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INFLUENCE OF CAPACITY DESIGN METHOD ON THE SEISMIC RESPONSE OF R/C COLUMNS

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ABSTRACT

The paper aims at assessing the influence of the design procedure followed in designing the columns of a reinforced concrete (R/C) building, on the performance of the columns, as well as of the structure as a whole, when subjected to seismic loading, to identify potential weaknesses in currently adopted procedures, and to present a new procedure which is based on currently available powerful analytical tools, and results in increased reliability with regard to seismic loading. Two case studies are presented, involving multistorey reinforced concrete buildings with frame and dual structural systems subjected to various appropriately scaled input accelerograms. The results obtained indicate that capacity design of columns results in adequate safety margins against failure even when the adopted overstrength factors are quite low, but hinging in columns is not avoided unless very high overstrength factors are used; the suggested novel technique of capacity design led to a substantially improved seismic performance with a minimal increase in cost.

KEYWORDS

Capacity design; reinforced concrete columns; seismic codes; dynamic inelastic analysis.

INTRODUCTION

The importance of *columns* with regard to the behaviour of structures subjected to lateral loads, in particular accidental loads of dynamic nature, has long been recognised and a higher degree of protection is sought in the design of these critical elements; this is typically achieved by the so called "*capacity design*" procedure which sets aside the results of analysis and aims at establishing a favourable hierarchy of strength in the structure, by ensuring that the strength of columns is higher than that of the adjacent beams, with possible allowance for beam overstrength (Paulay, 1979; Penelis and Kappos, 1996). Although capacity design procedures are since widely used in many countries, the numerical values to be attributed to overstrength factors, in relation to the acceptable performance criteria (which are different at each limit state), are still based mostly on empirical judgement and reason, rather than on quantitative data derived using appropriate scientific methods (Pinto, 1994).

Early work at the Univ. of Canterbury (Paulay, 1979) including inelastic dynamic analysis of various buildings, was used for developing the pioneering NZS3101 Code (Standards Assoc. of New Zealand, 1982). Among later studies, probably the most comprehensive one is that by Dolce and Evangelista (1990) which addressed plane two-bay frames with four or eight storeys, designed for a large number of overstrength factors (ranging from 1.0 to 3.0). Recent work includes that of Colangelo et al. (1995) which is based on the equivalent linearization procedure of reliability analysis; the work addresses randomness of mechanical properties, in addition to that of the dynamic input.

The *aim* of the research presented herein is to assess the influence of the capacity design procedure on the performance of columns and of the structure as a whole, and to explore the possibility of developing new procedures which might combine lower cost of construction with increased reliability with regard to transient dynamic loading. The latter procedure is based on the concept of incorporating currently available analytical tools for *dynamic time-history analysis* into design; this is currently allowed by the Eurocode 8 (EC8), but no specific guidance is given, except for the selection of input motions.

PROBLEM STATEMENT - CRITIQUE OF EXISTING PROCEDURES

The fundamental idea behind the capacity design of columns for flexure is that at any beam-column joint the moment input to the columns is equal to the sum of the beam moments, $M_{b1}+M_{b2}$, and if the beams are allowed to yield under the design earthquake (according to normal current practice), this input does no longer depend on the characteristics of the earthquake. Hence, the maximum moment input M_{ci} to *each* column (i=1, 2) is equal to $\lambda_i \Sigma M_{ub}$, and the requirement that no yielding occur in the column can be written as

$$M_{yci} > \lambda_i \Sigma M_{ub} \tag{1}$$

where M_{yci} is the yield moment of column i and ΣM_{ub} is the sum of maximum moments that can be developed at the beams subsequent to yield at the bottom and the top. It is worth pointing out that eq. (1) is not the same as those included in most current seismic codes which require the *sum* of the column strengths to be higher than $\gamma_R M_{ub}$, where $\gamma_R \geq 1$ is a beam moment magnifier, typically varying between 1.15 and 1.40. It is also interesting to note that the factor λ_i used herein to denote the fraction of ΣM_{ub} carried by column i, can be larger than 1 if either column is in single curvature (as the case would be in a structure with very flexible beams or with flat slabs).

