METHODOLOGY FOR THE DYNAMIC ANALYSIS OF BUILDING STRUCTURES

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SUMMARY

The safety checking of building structures against earthquake actions envolves some problems which are not generally present when checking against other actions. These problems mainly arise from the need to consider explicitly the energy dissipating behaviour of the structure. The methodology presented in this paper deals with the assessment of earthquake action effects for safety checking purposes and considers the levels at which dynamic analysis may be performed, the classification and modelling of structural systems, and the techniques to be used for the performance of dynamic analyses.

INTRODUCTION

The recent years have seen significative advances in Earthquake Engineering and in the art-science of Code Writing. When the updating of the 1961 Portuguese earthquake resistant code (Ref. 1) was considered, the incorporation of those advances presented some problems. In effect it was desired that the code was to follow the (perhaps rationalistic) format of the CEB-FIP Model Code (Ref. 2). Hence the Portuguese code is not only a code for earthquake protection, but deals with all types of actions that must be considered in the design of buildings. Moreover the code was also intended to be used by professionals with small to none proficiency in earthquake engineering. These two requirements were not very congruous to the modern earthquake design philosophy, where the designer, working only with lateral forces or linear dynamic analysis, is expected to limit the deflections of an actual nonlinear system while providing for the global and local ductility demands. Thus the elaboration of the new Portuguese code was a lengtly process that resulted in the version which was published as law on 31 st May 1983 (Ref. 3). The sucess achieved is attributable to an in-depth structuring of the knowledge and techniques available. This structuring involved the organization of the code provisions as a whole deductible from the "first principles" of Earthquake Engineering and the Theory of Structural Safety.

The elaboration of the Portuguese code mightily benefited from similarly directed activity at the international level, which produced notable advances of the subject (Ref. 4,5 and 6). However as the Portuguese code drafting committee had to pay heed to a smaller spread of opinions and to address a narrower scope of situations (almost limited to reinforced concrete structures for

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practically buildings only), the methodology that was elaborated as a background to the code provisions could be more developed than the ones that implicitly back up the comparable international documents. It seems thus worth while to present briefly the principal ideas behind that methodology.

PURPOSE OF THE METHODOLOGY

The design of an earthquake-resistant structure may be thought of as consisting of two phases. In the first phase the structural elements are chosen, proportioned and distributed by the different parts of the structure, in order to get a "good" earthquake behaviour, as can be assessed from a qualitative analysis of the intended structure. The influence of the building configuration and of the correct distribution of the structural elements on the earthquake behaviour of the building has increasingly been recognized (Ref. 7). However, in spite of its importance, the consideration of building configuration lies outside the scope of a methodology for analysis, mainly because at present it is not possible to quantify explicitly its influence in terms of independently defined variables. Thus the methodology of analysis has to deal with the influence of configuration in either a very accurate or a very crude way, i.e., either overleaping the question by resorting to so accurate an analysis that the influence of configuration is resolved into the mutual influence of the building elements, or eluding, in a certain sense, the question by aggregating the configuration influence together with other uncertainties into behaviour coeficients which, in most cases, do not explicitly depend on the configuration.

The second phase is basically an analysis phase. In effect, it is then necessary to verify if the strength and ductility of the structural elements are adequate for the internal forces and deformations that are imposed on them, by a set of action combinations which are deemed representative of the severity of the loads that will act on the building. Unlike what happens with the determination of the effects of other types of loading, which can be performed assuming that the structure behaves linearly, the earthquake action effects cannot be assessed whitout consideration of the complex non-linear energy dissipating behaviour in which lies the true capacity of a structure to overcome strong earthquake shaking. The simplifications that must be assumed in order that the design process shall not become too time-consuming or expensive, are so crude and far reaching that the processes of structural analysis used in checking safety are not to be discussed outside the scope of the safety verification itself. Thus, the uncertainties about the structural behaviour can be lumped together with the uncertainties about the safety verification process (assessment of the seismicity, choice of the risk level,...).

