

Earthquake resistant design: Research developments in basic and in normative research

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ABSTRACT: Harmonization of the norms at a supranational level calls for more visible and demonstrable bases. This poses a new challenge to applied research, which is called upon to provide hard arguments pro or against traditional choices, and to improve on several aspects entering in the design process. This process is seen as a system with strong interactions among its elements, which makes real progress to be dependent on the coordination of separate research efforts. The aspects thought to be more critical are reviewed, and an assessment is made of their present state of advancement with respect to code necessities, and of the prospects for further developments.

1 INTRODUCTION

The paper is the reflection of a line of activity pursued by the author for more than a decade, by trying to maintain a link between new research findings and the progress of the technical norms. In spite of two psychological obstacles, i.e., the consciousness that no matter how much work is spent on it the result will never be perfect, coupled with the anticipation that it will more frequently meet with attitudes of refusal than of reasoned appreciation, the activity retains a unique combination of attractive aspects.

From the side of the research, the obligation of keeping oneself abreast with the pertinent developments in a world scale and in a large spectrum is an exercise favouring maturity of judgement and ability to discern; further, it gives the perspective for promoting research efforts in the areas known to represent weaker links in the spectrum and, finally, provides reliable inspiration for personal research work.

From the side of the codes, it is well known that structural design codes do not aim at portraying the physical reality per sé: they are logical systems which operate through a rational use of possibly rough and even conventional models with the end purpose of producing designs as uniformly reliable as possible. The decision on whether to introduce a new research result in the system must therefore be based on the possible benefits related to its ultimate purpose, and this utility-based type of decision may not be always immediately obvious.

The development of a critical attitude in evaluating the actual relevance of new results, and of a faculty of synthesis for combining them into the complex whole, comes along quite naturally with the responsibility of code making.

In the following, the author offers his views on the

present state and on the perspectives of progress on a selected number of topics which are today of central interest for code-minded researchers.

2 MODELS FOR NONLINEAR ANALYSIS

Excellent S.o.A. reports on this subject are already available (Fardis, 1991), or in preparation (Filippou, 1993), and the reader is referred to these sources for a thorough treatment of both the present state and its historical development; a moderate amount of information and familiarity with the subject is sufficient, however, for the purposes of the following discussion.

One aspect of general nature and of great consequence that might be considered in the first place is whether a widespread use of nonlinear methods of analysis for ordinary seismic design is to be foreseen, less or more imminently but inevitably, and whether steps should be taken in the norms to encourage this development. Such a question is not one that can receive a quick and straightforward answer.

Starting with the timing, it is sure that it cannot be for tomorrow, it for no other reasons because of the fact that nonlinear analysis programs generally available to civil engineers are too poor for the purpose. Secondly, there are serious educational problems, since both the basic theory behind, together with the fine details of its implementation, and the practical handling of nonlinear programs require a scope of knowledge well beyond the present graduate level.

Of no lesser importance, however, is the consideration of the changes that the use of nonlinear methods of analysis would necessarily bring about in the entire design process. In brief, nonlinear analyses yield as output a much larger and more articulated set of quantities than linear elastic ones do, and objective and

codifiable procedures for judging the acceptability of a given design based on these data have not been elaborated so far.

Elastic analysis is, especially for seismic design, recognisedly conventional, as it is conventional to make demand-capacity comparisons on an element-by-element basis and in terms of stresses or stress resultants. But this conventional procedure has been calibrated over the years to produce results which are acceptable in terms of safety and economy for a large class of structures, so that it has now acquired a well defensible status. Replacing this procedure by another one, more realistic and far reaching, based on more pertinent quantities like maximum or accumulated strains, degradation of strength and of stiffness, etc., at both a local and global level, will undoubtedly require a rather long time for maturation. And this leads back to the question raised initially: it would make little sense to push for a generalized use on nonlinear analyses until a corresponding conceptual framework for utilizing their results will be well on the way of being established.

But to recognize that nonlinear analyses are not yet ready for use in codified design (and viceversa) still leaves the question of their role and importance totally unprejudiced, and not only for important and special structures, but also for the ordinary buildings which constitute the subject of the present discussion.

