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EARTHQUAKE DESIGN OF REINFORCED CONCRETE STRUCTURES BASED ON LOCAL DUCTILITY COEFFICIENTS

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SUMMARY

A methodology to improve the earthquake design of R/C buildings is presented. This methodology idealizes the decision-making processes in the design of a building and the effects of future earthquakes. The concept of a design process is put forward as directly related to decision-making and only as indirectly related to structural performance under earthquake vibration; this concept is ilustrated through a discussion of the "Local Ductility Coefficients" process. The quantification of the values of those coefficients is carried out for R/C frame structures, for ilustration purposes and to support a preliminary assessment of the advantages of this design process.

INTRODUCTION

The optimization of design methods have ever been considered an important goal; in a large number of engineering fields, this optimization can be equated, under general agreement, with the minimization of an well-defined loss function e.g. the weight or cost of the structure. The substancial risks and expenses involved in earthquake engineering and the long term consequences of alternatives selected under large uncertainties and deficient information, ensures that no simple criterium should be expected to obtain the accord of all parties involved . Hence, it becomes appropriate to identify, under general consensus, the principles and guidelines that shall govern the optimization of design methods and then derive, as far as possible by rational procedures, the design methods themselves or, at least, quantify the values of their main parameters. A set of principles for earthquake design was presented in 1977 (Ref. 1) and a state-of-art monograph was published in 1982 (Ref. 2).

The rationalization of the design methods have underlain the preparation of the recent (1983) portuguese earthquake resistant code (Ref. 3); this rationalization was substantially based on the definition of levels for performing the earthquake analysis of building structures (Refs 4,5), by matching the sophistication of the idealizations of building structure an earthquake vibration. The first level of analysis is an exact level, in the sense that no simplifying hypothesis are assumed; thus, nonlinear behaviour of the structure is considered and the earthquake action must be represented by a sample of a stochastic process i.e. by several time-series of acceleration; since step-by-step integrations must be resorted to, analysis are so expensive that this level can only be envisaged as a reference for assessing the validity of the simplifying assumptions of the other levels. The second level of analysis is defined by the avoidance of explicit nonlinear analysis, which is accomplished by the consideration, for suitably defined classes of structures, of behaviour (or ductility) coefficients; those coefficients are supposed to transform the values obtained by a linear dynamic analysis into the values that would be obtained if a nonlinear analysis were performed. The third level of analysis deals with the possibility of analysing the structure only along some appropriate directions and the fourth level with the substitution of the dynamic analysis by an equivalent static analysis with a set of horizontal forces. It should be emphasized that the new portuguese code formally deals with the checking of the safety of structures and not with its design; it has happily been proved possible to establish that if structures are designed according to some explicitly defined method, the safety checkings would be observed ipso facto (Ref. 5).

In this paper is investigated the possibility of obtaining better earthquake behaving structures through a 2nd level design process based on local ductility coefficients and not on a global ductility coefficient. The equivalence between design process and safety checking is established in the context of the methodology presented in the next section. Only a non-collapse requirement will be considered, in spite of recent studies (e.g. Ref. 6) which highlighted the possible greater importance of serviceability (and reparability) requirements.

BASIC CONCEPTS AND PURPOSE OF THE DESIGN METHODOLOGY

The methodology is based on seven basic concepts, some of which are interdependent in the sense that presently they can only be defined by recurrence; yet some steps have been made already towards a more general definition (Ref. 6).

a) Structural system: an arrangement of structural elements which provide stiffness and resistance in horizontal directions, e.g.: a frame, a shear-wall, a coupled shear-wall ... The distinction between structural systems shall consider their behaviour in the post-elastic range; hence a frame designed to have plastic hinges in the beams shall be considered a different structural system from a frame with hinging occurring in the columns.

b) Control variables: the variables that quantify the behaviour of the structure in terms of its ultimate earthquake withstanding capacity, e.g.: interstorey drift, damage indexes, ... Some types of control variables are generally associated to some types of structural systems (e.g. frames and interstorey drift).

c) Class of structures: the ensemble of those structures with the same type(s) of control variables and such that those variables take on similar values under similar earthquake actions. It is expected that buildings of about the same height, with the same type of structural system, and design by the same design process shall belong to the same class of structures.