On the basis of eq. (1) it is now clear that there are distinct sources of uncertainties associated with the capacity design of columns.

(i) Uncertainties relating to *beam overstrength* (ΣM_{ub} in eq. 1): The main parameters affecting beam overstrength are
   a. *Strain-hardening* of the steel bars in the beam which have entered the inelastic range.
   b. *Contribution of slab reinforcement* in the "negative" flexural capacity (M^{-}_{ubi}) of the beam. Modern codes such as EC8 (CEN, 1995) recognise these effects by specifying effective slab widths, which however are usually much smaller than the maximum values measured in relevant tests in particular at interior joints.
   c. *Beam yielding in two directions*, which can induce a moment up to 41% ($\sqrt{2}$:1) higher than that indicated by eq. (1) along the diagonal of a square column.
   d. *Material variability*, in particular that associated with yield strength of steel.

(ii) Uncertainties relating to *column understrength* (M_{yci} in eq. 1): The main parameters affecting column understrength are
   a. *Variations in axial loading* in the column due to varying seismic overturning moment and to the vertical component of the earthquake.
   b. The *biaxial strength* of a column is typically of the same order as its uniaxial strength, while, as pointed out previously, the moment induced (by simultaneously yielding beams) in a skew direction may be up to 41% higher than that along either of the column axes.
   c. *Construction tolerances* may lead to reduced strength of the column.
   d. *Material variability*, in particular that associated with yield strength of steel.

(iii) Uncertainties relating to the *distribution of moment input among the columns* above and below a joint core (λ_i factor in eq. 1): This is an important factor commonly disregarded in many codes and is affected by the following parameters

a. *Dynamic magnification* of column moments, with respect to the values calculated from a standard "equivalent-static" or "simplified modal" analysis.

b. *Stiffness changes* in the members framing into a joint, in particular the beams, which subsequent to yielding have a much lower stiffness than that assumed according to any current seismic code.

(iv) The presence of *infill panels*, in particular brick masonry or concrete masonry walls, will significantly affect the stress distribution in a building, and eq. 1 is no longer applicable.

Based on the foregoing discussion it is clear that there are at least nine factors (ii.b and ii.d are essentially the same factors as i.c and i.d) that influence the capacity design of columns, and a rigorous procedure should account for all these factors, without being overconservative (as the New Zealand Code is felt to be).

DESIGN PROCEDURES

Two regular R/C buildings were designed, one with a structural system consisting solely of frames, and one dual (wall and frame) structure. Due to symmetry and regularity in plan the two buildings were idealised as the plane structures shown in Fig. 1.

As the main purpose of the study was to evaluate the effect of column capacity design procedure, the beams of the structure (and the wall in the dual system) were designed according to the CEB (1985) Model Code, for an effective peak ground acceleration A_d=0.25g and the intermediate ductility level (DL II); the same code was also used for the shear design of all members. This code ensures an acceptable seismic reliability without being too conservative with regard to hoop reinforcement requirements, as the case was found to be with EC8 (Kappos and Antoniades, 1995). The present study focuses only on *column* design. The flexural design of columns was carried out using different capacity design procedures, as described in the following.