Reliability studies may be performed at three levels (Ref. 2). Level 3 is the "exact" reference level, where resistances (expressed as limit states) and actions are thoroughly probabilized; thus, at least in principle the probability of exceeding a limit state (probability of failure) may be computed. Level 2 comprises those design processes where a discretization of a level 3 formulation is worked out so that the probability of failure can be obtained from a first-order analysis. In level 1, actions and resistances are described by their characteristic values which are compared through design algorithms which bring into play partial factors of safety. Levels 3 and 2 are clearly not adequate for design purposes. At level 1, the verification of safety as re

gards earthquake actions is performed comparing the design value of the strength of the different structural elements with the corresponding design values of the earthquake action effect. This design value comprises the only the effects of the design earthquake action but also the effects of the other loads and actions likely to be present at the time of the earthquake. The design value of the earthquake action effect is obtained by the action combination rule (Ref.8):

$$S_{dE} = \sum_i \gamma_{gi}\, S(G_i) + \gamma_{qER}\, S(\gamma_{qEI}\, Q_E) + \gamma_{qE} \sum_{j \neq E} \psi_{Ej}\, S(Q_j)$$

where:

- S_{dE} - design value of the earthquake action effect.
- S_{Gi} - action effect of the i-th permanent action.
- S_{Qj} - action effect of the j-th variable action.
- $S(\gamma_{qEI} Q_E)$ - action effect of the design earthquake action.
- γ_{gi} - partial factor of safety for the i-th permanent action.
- γ_{qEI} - partial factor of safety for the earthquake action.
- γ_{qER} - partial factor of safety for the effect of the design earthquake action.
- γ_{qE} - partial factor of safety for the variable action other than the earthquake action.
- ψ_{Ej} - quotient between earthquake combination value of the j-th variable action (quasi-permanent value) and its characteristic value.

Note that Q_E is the earthquake action, $\gamma_{qEI}\, Q_E$ the design earthquake action, $S(\gamma_{qEI}\, Q_E)$ the action effect of the design earthquake action, and $\gamma_{qER}\, S(\gamma_{qEI}\, Q_E)$ receives no special designation because the design value of the earthquake action effect incorporates the effects of the other actions and γ_{qER} cannot be defined without reference to the combination rule.

As the earthquake effects are computed taking in account nonlinear behaviour the partial safety factor γ_{qE} for the variable actions (in the combination rule where the seismic action is the basic action) must be splitted in two components when applied to the seismic action itself: γ_{qEI}, which applies to the intensity of the action, and γ_{qER} which applies to the response of the structure.

The effects of the permanent and variable actions other than the seismic action are computed assuming linear behaviour. The effects of the earthquake action are computed considering the nonlinear hysteretical behaviour of the structure, loaded with the mass corresponding to the mean value of the as sampled distribution of the permanent and variable actions likely to be present when an earthquake occurs.

The rules for the computation of the action effect of the design earthquake action are the object of the methodology presented in this paper. These rules are not intended for any other use, for instance, to assess how a real building behaved during a past earthquake event. The main benefit to be expected from their application consists in the harmony between, on one side, the power and resolution of the analysis and, on the other side, the complexity of the structural behaviour, and the depth of the results that are needed. In effect, it is clear that is as unfit to use a very powerful analysis for very simple structures as to use a crude analysis for complex cases.

Since safety checking at level 1 is an approximate process, the methods for computing the earthquake effects should likewise be also approximate methods. The recourse to exact methods for determining the seismic action effects are only justified, in principle, for the assessment of the approximate methods and the calibration of the partial safety factors.

THE LEVELS OF ANALYSIS

The approximate methods for analysis can be classified into four levels according to the assumptions regarding the structural behaviour that are made (Ref. 8). From these assumptions, simplified computation procedures will follow.

The first level[(I)] of analysis is the exact method level, that is, no simplifying hypotheses are assumed in the analysis. Thus, step-by-step methods for the integration of the equation of motion must be resorted to, in order to reproduce the nonlinear behaviour of the structure; further, several samples of the earthquake action must be considered. In consequence, analyses become so expensive and time-consuming that this level can only be envisaged as a reference for assessing the validity and adequacy of the simplifying assumptions of the other levels.