d) Design process: an ensemble of instructions which permit the determination (explicitly or implicitly) of the limit elastic resistance and ductility capacity to be provided in each structural element, e.g. an earthquake resistant code plus an handbook on structural analysis. In the present paper only those design processes which comprise a linear dynamic analysis will be considered (or an equivalent horizontal forces static analysis (Ref. 4).

e) Design limit states: the maximum values the control variables can present, with the structure acted by the design seismic action, within the frame of a given design process.

f) Design seismic action: an earthquake - like vibration, representative of the severe earthquakes with highest likelihood of ocurrence, or any idealization thereof that can be used as the earthquake input in a linear analysis, as a response spectrum.

g) Acting earthquake motions: a group of earthquake vibrations (recorded or artificial) for which the earthquake withstanding capacity of the building is assessed in terms of collapse occurrence. In general those vibrations will have greater severity than the design seismic action and will represent "critical" earthquakes in terms of building survival.

The purpose of the methodology is to model the decision making process behind the design of a building and the occurrence history of earthquakes it will be subjected to, after it is built. Concepts a), b) and g) are of an objective nature, concepts d), e) and f) are of a "strategic" nature (related to decision making) and concept c) is a smoothness assumption so that results obtained for a few cases may be deemed of a wider applicability; concepts e) and f) are subordinated to d).

THE METHOD OF LOCAL DUCTILITY COEFFICIENTS

It is assumed that the structural systems of the buildings to be considered may be idealized as an association of plane structural systems (tichoidal structures). Tichoidal structures are defined (Ref. 5) as structures whose elements are disposed in the neighborhood of a vertical plane and with a stiffness, relative to horizontal forces, much greater in the direction of that vertical plane than in the orthogonal direction; moreover, moderate displacements in this latter direction should not cause significative stresses in the structural elements; hence plane frames, shear-walls and coupled shear walls are tichoidal structures.

In the present application of the method of local ductility (LD) coefficients, ductility coefficients are defined separately for each tichoidal structure and for the elements in a given type of structure, the LD coefficient is obtained by the product of the coefficient of the element (ED) by the coefficient of the structure (DS): LD = ED.DS. Hence, ED coefficients are quantified for all structural elements in frames, shear-wall, ... Classes of buildings are defined in terms of the distribution in plan of the component tichoidal structures; to each tichoidal structure, in a given class of buildings, a specific value of the DS coefficient is attributed.

The method of the LD coefficients is very similar to other design methods recently proposed (Ref. 7,8). Its application to asymmetric structures and structures with vertical irregularities may be found elsewhere (Ref. 9,10).

EXEMPLIFICATION STUDY

To illustrate the possibility of quantificating the LD coefficients design process an exemplification study was carried out for symmetric buildings with frame structures (Ref. 11). Buildings with 2, 4 and 8 storeys were considered with plan dimensions of 15 x 15 m^2. Those buildings were designed for a reference earthquake with a peak acceleration of 50 cm/s^2 according to 6 design processes, each design process being specified by a given distribution of LD coefficients. The influence of torsion was disregarded and thus to the DS coefficient was attributed an unit value. The behaviour of the buildings was idealized as a shear-beam, with nonlinear characteristic modeled by Takeda type force-deformation loops; hence the structural variables are the storey shear forces and storey drifts, and the ED coefficients are defined for the storey shear forces. Thus, the yielding value of each storey shear force is obtained from the shear force given by a linear analysis multiplied by the correspondent ED coefficient.

For expediency reasons, however, the 6 design processes were defined in terms of distributions of horizontal static forces F_j^i . In process I the forces were assumed constant (F_j^I = const.), and in process IV the forces were assumed proportional to the height h_j of the correspondent storey ($F_j^{IV} = k\ h_j$). Processes II and III may be considered to lie between processes I and IV ($F_j^{II} = 0.67\ F_j^I + 0.33\ F_j^{IV}$; $F_j^{III} = 0.33\ F_j^I + 0.67\ F_j^{IV}$). Processes V and VI are caracterized by and inverted triangular distribution of horizontal forces but with an additional concentrated force as the top, with a value of about 3% and 7% of the total base shear, respectively. Hence, from process I to VI there is a continuous tendency to a more uniform distribution of shear force resistances along the height of the structure. The horizontal forces were normalized by the criterium that the total sum of the storey shear forces should be equal for all designs; hence the yielding value for the first storeys decreases as the order of the design process increases. The purpose of this normalization criterium is to make plausible that the cost of the differently designed structures is the same and consequently, the results can be comparable. Design process IV, for a peak acceleration of 50 cm/s^2, was taken as the reference.