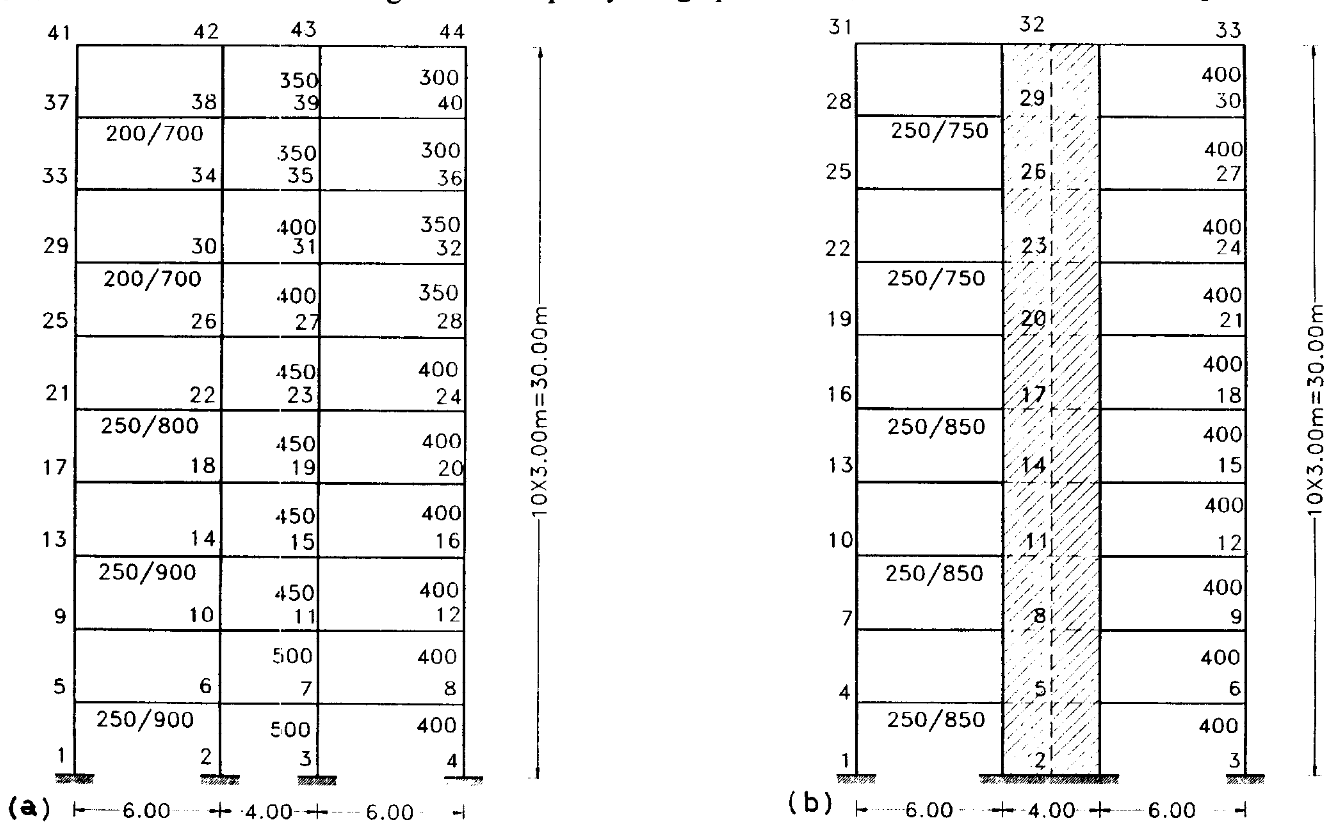

Fig. 1 Geometry of the structures analysed; (a) Frame; (b) Dual system (wall thickness 250mm)

Currently existing methods

The methods considered included:

(i) The "traditional" capacity design procedure adopted by various american and european codes, involving factoring beam strengths at a beam-column joint by appropriate overstrength (γ_R) factors and providing columns with a sum of resistances exceeding that of beams. The γ_R values used were 1.00 (with and without an additional dynamic magnification factor ω_D, as suggested in the CEB Code), and 1.50, a reasonable upper limit for practical design, as may be inferred from previous studies (Dolce and Evangelista, 1990, Colangelo et al., 1995).