The progress of seismic design, in terms of increased rationality of the procedures with the consequent improved predictability and uniformity of the performances, is conditioned to a corresponding increase of our ability to analyze accurately the response of different typologies, and of different layouts within each typology, and to relate the analyzed responses to the criteria and methods used in the design.

Incidentally, a demand for the infusion of more rationality in the norms is a reality deriving in good part from the international movement towards harmonized supranational codes: harmonization can more easily be obtained by having recourse to rational arguments, rather than to those of tradition.

On the other hand, a closer relation between response behaviour and design choices cannot reasonably be expected at the necessary scale from the experimental side, because of its obvious inherent limitations of scope: we are thus led to the conclusion that in the future actual physical tests will more and more serve as benchmarks for the verification of the analytical models, which will be the real working tools for the job of improving the procedures and their calibration.

Be it due principally to the international drive for harmonized codes or to other concurrent factors, like the occurrence of disastrous events which have shown the inadequacy of the existing methods of analysis, or the need of assessing and upgrading the safety of structures built to past inadequate standards, it is however a source of comfort to note that work on analytical models has gained a remarkable impetus in the last period, with good prospects for rapid

substantial improvements over the situation until now.

A classification of the numerous types of models is in order, if a few comments are to be made on those considered as more useful or promising.

With reference to models representing the behaviour of entire elements of frames, the simplest scheme has no less than three entries: a) *the type of behaviour covered*: simple bending, bending with axial force, biaxial bending with or without axial force, bending with or without axial force and shear, etc.; b) *the type of approach for describing nonlinearity*: phenomenological or fundamental; c) *the type of discretization used*: inelasticity lumped at the element ends, inelasticity spread and averaged over portions of the elements, discretization extended over the full length of the element.

For a given type of behaviour, almost all the possible combination of entries b) and c) can be found in the literature, although the phenomenological approach has difficulties in incorporating with sufficient accuracy the variation in time of the axial force and, also, it cannot practically tackle at all the combination of biaxial flexure with axial force.

One might wonder about the need for simpler models once the more comprehensive ones are available: the answer is that the latter have far greater computational requirements, so that their use is only warranted when they are really needed. Typically, beams in frames are modeled by using pure flexural elements, while for columns it all depends on whether the variation of the axial force due to the lateral excitation is significant or not. In the positive case, accounting for this effect may become essential, since the variation of the axial force affects considerably both the stiffness and the strength, and hence the ductility demand on the elements.

The category of purely flexural elements is the one having received the greatest attention so far, and the number of the variants proposed is almost uncountable. To simplify, three broad classes are worth being discussed, either because they are still in prevalent use, or because they will likely supplant the others some time in the future.

The first class comprises the earliest models and is characterized by the two following basic choices: all inelastic behaviour is "lumped" at the elements ends, in the form of concentrated springs serially connected with the linear elastic beam element; the moment-rotation relationship of the springs is described by means of phenomenological models characterized by sets of parameters and rules varying in number and in complexity.

Although the empirical nature of the hysteretic models of the springs may not suit all tastes, their merits cannot be discounted, since during the years these models have been perfected to the point of reproducing with great realism even minor features of the experimentally observed behaviour, some of which, as for example the effect of the spalling of the concrete cover, the loss of strength due to the progressive disintegration of concrete, the effect of bars slip, etc.,

cannot be represented at all by many of the so-called 'physical' or 'fundamental' models.

To prove the usefulness of the hysteretic models stays the uninterrupted flow of new proposals, started in 1965 by Clough (Clough et al., 1965) whose well known simple model is still the workhorse in the codes of widest use, and continuing to the present time, see for ex. the recent development of earlier models presented in 1992 by Coelho (Coelho, 1992), which can describe pinching as well as stiffness and strength degradation.

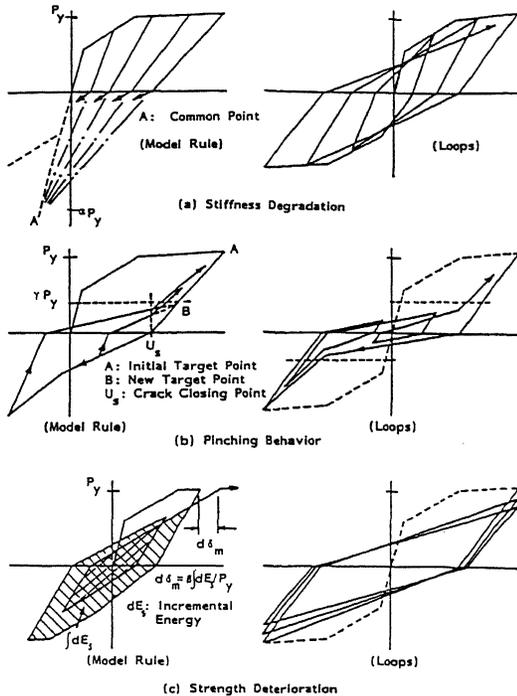


Fig. 1 - Three-parameters hysteretic model by Park et al. (1987)

