procedure used has succeeded in preventing the formation of plastic hinges in areas other than the ground level and the top storey. It is worth noting that even for the maximum intensity (A_{ef}=0.40g) considered, no vertical element sidesway mechanism is expected to form, mainly because the wall remains elastic in all but one of the storeys above the ground storey. In the most critical situation (6th storey) the columns yield at both ends, while the wall yields at its bottom end only.

The envelopes of response quantities for some characteristic levels of earthquake intensity are shown in Fig.4. From the results presented herein it appears that the main design objectives, as stated in the MC/SD, have been acheived. In fact, as shown in Fig.4(a),(b) the displacements calculated for the design intensity (A_{ef}=0.15g) are systematically lower than the values intended by the Code (design displacements factored by the behaviour factor, K). The only storey where the interstorey drift is somewhat higher than that predicted from the Code is the ground storey, obviously because this is the only place where a hinge forms in the wall. It is worth mentioning that, for the design intensity, the interstorey drifts (Fig.4b) only slightly exceed 0.2% the storey height, which is the threshold of damage to nonstructural elements, as specified in many codes.

In the case of the maximum credible earthquake, taken here as that corresponding to A_{ef}=0.40g, the design requirement is that of "no collapse"(Ref.1). As suggested in a previous study(Ref.4) the collapse criteria should include the formation of vertical element mechanisms, the exceedance of a limit value of the interstorey drift(2% the storey height) and the exceedance of the plastic deformation capacity in one or more members. In Fig.3(c) it is shown that no mechanism is expected to form, while, as shown in Fig.4(b), the peak values of the interstorey drift do not exceed 0.73% the storey height, which is well below the specified limit of 2.0%. With respect to the plastic deformation capacity, it was found that the critical elements are the columns, since the available ductilities of the beams and the wall were far in excess of the requirements indicated by the analysis(Ref.3). Although the rotational ductility factors are higher for the columns in the upper stories(Fig.4c), it was found that the situation is more critical for the columns at the ground storey, due to the considerably higher axial loads acting on these members. The plastic rotation requirement for the ground storey column was found to be Θ_p=0.00259 rad, combined with an axial load equal to 0.34N_u, where N_u is the uniaxial compression strength of the member. The corresponding rotational capacity was calculated to be Θ_p=0.00654 rad, that is 2.5 times the corresponding demand. It should be noted here that the plastic rotation capacity in the stories above the first ranged from 3 to more than 10 times the corresponding demands, indicating that the transverse reinforcement of the columns was apparently far in excess of that required in the case of the maximum credible earthquake.

Finally, in order to study whether shear could have been a problem for the wall, the maximum values of wall shears, for A_{ef}=0.40g, were (conservatively) compared with the capacities predicted by the ACI 318-83(Ref.5). As shown in Fig.5, the maximum seismic shears are always lower than the corresponding capacities, with a minimum "safety factor"(at the second storey, where the height changes from 4.5 to 3.0m) of 1.34. It should be kept in mind that the ultimate shear capacities of the wall will be well above those predicted by the ACI equations(Ref.3).

CONCLUSIONS

The present study indicated that the behaviour of a dual structure, designed to the provisions of the CEB MC/SD(Ref.1) is satisfactory, characterised by relatively low displacements, uniformly distributed along the height and column ductility requirements lower than the corresponding available ductilities. The key to this satisfactory performance was that, as intended by the capacity design procedure used, the wall yielded at ground level only, even for earthquake intensities far in excess of that considered in the design.

On the other hand, the present study indicated that the behaviour factor specified for this type of structure appears to be too conservative, at least so far as shear does not govern the behaviour of the wall. Also the ductility supply in most of the upper stories was much higher than the corresponding demand, for both the beams and the columns. It is believed that a more rational balance between strength and ductility can be achieved if inelastic seismic analysis is used as a design tool, as suggested by some investigators.