(ii) The Eurocode 8 (CEN, 1995) procedure of the a_{CD} (effect of $\gamma_R \Sigma M_{ub}$) and δ (moment-reversal) factors; the later accounts for the reduced possibility of hinge formation at the bottom of gravity load dominated beams. Two cases were considered, one wherein only column flexural design was modified with regard to the reference structure, and one where the full set of EC8 provisions was considered.

(iii) The approximate method of doubling the seismic moments (M_E) in columns, which is analogous to the procedure suggested by some national codes for low or intermediate ductility structures.

(iv) A modified EC8 procedure (which drops the δ–factor), adopted by the new (1995) Greek Seismic Code.

Suggested new method

In addition to the foregoing, a novel procedure of designing columns on the basis of inelastic dynamic analysis, wherein columns are assumed to remain elastic, while beams are allowed to yield, was applied. The part of the procedure regarding flexural design may be summarised as follows:

(i) *Design of the beams* (and *wall* critical regions, in dual structures) according to standard code procedures, for seismic as well as for gravity loading; design moments coincide with those calculated from analysis.

(ii) *Detailing* of the flexural reinforcement of beams (and wall critical regions), taking into account minimum code requirements and convenience of construction (e.g. use of a limited number of bar diameters).

(iii) Selection of an appropriate set of input *accelerograms*, using the techniques described in some modern seismic codes, such as EC8; either artificial, spectrum-compatible, or actually recorded motions appropriately scaled to match the design spectrum, may be used (see also section on input motions).

(iv) Construction of a *model* of the structure wherein beams are modelled as yielding elements, with their strength based on the reinforcement actually present (including that in the adjacent slab), and with due consideration of factors such as stiffness degradation. In the same model, columns, as well as portions of walls (when present) intended to remain elastic, are modelled as *elastic* members. With regard to initial stiffness assumptions, it is recommended to use 40% to 50% the gross section stiffness (EI_g) for beams and 75% to 80% EI_g for columns (Paulay, 1979; Kappos, 1991), to account for (pre-yield) cracking, which is different in each type of member. For convenience of design this model may also be used for the first set of analyses (step one).

(v) *Time-history analysis* of the model described in the previous step for the selected set of input motions; calculation of critical moment (M) and axial load (N) combinations for each column (and wall) critical section.

Design and detailing of columns (and walls); for a column subjected to *biaxial* loading, consideration of the following three combinations will be sufficient for most practical purposes (M_y and M_z are the moments acting along the two main axes of a column): (1) max$|M_y|$, and corresponding M_z and N; (2) max$|M_z|$, and corresponding M_y and N; (3) min $|N|$ (compression) or maxN (tension), and corresponding M_y and M_z. For uniaxial loading two combinations will suffice. The foregoing are valid on condition that the axial load limitations imposed by EC8 are respected; these limitations should be checked using the min N (max compression) calculated in the time-history analysis.

The key to the success of the previously described method is to provide in the model used for the time-history analysis for as many of the uncertainties described in the previous section as feasible; indeed the only factor that is difficult to include in the method is material variability (several analyses have to be carried out with various combinations of material characteristics and this tends to make the method less efficient. Tolerances (if they can be estimated) may be easily accounted for by slightly modifying the design resistance of columns.

Final designs

In designing the two buildings according to the various methods, care was taken to retain some common features in all designs (in addition to using the same beams). A key aspect in this regard is that the reinforcement ratio in columns was kept as uniform as possible, so that the ductility of the various designs (which is affected by this ratio, both directly with respect to the rotational capacity, and indirectly with respect to the shear stress that can be developed) remained essentially unchanged. This resulted in increases in column dimensions of up to 100 mm.

METHOD OF ASSESSMENT

Modelling aspects

The analysis of the structures was carried out using DRAIN-2D/90, an extended version of the well-known DRAIN-2D program, developed at the University of Thessaloniki. Standard point hinge modelling was used for R/C members, with degrading hysteresis constitutive laws governing the behaviour of each hinge. Details of the modelling assumptions may be found in Kappos (1991) and Penelis & Kappos (1996). It is noted that negative yield moments in beams were estimated assuming some additional contribution from the slab reinforcement which increased the area of top bars by 12 to 16%.