As an example of the versatility combined with relative simplicity achieved by the recent models, a brief mention will be given of the one developed by Park et al. (1987), which has a tri-linear skeleton and three parameters for governing the hysteresis, as shown in fig. 1.

Stiffness degradation, represented by α , is accounted for with the introduction of the point αP_y lying on the extrapolated first loading branch, so that all unloading branches tend toward it until the load reverses its sign.

Pinching effect is described through the introduction of an horizontal threshold at the level γP_y . Reloading lines now tend toward the intersection of this threshold with the previous unloading line (point B in fig. 1) until they reach the crack-closing point (point U_s in fig. 1), after which they tend to the previously attained maximum point.

Strength degradation is described by the parameter β , which gives the extra displacement needed to reach the

previous maximum load level and is taken as a function of the (normalized) total dissipated energy.

Suitable combinations of the parameters enable the simulation of several existing simpler models, as for ex. the Clough, the Takeda, the origin-oriented models, etc.

Lumped inelasticity models have, however, a basic fault, as it is lucidly discussed by Filippou et al. (1988), whose consequences may be more or less relevant depending on the pattern of the bending moments and on its variation in the course of the response. The fault does not lie with the hysteretic rules (which can be effectively utilized as sectional properties in the distributed inelasticity models), but in the "lumping" itself.

Lumping is in fact the determination of the parameters of the end springs as function of the inelastic component of the stress state along the element. This relationship is unique for a given state (for example, when all the sections included in a fixed portion are on their loading branches with equal stiffnesses), but since the state changes continuously during the loading history the lumping should vary accordingly, a fact that would destroy the usefulness of the criterion. The errors incurred by the lumped plasticity models when the pattern of bending moments is different from the postulated one can be very large, a shortcoming that greatly reduces the generality of use of these models.

Midway between the lumped and the continuously distributed plasticity there is a second class of models, usually denominated as spread plasticity. Here too, a sort of lumping is retained, in the sense that all inelastic effects are taken care of by a separate element, having a rigid central portion and two lateral zones of finite length called plastic zones. This element is then combined with a fully elastic beam element to give the total behaviour.

Such is for ex. the model proposed by Filippou and Issa (1988), whose essential features are given below. At each side of the actual beam only two sections, which delimit the width of the plastic zone, are monitored at each step of the analysis: the one at beam-column interface and the one where the value of the bending moment equals the yield one. Since the position of the second section varies during the response history the plastic zones have variable lengths. The stress state within, and hence the behaviour of each plastic zone is assumed to be controlled by the end section only, with the following three cases being possible: a) the end section is in a strain hardening phase: the entire zone is assumed to be in the same phase and its stiffness is the post-yielding stiffness of the moment-curvature relation; b) the end section is unloading: the entire zone is assumed to be in the same state and its stiffness takes on an infinite value, since the elastic deformation at unloading is already reproduced by the elastic element; c) the end section is reloading: it is assumed that the stiffnesses of the zone is the average of the stiffness at the two ends of the zone. Based on the previous assumptions, a 2x2 full flexibility matrix can be established for the "spread

plastic subelement", to be superimposed to the ordinary 2x2 matrix of the elastic beam element, and then inverted to give the tangent stiffness matrix for the step.

The model outlined has proven to be quite accurate within its scope of applicability, and computationally very efficient.