For the design seismic action the Eurocode 8 5% response spectra was adopted and a sample of 10 motions with two independent horizontal components, with 10s duration each, was generated from a stationary stochastic process, such that the mean value spectrum would match teh Eurocode 8 response spectrum quantified for a peak velocity value of 10 cm/s. The acting earthquake motions were idealized by 4 types of earthquakes motions (A,B,C and D); each type was modeled by a nonstationary stochastic process and represented by a sample of 10 motions with two independent horizontal components. The characteristics of the nonstationary process were adapted to represent earthquakes in firm soil with magnitudes 6 (A), 7 (B), 8 (C) and 9(D) at such a focal distance that a peak ground velocity of 10 cm/s would result. In figure 1 are presented a component of each sample and in figure 2 are represented the sample statistics for the correspondent response spectra. The increase in low-frequency content with increasing magnitude is clearly apparent. The motion normalization by the peak velocity value was selected because this value is generally recognized as a good measure of earthquake severity.

For the shear-beam model of a building, the natural control variables are the ductility demand at the different storeys, but in this paper only the maximum ductility demand values will be considered. The correspondent design limit states are the available ductility of the structural elements. In table I are presented the sample average values of the maximum ductility demand for several combinations of design processes and earthquake actions. The average values of peak acceleration of each sample were scaled to 100 cm/s^2 and 200 cm/s^2 (2 and 4 times the design peak acceleration) and is indicated by the number following the letter representative of each earthquake action (EU represents the Eurocode 8 action). Several interesting conclusions may be drawn from this table, but it should be referred that the very high values that are obtained in some cases are due, in general, to a concentration of the nonlinear behaviour at the top storey of the building; the lowest values are associated to an almost uniform energy dissipation along the height.

Conclusions . Different design processes originate very different ductility demands. In most cases of interest, the increase in ductility demand is faster than the increase in peak acceleration; hence the ratio between behaviour coefficients of structures designed to different ductility levels should be smaller tha the ratio of their available ductilities. The use of a stationary model of the earthquake action for the design seismic action (in this case, the Eurocode 8) gives a fair estimate of the ductility demands expected from a wide range of ground motion characteristics; however long duration earthquakes appear to originate large ductility demands. Optimal design processes may be identified as those which conduce to a minimum value of the maximum ductility (for most earthquakes); but design processes near an optimal one are almost as good. The optimal processes are very similar to

Table I - Maximum ductility demand in frame buildings

EARTHQ. ACTION	2 STOREYS				4 STOREYS				8 STOREYS			
	I	II	III	IV	I	II	III	IV	I	IV	V	VI
EU100	5.8	5.0	2.2	2.6	2.8	1.6	1.3	1.2	23	8.9	3.6	2.4
EU200	18	13	10	12.0	11	7.2	3.5	1.7	33	14	8.9	8.1
A100	4.8	3.9	2.5	2.3	1.9	1.3	0.9	0.8	20	7.4	3.1	2.3
A200	12	10	6.5	7.1	7.4	7.4	1.5	1.7	30	16	8.9	7.7
B100	3.8	2.8	2.0	2.1	1.8	1.1	0.9	0.8	21	6.3	2.9	2.0
B200	11	9.1	5.8	7.5	6.6	4.0	1.7	1.7	36	17	7.5	6.0
C100	3.6	2.6	1.7	1.9	1.5	1.1	1.0	1.0	28	5.4	3.3	3.2
C200	12	11	5.8	7.9	7.6	4.8	1.9	1.9	45	16	9.7	9.3
D100	5.2	3.7	2.0	2.4	3.9	2.0	1.2	1.4	44	5.9	8.1	7.6
D200	31	25	23	26	17	12	3.6	4.8	64	17	20	25

Note: The roman numbers denote design processes

Table II - Optimal ED coefficients

No. of storeys	Optimal process	Storey no.							
		1	2	3	4	5	6	7	8
2	III	0.96	1.06						
4	III	0.93	1.00	1.05	1.09				
8	VI	1.03	1.03	1.02	1.01	1.00	0.98	0.93	0.82

the processes presented in recent earthquake resistant codes. The number of storeys has some influence in the optimal design process: for higher buildings, a more uniform distribution of shear resistances shall be provided. The ED coefficients for the optimal design processes are presented in table II; those values are not very different from the unity, hence a global ductility value can be used with confidence for regular buildings, as are those considered in this study; however, this conclusion cannot be extrapolated for irregular buildings (Ref. 9,10).