The second level of analysis is defined by the adoption of procedures with which an estimate of the nonlinear behaviour of the structures may be obtained. Those procedures consist, typically, in correcting the results of a linear dynamic analysis with the help of behaviour factors. Those factors are supposed to transform the values that were obtained in the linear dynamic analysis into the values that would be obtained if a nonlinear analysis were performed. At least two behaviour factors have to be defined: one for displacements, deformations and strains and one for forces, internal forces and stresses; in principle there will be a class of deformation behaviour factors and a class of force behaviour factors. Those factors obviously depend on the configuration of the building, on the type of the structural system, its dynamic characteristics, the ratio of the structure "yield" strength to the severity of the design earthquake action and on the "design process" (Ref. 9 and 10). The inaccuracies involved in this approach are such as to need no further emphasis. In order to diminish these inaccuracies it is necessary to go for ever finer classifications of the characteristics upon which the behaviour factors depend. For instance in the Portuguese code (Ref. 3) buildings are first classified into regular and irregular buildings according to their configuration and structure, then they are divided in conformity to their structural system: in a first phase as regards the geometrical disposition and relative stiffness of the structural elements, in a second phase as regards the ductility capacity of their structural elements; lastly, it is recommended (in a comment) to consider if the disposition of the "nonstructural" elements, like infill walls of brick masonry, should not entail a modification of the behaviour factors.

The third level of analysis is characterized by the possibility of analizing only the behaviour of the structure along some appropriate directions

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(I) Ordinal numbers are used for the levels of structural analysis while cordinal numbers are used for the levels of the safety checking process.

(in general, the longitudinal and transversal directions) and taking in account the torsion and other spatial phenomena with the help of some simplified rules of an empirical character.

The fourth level of analysis comprises all procedures which do not include a dynamic analysis as such. Those procedures consist in general in the determination of the internal forces and deformations of the structure under the action of a set (or sets) of horizontal forces, whose intensity and distribution are quantified from the severity of the earthquake action and from a crude estimate of the fundamental frequencies of the structure.

The progression of the simplifiyng hypothesises adopted in this classification is clearly not the only one possible or desirable; it suffices to remember the most valuable knowledge gained from the many nonlinear analyses performed with in-plane models of buildings. However, for safety checking purposes, at level 1, such classification is thought to be the more reasonable .

STRUCTURAL SYSTEMS

For analysis purposes a structure can be divided into structural systems and secondary structural elements. The classification of a structural element as part of a structural system or as a secondary structural element depends on the aim of the analysis. For example, if the effects of horizontal actions are to be studied in a framed structure with slab panels, the structural systems are the frames themselves, whereas the slabs may be considered as secondary structural elements; on the other hand, if gravity loads are considered it is obvious that the floor structures, which do include the slab panels, have to be taken as structural systems.

The domain of validity of the simplifying assumptions (namely for the passage from the first to the second level, which entails the definitionofthe behaviour factors) can only be defined if the structural systems are accurately identified booth in terms of the relative sizing of their structural elements, and in terms of their behaviour under earthquake actions. This identification was developed in the Portuguese code only for building structures, it was not found possible to develop an equivalent identification for bridges or other type of structures for which behaviour factors must be atributed, of necessity, from the consideration of only their behaviour under seismic actions.

Since the structural tube system is not used in Portugal, all structural systems may be classified as plane systems, or better, as tichoidal systems (from the Greek: wall like). Tichoidal systems are defined as systems whose structural elements are disposed in the neighborhood of a vertical plane and with a stiffness, relative to horizontal forces, much greater in the direction of such vertical plane than in the orthogonal direction; moreover, moderate displacements in this latter direction should not cause significative stresses in the system structural elements. This classification takes in account first the geometrical form and relative sizing of the structural elements (i.e., the stiffness properties of the structure), and then the relative strength of the structural elements and, for a given structural element , the relationship between its strength for different internal loads, i.e., the type of energy dissipating mechanism that is developed when the elastic limit is exceeded. It should be remarked that the values for the relative limit elastic strength have to be defined, whereas post-elastic ductility, for the moment, is assumed to be unbounded.

used) with or without a static condensation of all the degrees-of-freedom which are dependent on the storey's three degrees of freedom, or from the compatibilization, at storey levels, of plane models of tichoidal structures. The 3DFPS model is mandatory for the first and second levels of structural analysis.