The third and last broad class of models needs not to be much elaborated upon: it should appear as the natural choice for anyone accustomed to finite-element approaches for structural analysis. The actual element is discretized into a number of segments, and the resulting internal sections are monitored at each step for the purpose of deriving their tangent flexibilities, these latter being in turn generalized to yield the nodal flexibilities. The sectional behaviour can indifferently be described by a phenomenological law, by more or less idealized models based on variants of plasticity theory or, as it is more often the case, they can be constructed from first principles by subdividing the sections into fibers and using appropriate constitutive laws for concrete and steel. These last so-called 'fiber' models are almost as old as those of the first two classes, which have won the field due to their computational superiority.

The balance, however, is likely to change progressively in favour of the more rational models. To speak for this change there is on one hand the drastic increase in efficiency reached in the latest proposals, and on the other hand the fact that phenomenological models lose very rapidly simplicity with increasing completeness of the state of stress to be covered.

A cursory review of the models available for elements with axial force, in uniaxial or biaxial bending, suffices for recognizing the seriousness of the situation. It is serious because the presence of a variable axial load in the columns of frames represents the ordinary case, not the exception, so that the limitations one has in reproducing this effects define also the limits one has on the overall capacity to analyze frames. By far the majority of the codes of general availability have in their libraries of beam-column elements the lumped inelasticity models only, which at best are unreliable for use in columns of frames.

Ruling out this class of models, one is left with a relatively small group of spread plasticity models. As regards the sectional behaviour, two basic alternatives exist. The first one consists in starting from phenomenological moment-curvature relationships relative to a discrete set of axial load values or, better, from analytically derived functions: $M = M(\epsilon_o, \varphi)$ $N = N(\epsilon_o, \varphi)$ (ϵ_o = axial strain), and then in using these curves/functions to evaluate the partial derivatives:

$$\frac{\partial M}{\partial \epsilon_o}, \frac{\partial M}{\partial \varphi}, \frac{\partial N}{\partial \epsilon_o}, \frac{\partial N}{\partial \varphi}$$

These partial derivatives are the coefficients of the sectional tangent stiffness matrix. As it can be seen, this approach is only practicable for uniaxial bending. Along these, or even further simplified lines are for example the models proposed by Takayanagi-Schnobrich (1976) and Coelho (1992).

The second alternative for deriving the cross sectional properties is to make reference to plasticity theory. Classical strain-hardening plasticity in the 3D space of M_1, M_2, N has been used by Tseng-Penzien (1975), and various versions of the Multi-Surface Plasticity theory, incorporating ad hoc rules for reproducing the stiffness degradation at loading, and the decrease of the moduli during unloading, have also been developed, see for ex. Chen-Powell (1982). While the former approach is too inaccurate in the description of hysteresis, the second has proven to be able to reproduce the experimental observations, but at the expense of considerable complexity: it appears that no further work is being spent presently to refine these models.

More promising, because of its greater flexibility in reproducing the sectional behaviour along general load histories is the Bounding Surface type of Plasticity adopted recently by Sfakianakis-Fardis (1991). The B.S. is defined in the space of the nondimensional stress resultants: μ_1, μ_2, ν , as the locus of the points representing combinations of peak values of μ_1, μ_2 for given values for ν (the model cannot handle strain softening), and is described by means of analytical expression which are of general applicability for the usual types of reinforced concrete rectangular sections.

The plastic components of the incremental deformation vector are essentially dependent on the two plastic flexural and on the plastic axial moduli, and in turn these moduli depend, as it is usual in the B.S. approach, from the distances of the current stress point to the surface, measured along appropriate directions. A not negligible amount of guesswork and curve-fitting is necessary to find the proper analytical expressions and the values of the parameters, but finally the model is calibrated for being applicable to a broad range of practical cases. The B.S. sectional model has been already utilized by the same authors to implement a distributed plasticity member model, in which the sectional flexibilities are integrated numerically at a discrete number of internal stations to obtain the element flexibility matrix. Further, the distributed plasticity member has been introduced in the shell-code ANSR, and cases have been run in 3D for up to three-storey buildings.

We have thus entered the category of distributed inelasticity models for uniaxial or biaxial bending with axial force, also called fiber or filament models when the analysis the sections is made by discretizing them into full-width bands or rectangular grids, respectively. From the limitations which have been pointed out concerning the phenomenological and classical plasticity models, this category emerges as the only one to have the right credentials for fulfilling the role of numerical simulators of physical tests.