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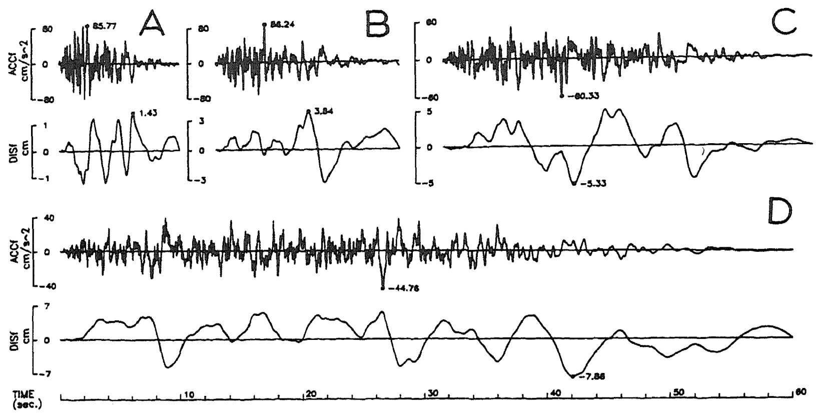

Fig. 1 - Typical acting earthquake motions

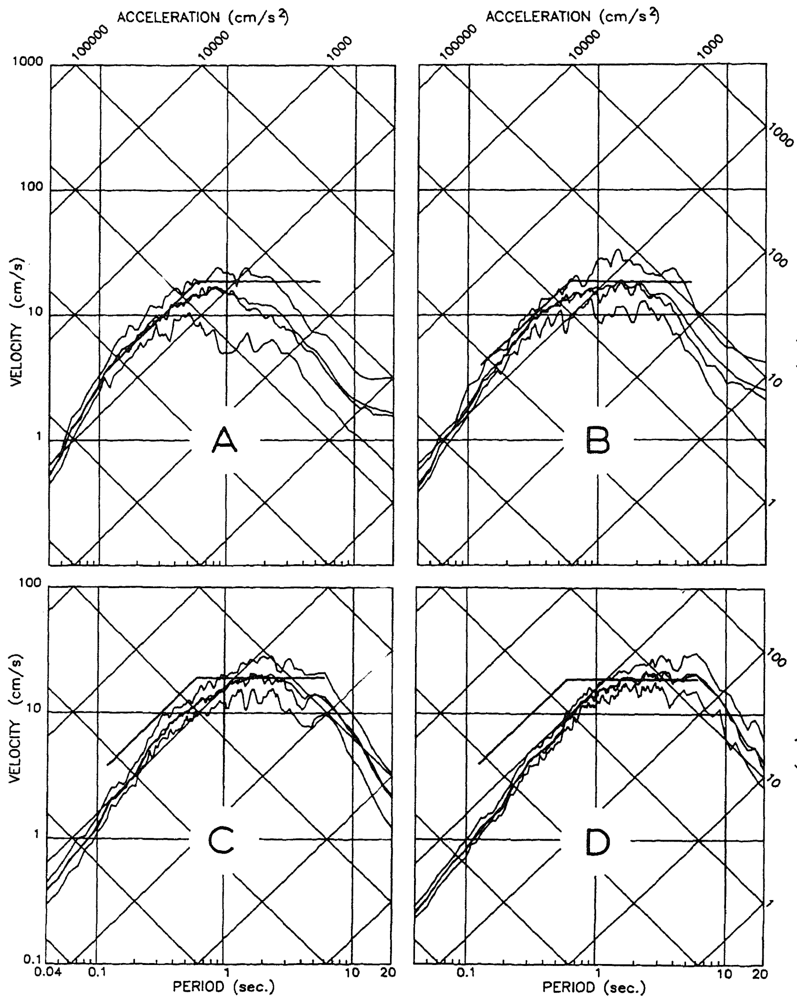

Fig. 2 - Statistics of the 5% response spectrum for the different types of acting earthquake motions (mean values and the 10%, 50% and 90% fractiles) compared with Eurocode 8 response spectrum (normalization factor: 10 cm/s peak velocity).