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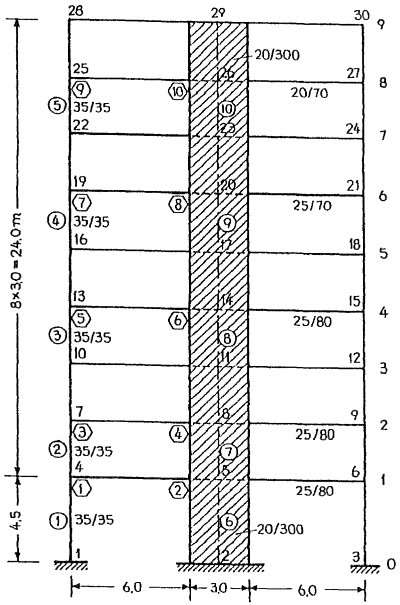

Fig.1 Geometric data for the structure analysed (section dimensions in cm).

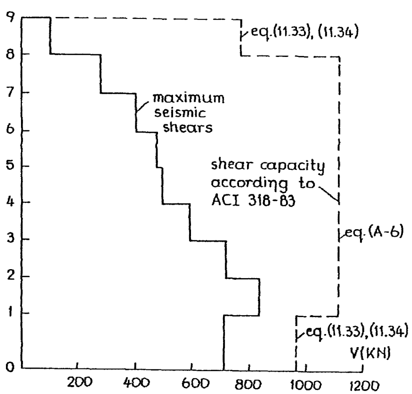

Fig.5 Maximum seismic shears and corresponding capacities according to ACI 318-83.

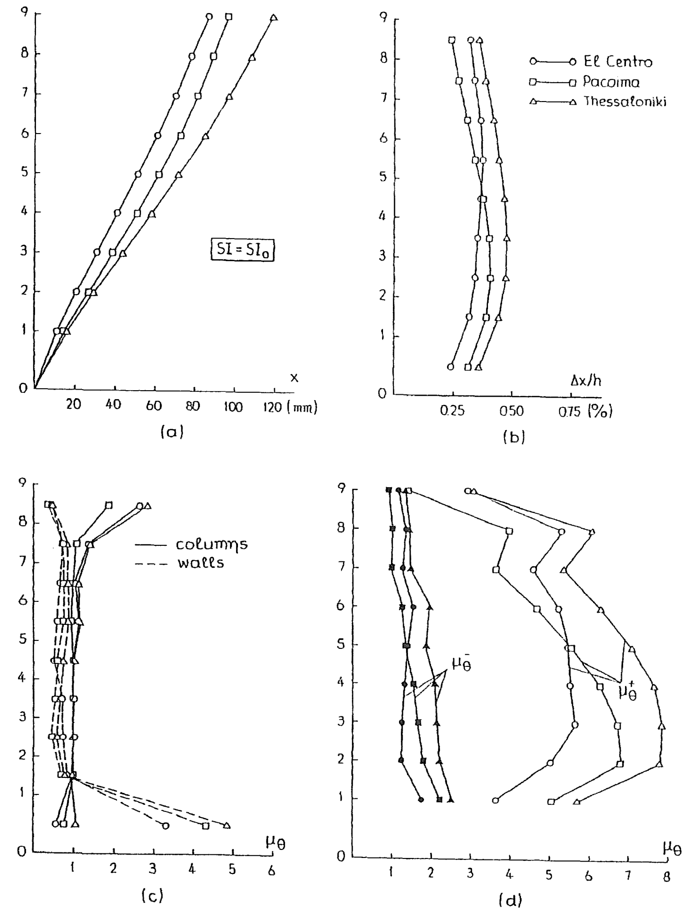

Fig.2 Maximum response quantities for various motions normalised to $SI=SI_o$.(a)Storey displacements,(b)Interstorey drifts,(c)Rotational ductility factors for vertical elements,(d)Same for beams.