The recent past has seen an accelerated progress in this field; most notably with the appearance of the works of Zeris (1986), Vaz (1989), of the above-mentioned work of Sfakianakis-Fardis (1991) and Filippou et al. (1992). But this progress should not be

mistaken for final achievement. Much in fact remains to be done in improving these models under several respects: reliability, ease of use and computational efficiency among the firsts, and time will pass before they will be routinely used for extensive studies in support of the normative procedures. All in all, however, the situation in this basic field is so dramatically improved in so short a period of time that a note of optimism should be permitted.

Before leaving this subject it is, in conclusion, *mandatory to make at least a mention to a related area* whose importance is growing fast and is taking the worlds of the research and of the profession almost unprepared to react adequately. It is also related to modeling and analysis, but its concern are the existing, and possibly faulty, structures, not the ones designed according to modern or less modern seismic codes. Yet, rational and economical decisions for repairing and/or upgrading these structures cannot do without an analytical assessment of their capacity. The number of different situations to be found in practice is very large, but typically the greatest (or the most difficult to overcome) deficiencies are in the detailing of the reinforcement, e.g. insufficient confinement to prevent bars from buckling, insufficient anchorage of bars, inadequate lap splicing, etc., and on the shear capacity, which might be less than, or comparable to, the flexural one.

All of the previously reviewed models do not incorporate any of these effects, and this is justified since these phenomena are simply ruled out by the provisions of the modern codes. On the other hand, it would be either unfeasible or out of proportion to develop necessarily complex accurate models, as it could be done, for ex. by using sophisticated nonlinear F.E. models. Rather, simplified mechanical models for the basic mechanisms, ad hoc experiments on specimens which reproduce the actual defects, and much ingenuity in the development of behaviour and capacity models for the members, appears to be the mix which has enabled the first pioneering successes to be achieved. A detailed account of some models developed for the assessment of existing concrete bridges is given by Priestley (1991). Two examples taken from this reference are used here to illustrate the nature of the problem.

Figure 2 refers to a column element whose reinforcement at the plastic hinge section is in different anchorage and confinement conditions. Line 1 is the reference case of a satisfactory situation; the difference between lines 2 and 1 is due to a lower confinement of the former, which anticipates the crushing of concrete, and does not allow for the development of steel strain-hardening (the points marking the start of a decay at the extremities of lines 1 and 2 define the values of the ductility beyond which the crushing of concrete core and the buckling of bars proceed rapidly). Lines 3 and 4 refer to lap splicing of the reinforcement, without and with confining hoops, respectively. When hoops are absent the bars can develop only a fraction of their

capacity (line 3) and beyond a maximum value the moment degrades towards a value which is based on the assumption that the bars are sliding with the sole restraint of friction. Line 4 represents a situation in which the bars are allowed to reach yield, but the confinement is too low for permitting large ductilities, and after a certain value of μ the bars start sliding.

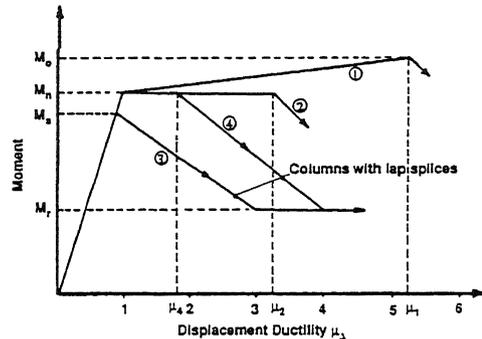


Fig. 2 - Effect of anchorage conditions on available strength and ductility (Priestley, 1991)

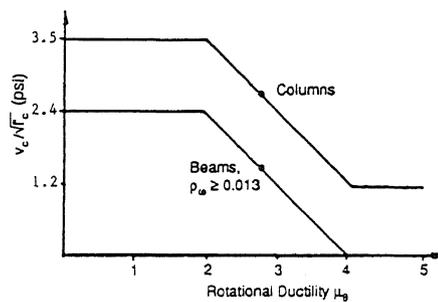


Fig. 3 - Reduction of shear strength with increasing ductility (Priestley, 1991)

Figure 3 refers to the contribution of concrete to shear strength, in beams and in columns, as a function of the ductility required to the element. The flat portions give the suggested values for low ductility levels, while for $\mu \geq 4$ the contribution is indicated as zero, because of the loss of aggregate interlock when crack width becomes too large. An important consequence of the rapid degradation of the shear strength with increasing ductility is that the shear capacity, even if initially larger than the flexural one, may become lower in the course of the cyclic loading, thus precipitating a brittle type of failure.