The third level of structural analysis is associated with structural models which are an aggregate of the plane models of the tichoidal structures whose vertical planes are parallel. The use of these one-degree-of-freedom per storey (1DFPS) models is admissible when the building configuration is nearly symmetric and the structural elements are disposed in a regular pattern (for instance, in an orthogonal or an equilateral triangular pattern). The 1DFPS model is at disavantage with the 3DFPS model as regards the consideration of torsion effects and the possibility to deal with non-patterned structures which are not amenable to plane analysis.

In engineering terms the use of the 3DFPS and 1DFPS models involves different constraints. In effect, while the 1DFPS model may be analysed under further simplifying assumptions, up to where only a desk calculator is needed, the 3DFPS model nearly always will need a computer. The increasing availability and capacity of the minicomputers, however, makes previsible that, if software is readily available, increasingly more structures will be analyzed by the 3DFPS model.

DYNAMIC ANALYSIS

The use of the different techniques of dynamic analysis is, of course , governed by the requirements of the different levels of structural analysis . It is worthwhile, however, to highlight some aspects, of a technical nature , which, if ignored, may defile the analysis.

At the first level, step-by-step integration of the nonlinear equations of motion is used. The problem here is about the choice of the time-series of acceleration to be used in the analysis. As the response of the structure, in the nonlinear range, is very sensitive to the details of the ground motion , several time-series should be used. Thus it is necessary to think how the value of the action effects is to be extracted from the several responses. Assuming that the maximum value of the response to an earthquake motion characterizes adequately (i.e. for design purposes) the response, one can define the value of the action effect as the average of the maximum values (the highest of the maximum values would not be appropriate because inter alia that value increases with the number of time-series used). Therefore to perform analysis at the first level it is necessary to have an ensemble of ground motions fit to support the definition of averages. Ensembles with this property are stochastic processes if they have a minimum of mathematical structure. The case for the use of stochastic processes as a model for earthquake actions has been often argued (Ref. 11, 12, and 13). Stochastic processes are also very useful at the second level of analysis, because they make it possible to compute the maximum value of the response, in the linear range, directly without reference to the maximum values of the response of each mode of vibration, contrary to what happens when the more usual response spectra representation of the ground motion is used. Thus is sidestepped the need for combination rules of the maximum response of the modes, which are not simple because at this level we are dealing with 3DFPS models which present nearly always very close natu-

ral frequencies. At the third level of analysis natural frequencies are never close and the spatial variability of earthquake shaking seldom must be taken into account. Hence, the advantages of the stochastic process idealizations of ground motion over the response spectrum idealization are small, and the latter may be used at will.

COMPATIBILITY BETWEEN DYNAMIC AND STATIC ANALYSES

A last subject of consideration is the compatibility of the results of the dynamic analysis with those of the static analyses needed to obtain the values of the other actions needed in the combination rule for safety checking. In effect, the results of a dynamic analysis may be expressed as the maximum value of the deformation between storeys or of the internal forces in each structural element. When the earthquake action effects are expressed as the maximum value of the deformation between storeys (or equivalently as an envelope of shear forces or as a set of horizontal forces) it is necessary to transform the storey deformations into internal forces in each structural element, taking in account that this is a nonlinear transformation (unless behaviour factors are already calibrated to operate on the internal forces obtained from a linear transformation of the maximum storey deformations). Another aspect to consider is that the combination rule for safety checking is mainly appropriate to check strength, not ductility. Thus, ductility, at the level of the structural elements, must in general be checked in terms of available ductility, as provided by the detailing procedures followed in the design of the energy-dissipating zones of the structure, and also of required ductility. The values of the required ductility are estimated on the basis of the energy-dissipating mechanism that underlies the classification of the structural system and controls the values of the behaviour factors. In some cases the values of the behaviour factors will be bounded by the stability of the energy-dissipating mechanism; in other cases they will be bounded by the maximum value of ductility that can be made available at the structural element level.

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