The possibility of failure in each member, as well as in each storey of the buildings, was checked by applying appropriate *failure criteria,* local as well as global. *Local failure* corresponds either to the exceedance of the available plastic rotation capacity of a R/C member, taking the adverse effect of high shear stresses into account, or to the development of a shear force exceeding the corresponding capacity of the member; the latter is checked with respect to all possible failure modes, that is diagonal tension, diagonal compression (web crushing) and sliding shear. As the shear failure criterion is based on strength rather than on deformability, it is generally more conservative than the rotational capacity criterion. *Global failure* is assumed to coincide with storey failure; a dual criterion based on a limiting interstorey drift of 2% and the development of a sidesway collapse mechanism involving all vertical members has been adopted for assessing storey failure. Details of the local and global failure criteria used may be found in Penelis & Kappos (1996).

Input motions

Most of the available records from earthquakes that caused serious damage, including collapses and casualties, in Greece during the last twenty years were used. Six of the records were from the earthquakes of Volvi (1978), Alkyonides (1981), and Kalamata (1986). All these records are characterised by the fact that they come from surface earthquakes with small epicentral distances (these are the typical destructive earthquakes in Greece); hence the selected set of records can be considered as representative for the design of structures in Greece. For comparison purposes, the well-known N-S component of the El Centro 1940 record was also included in the study. These seven records were used for all structures. A second set of four greek records (Argostoli 17.1.83, N-S and E-W components; Argostoli 23.3.83, E-W component; Edessa 1990 N-S component) and the well-known S16E component of the Pacoima Dam (1971) record (representing an event significantly different from those commonly occurring in the area under consideration), were used in the case of the structures designed to the suggested new methodology, in order to study their reliability when the earthquake input is different from that used in the design. All records were normalised to fractions of the intensity of the design earthquake (A_d=0.25g), using the modified Housner technique suggested by Kappos (1991), correlating the velocity spectrum of an input motion to the corresponding code-derived one.

DISCUSSION OF RESULTS

A selection of the most characteristic results from the extensive parametric study involving the sixteen different (as far as columns are concerned) designs and the seven+five records scaled to various intensities, are presented in the following.

Shown in Fig. 2 are the distributions of plastic hinges in the two structures designed for the minimum capacity design requirements (γ_R=1, no ω_D factor); open circles correspond to yielding at one face of a member, and black circles indicate yielding at both faces. As expected, this minimal design of the columns does not prevent hinge formation in these members under the design earthquake. In fact at the sixth and the eighth storey of the frame the potential of a column sidesway mechanism is detected (Fig. 2a); however due to the fact that yielding does not occur at all hinges at the same time, this mechanism is not as critical as in the case of statically applied loading (Kappos, 1991). Hinges form at all columns of the lower storeys of the dual structure (Fig. 2b), but in this case the wall remains in the elastic range (except at its base) and no sidesway mechanism can form. Hence if the performance criterion is to avoid column hinging (above the ground level), this capacity design procedure is obviously inadequate. On the other hand, the plastic rotation requirements in the columns are quite moderate in both structures, the maximum values not exceeding 0.006 rad. Plotted in Fig. 3 are the average (over 7 motions) and the peak values of plastic rotations ($\theta_{p,req}$) recorded during the dynamic analysis, together with the corresponding available rotational capacities ($\theta_{p,avail}$) calculated on the basis of the previously mentioned local failure criteria for the most critical motion (conservative approach). It is clear from Fig. 3 that the capacities are well above the corresponding demands the min "safety factor" ($\theta_{p,avail}/\theta_{p,req}$) being at least 1.9 (third storey interior column of frame structure). Checks of the column shear capacities have shown that the corresponding safety factor is 2.5 or more for all columns. It is reminded that the hoop reinforcement, on the basis of which local failure criteria were evaluated, was significantly lower than that required according to EC8. Hence if the performance criterion is to ensure an adequate ductility (and shear capacity) of the columns under the design earthquake, it can be claimed that even this minimal capacity design might be adequate in this respect.