3 MEASURES OF DAMAGE

The accuracy and physical realism of the models for nonlinear analyses justify themselves only if the response quantities they yield are amenable to an

objective interpretation. The uniformity of this interpretation, also, is an essential precondition when calibration studies of code provisions are undertaken. And, finally, the interpretation must run as much as possible along physical lines, if a relationship between experimental results and analytical simulations is to be established. From the viewpoint of safety (limit-states other than the ultimate are also of interest, but will not be discussed here), the fundamental question is at what point a structural element and a whole structural system have to be considered as failed.

The answer is made difficult by the fact that the response variables used, be they stress-resultants, curvatures, strains, displacements, etc., refer to entities, like sections and elements, supposed to continue to exist until collapse occurs, while actually this latter consists in a process of progressive destruction involving buckling of bars, crushing of concrete and its expulsion, shearing along cracks, etc.

In front of this basic limitation, the best thing one can hope is to be able to find a *set* of response variables, or of quantities obtainable therefrom, that can be assumed as *indicators* of the build-up of a situation of collapse. The build-up is termed as *damage*, and the quantities as *damage indicators*; these latter are then combined into an empirically fitted expression to yield a *damage index*: D . By definition, $D = 1$ at collapse. The damage indexes proposed thus far in the literature do not contain more than two damage variables.

The purpose of this section is essentially that of stressing the comparative importance in the global spectrum of earthquake engineering of having a sound and agreed-upon definition of collapse; rather than attempting a literature review, which would result as very voluminous, a brief assessment of the present situation is given. For the interested reader, reference is made to the following well known papers and reviews of Banon et al. (1982), Park et al. (1984) and Chung et al. (1987), adding that several fresher contributions exist and continue to appear, although they can be considered more as elaborations on the theme than as new motives. To help in the discussion, the usual distinction between local (element) and global (structure) damage indexes is introduced. There are four response parameters which are most commonly used to construct local damage indexes, two of them representing extremal and two accumulated quantities.

The indexes based on extreme quantities are: a) the curvature or rotation ductility ratio: $D_\phi = \phi_{\max}/\phi_u$, b) the stiffness deterioration ratio: $D_K = 1 - K_{\min}/K_0$ (with K_{\min} representing the secant stiffness at the peak deformation and K_0 the initial elastic one), while the indexes based on accumulated quantities are: c) the accumulated plastic rotation: $D_p = \sum |\theta_p|/\theta_y$, and d) the normalized dissipated energy: $D_E = \int dE/0,5 M_y \cdot \phi_y$.

It can be observed that D_ϕ and D_K , as well as D_p and D_E describe essentially the same phenomena: statistical analyses in fact show that the correlations between the two indexes in each couple are close to unity, so that the

selection of a particular index from each class is practically a matter of preference. A combination of peak ductility and dissipated hysteretic energy is the prevailing choice, as in Banon et al. (1982), Park et al. (1984) and in more recent proposals also. The well-known proposal by Park writes as:

$$D = \delta_{\max}/\delta_u + \beta \cdot \int dE/P_y \cdot \delta_y$$

where δ_{\max} , δ_u , δ_y represent the maximum experienced, the ultimate and the yield displacement, respectively, and β is the strength deterioration parameter already presented in Section 2. For this latter, an empirical expression has been obtained from regression analysis of about 260 beam and column cyclic test data, relating β to the normalized axial stress (positive correlation with β), longitudinal reinforcement ratio (positive weak correlation) and confining reinforcement ratio (negative correlation). The simple and double parameters indexes above are characterized by coefficients of variation in the range of 0,25-0,45, a dispersion that should be regarded as relatively narrow, when considered in conjunction with the approximation errors of the analytical methods from which they are obtained, and with the variability intrinsic to seismic input: neither of these two factors has a lesser degree of dispersion.