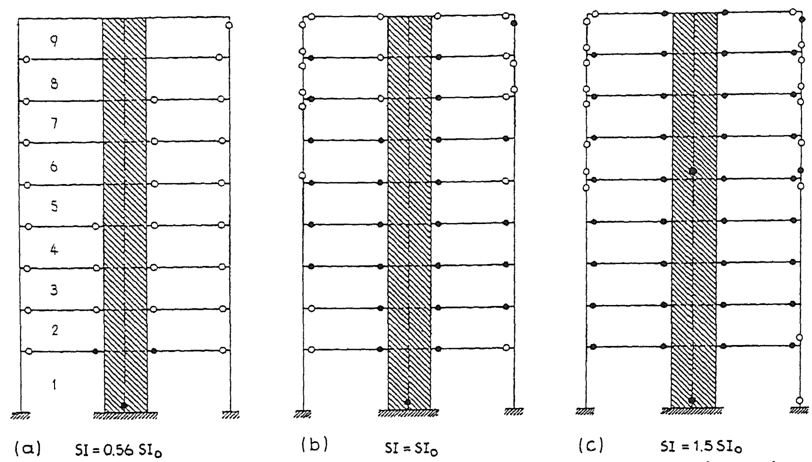

Fig. 3 Distribution of plastic hinges in the structure for various intensities of the input motion.

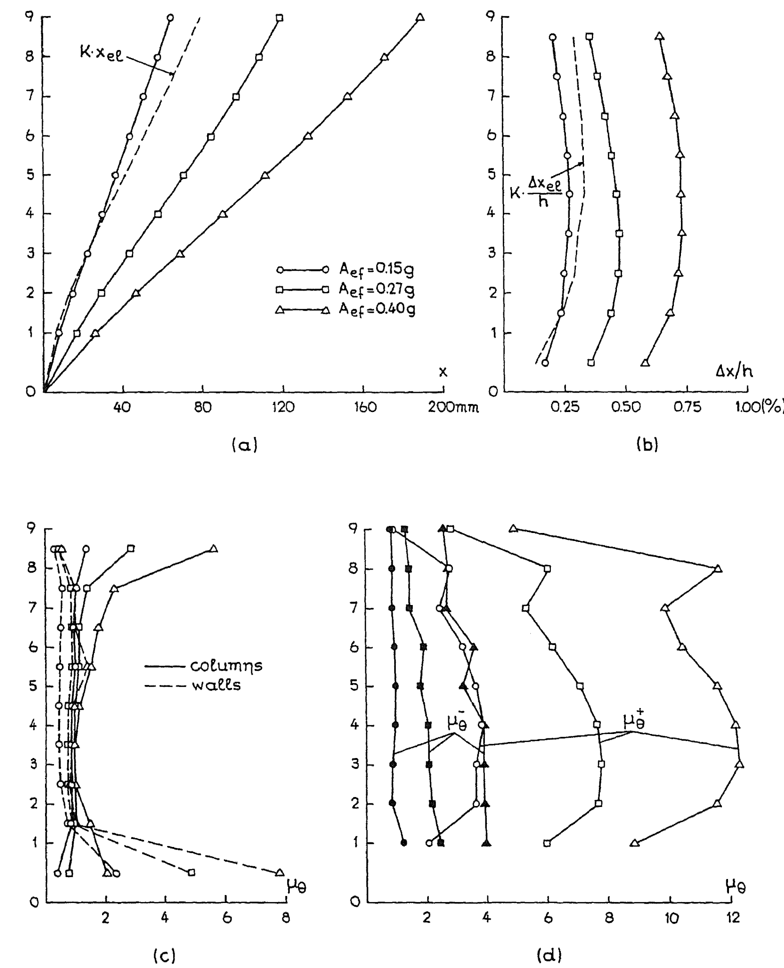

Fig. 4 Maximum response quantities for characteristic earthquake intensities (a, b, c, d as in Fig.2).