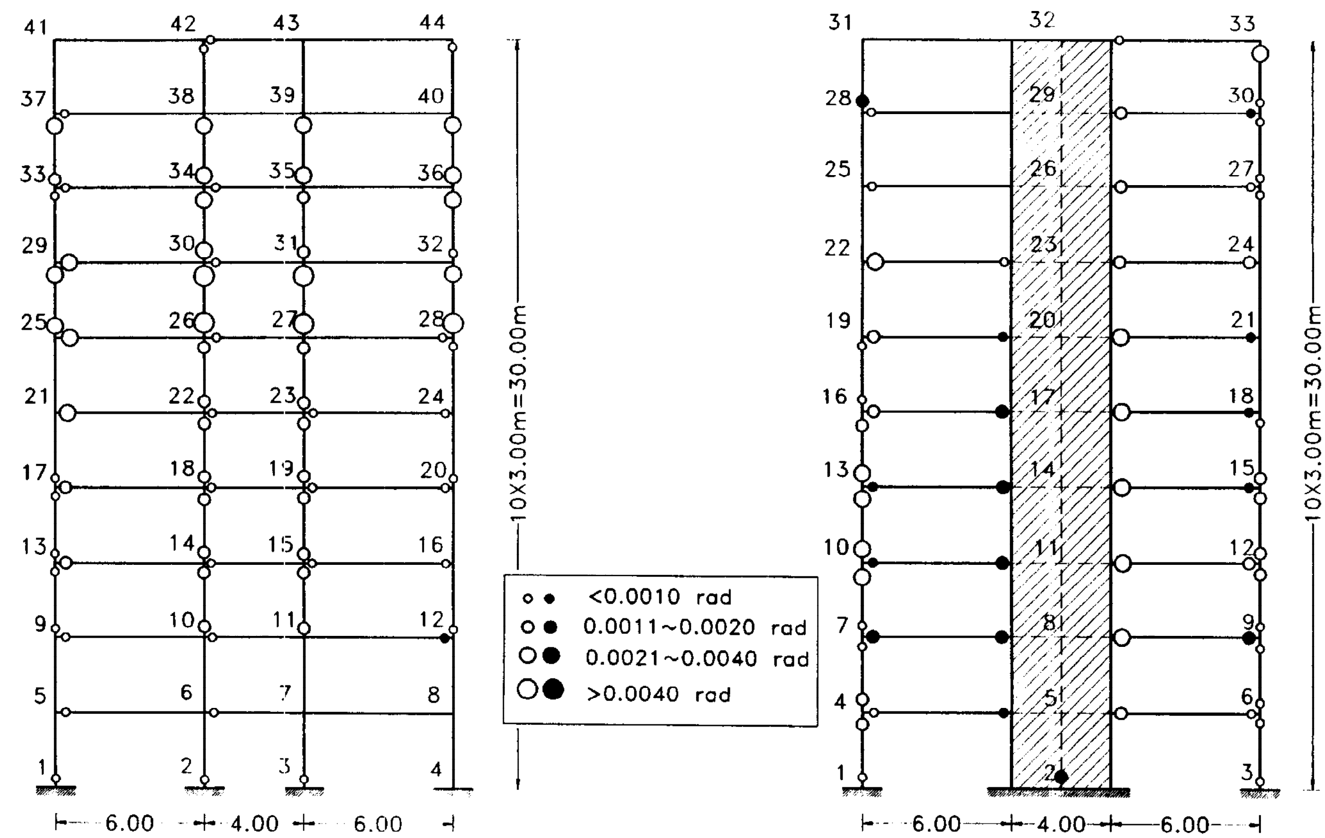

Fig. 2 Plastic hinge distribution in structures designed for γ_R=1.0

Increasing the overstrength factor either directly (γ_R=1.50), or indirectly by introducing the ω_D factor of the New Zealand and the CEB codes, did not significantly alter the previously described picture, as column hinging did occur, sporadic in the former case but in the latter one even a potential column mechanism appeared in the same storeys as in the structure designed for γ_R=1.0. The difference was that the required plastic rotations were now lower and the corresponding min safety factors