In conclusion, the situation concerning the diagnosis of the state of single elements could be considered as sufficiently advanced to meet practical needs. Difficulties arise, however, in extending the use of the local indices to obtain a measure of proximity to collapse for parts or for the whole of a structure. Only when storey mechanism are likely to form meaningful indexes can easily be established, in the form, of a sum of weighted element indexes:

$$D = \sum_i E_i D_i / \sum_i E_i$$

the weight being the energy that each element of the story can absorb, or putting a limit to the interstorey drift:

$$D = |\Delta_i|_{\max} / H \leq D_0$$

where D_0 is usually taken in the range of 2+3%.

When the structure is designed such that plastic hinges distribute more or less uniformly, no fully satisfactory criterion to judge its state has been found to date. The current attempts rotate around the concepts of global ductility (i.e., referred to the F- δ relationship for the entire structure), or of global stiffness deterioration (to be checked on the same F- δ curve), concepts that have been noted to be strictly correlated. Their implementation, however, is both analytically cumbersome and conceptually questionable, because of the generally loose link between static and dynamic response. Yet the lack of valid solutions is a serious problem, since the determination of the limit-state of collapse is exactly what is required for the calibration of the design forces, and the adoption of different criteria may well lead to estimates which are one order of magnitude apart.

This factor overshadows presently any gain in accuracy achieved in the methods of analysis and, at least from a conceptual point of view, could to some

extent call into question the very credibility of the code: it is a challenge worth to be faced with great expenditure of research efforts.

4 MAIN FACTORS IN THE DESIGN PROCESS

Fig. 4 collects the major variables entering in the seismic design process, where the term variable is used in the wide meaning of a non parametric entity defined in a proper space. They have been reduced to the

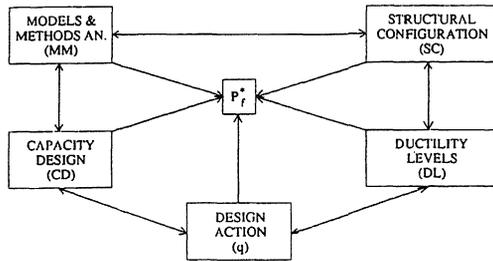


Fig. 4 - Components of the seismic design process

following five: *the structural configuration*: SC, which includes structural type and morphology, *the mechanical models and the methods of analysis*: MM, for which the codes normally foresee a variety of allowable choices, *the ductility level*: DL, that it is feasible and intended to achieve, *the capacity design procedures*: CD, whose extent also varies with the particular choices made for the other variables and, finally, *the design action*: q, here for the sake of extreme simplification identified with the value of the force reduction factor or, as it is commonly denominated in Europe, the "behaviour factor". This latter is actually a dependent variable in the sense that the other four variables are normally fixed first and then q is uniquely defined by the requirement of providing the design with the implicitly specified target probability

of failure: \bar{P}_f .

The purpose of the diagram is on one hand that of visualizing the interdependence of the five variables, which could be expressed by a symbolic equation of the form:

$$q = f(SC, MM, DL, CD, \bar{P}_f)$$

and to permit at the same time a separate examination of some of them.

4.1 Structural configuration

Although the importance of structural regularity in seismic design has been recognized from longtime now, the early papers on lateral-torsional coupling (as the problem is rather restrictively referred to in the U.S.

literature) having started to appear already at the beginning of the seventies, it is not until very recently that numerical investigations of sufficient width of scope and degree of realism have been produced. This situation of relative lack of background and of scarcity of conclusions of general validity is due to the compound effect of two causes: the onerosity of conducting non linear dynamic analyses on realistic models of buildings with the accompanying effort of interpreting the results of the analyses, and the extreme variety of possible types of "non regularity", which makes the selection of a reasonable number of significant parameters a very challenging task.

Not to mention the difficulty of relating the combination of the values of the parameters to a single figure quantifying the appropriate value of the design seismic intensity.

The considerable efforts spent on the subject in the few recent years are not yet sufficient to cover the whole field and to allow conclusions of practical usefulness; here an arbitrarily selected small sample of recent studies is reviewed, each one shedding some light on the problem. A striking feature that will be noted and which is common to the majority of the available investigations, is the diversity in the approaches, in the choice of the representative structural models, in the input used, in the methods of analysis and, most importantly, in the criteria of evaluation of the results. Thus, the appreciation of their practical significance and of their usefulness for deriving general design rules constitutes a major portion of the work awaiting to be performed.

The first study presented is a recent work by Goel-Chopra (1991): the general approach adopted and the main results obtained are summarized in the figs. 5, 6 and 7.