were 2.3 for both designs (but not for the same structure; the columns of the frame were the critical ones in the former case and those of the dual system in the latter); moreover the safety factors were generally higher than for γ_R=1.0. Therefore, increases of γ_R of up to about 50% do not preclude hinge formation in columns, as also demonstrated in previous studies (Dolce and Evangelista, 1990).

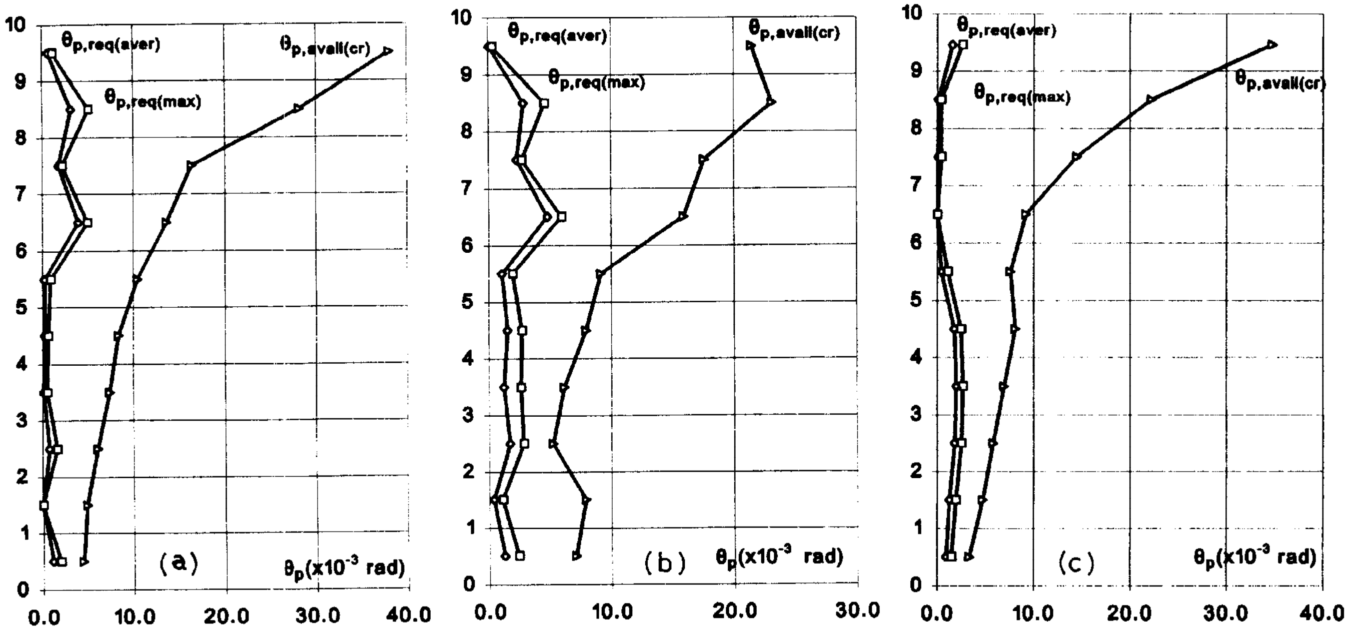

Fig. 3 Required and available plastic rotations in the columns of the structures designed for γ_R=1.0
(a) Frame, ext. columns; (b) frame, int. columns; (c) dual system.

The overstrength factor adopted by EC8 for DC "M" structures is γ_R=1.2 and in the upper storeys it is somewhat reduced through the δ-factor. Hence, it was no surprise to find out that the behaviour of columns in the structures designed only to the EC8 provisions for column strength was very similar to that of the previously discussed designs, with a potential mechanism appearing at the sixth storey of the frame (where a column taper occurs) and a minimum safety factor with regard to rotational capacity of 2.8. Introducing the remaining EC8 provisions for columns resulted in some improvement, with a minimum safety factor of 5.0 (in the frame); however a potential mechanism now appeared at the eighth storey.

The modification to the EC8 capacity design of columns introduced in the new (1995) Greek Seismic Code consists in dropping the δ-factor and distributing the sum of beam moments (ΣM_{ub}) according to the seismic (not the total) column moments. This procedure led to a further improvement in that no potential mechanism appeared in either structure, but the minimum safety factor recorded at the base of the ground storey columns was only 1.9 (this became the most vulnerable part of the frame structure). Nevertheless factors of 5.5 or more were recorded in all other storeys.

Among the conventional capacity design methods, the best protection of columns was achieved, in the case that the seismic moments were doubled (regardless of beam strengths. Column hinging was essentially restricted to the base of the exterior columns of the ground storey, in both the frame and the dual system (the pertinent safety factor was at least 11.3), and at the top storey columns, as intended by the design; some minimal plastic rotations (0.001 rad, or less) were recorded at a few intermediate storeys of the frame. Hence this procedure may be deemed to satisfy (though not literally) the performance criterion of no hinge formation in columns other than those of the first and the top storey.

Finally, both structures designed to the proposed new methodology performed exactly as intended to, even when they were subjected to the set of input motions that had not been previously considered in the design process. Column hinging appeared only at the base of the frame structure, and at the base and the top of the dual system; for one from the second set of 5 motions one column section at the ninth storey just entered the inelastic range (θ_p=0.2·10^{-3} rad). Given that inevitably only a few critical load (M,N) combinations were considered in design, the foregoing appears to be an excellent performance, clearly satisfying the requirements of all modern design codes. This improved performance resulted by just a minor increase of material cost (column sections were up to 5 mm larger than those in the standard EC8 design), which can be more than offset if the *confinement* requirements in columns are drastically reduced in most storeys, on the basis that columns will remain elastic or very nearly so. A full calibration of this significant aspect of the proposed method clearly requires consideration of the maximum credible earthquake (in addition to the design one) and is currently under investigation; pertinent results will appear in a future CEB Bulletin on Seismic Design (CEB TG 3/2). It is worth pointing out that the suggested procedure is not limited to regular structures, and in fact studies involving partially infilled frames are planned for the near future.

CONCLUSIONS

The evaluation of various existing capacity design methods for columns presupposes the clarification of the related seismic performance criteria, which unfortunately is not done in most, if not all, current codes (the EC8 not excluded). If the target performance is to *avoid column hinging* under the design earthquake at all storeys (except at the base and the top of the structure), then the present study (as well as some previous ones) has clearly demonstrated that overstrength factors of 2.0 or more have to be introduced, even for the regular structures studied herein. If the target performance is to *avoid formation of column sidesway mechanisms*, the overstrength factors can be reduced (to some value higher than 1.5), while if the only requirement is to *provide columns with sufficient ductility* (in addition to adequate shear capacity) to withstand the design earthquake without serious damage, then γ_R values slightly higher than 1.0 might be appropriate, at least for regular structures.

Among the various procedures studied, the very simple method of doubling the seismic actions in columns resulted in a particularly good performance (again the issue of regularity has to be reminded here), while the suggested new procedure of capacity design led to a very satisfactory seismic performance, and may also lead to a reduced cost of materials compared to that resulting from the code methods, if an adequate balance between provided flexural strength and corresponding confinement is achieved.

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