The model is the simplest possible to examine the considered aspects: a rigid floor that can move with one translational and one rotational components relative to the ground, where it receives an excitation directed along one axis (y in fig. 5).

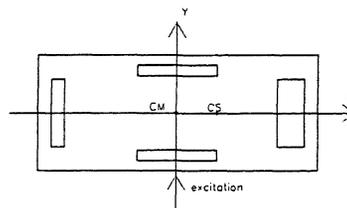


Fig. 5 - Two degrees-of-freedom asymmetric model (Goel-Chopra, 1991)

Notwithstanding its simplicity, the number of parameters required to fully characterize the model in the inelastic range (assuming a total of only four resisting elements in the two directions, as in fig. 5, is not less than ten. In order to make it feasible a

parametric investigation and the subsequent identification of their separate effects, the following major parameters only have been made to vary.

- the elastic translational frequency $\omega = (K_y/m)^{1/2}$, where $K_y = \sum_i K_{yi}$ is the sum of the element stiffnesses in the y direction;
- the ratio between the uncoupled torsional and lateral frequencies: $\Omega = \omega_o/\omega$, where $\omega_o = (K_o/mr^2)^{1/2}$ with K_o the torsional stiffness about the center of stiffness, and r the radius of gyration of the deck about CM;
- the normalized stiffness eccentricity:

$$e_s = \frac{1}{r} \frac{\sum_i K_{yi} \cdot x_i}{K_y}$$

the strength eccentricity:

$$e_p = \frac{\sum_i R_{yi} \cdot x_i}{\sum R_{yi}}$$

which locates the resultant of the yield forces of the resisting elements in the y direction;

- the normalized yield displacement $c = u_y/u_o$, where u_y is the yield deformation of the corresponding symmetric system and u_o is the peak elastic response of the symmetric system. The factor c can be roughly identified with the inverse of the behaviour factor.

For a symmetric structure, both e_s and e_p are equal to zero. For code-designed buildings, e_p is typically much smaller than e_s .

In addition to the five parameters listed above, a few important others have been considered and assigned fixed (assumed as typical) values.

These are: the ratio of the two uncoupled translational frequencies: ω_x/ω , taken as equal to one; the ratio between the torsional stiffness contributed by the elements orthogonal to the direction of the seismic motion to the total torsional stiffness, taken as equal to 0,5; the ratio of total strengths along the y-axis of the asymmetric and the symmetric structures, taken as equal to 1,0, and the viscous damping of the system, taken as equal to 5% of the critical.

A selection of the results of the analysis is reported in the figs. 6 and 7.

Figure 6 illustrates a phenomenon which might be regarded as extreme in the quantification presented, but whose importance to variable extents is physically plausible in almost all cases.

The seismic input is represented in this case by a displacement pulse, to which there corresponds a response spectrum having the customary features of the three almost constant response amplification branches.

The two diagrams represent the elastic (fig. 6a) and the inelastic (fig. 6b) (for $c = 0,1$) responses respectively, subdivided into the rotational and the translational contributions. It is seen that in the elastic case the presence of an eccentricity in the stiffness produces a significant amplification of the torsional response, while in the inelastic case this effect is almost absent, and the global response is less than or equal to the elastic one.

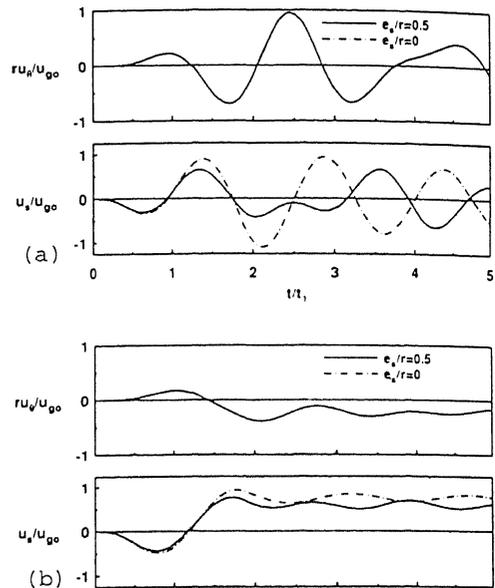


Fig. 6 - Rotational and translational responses. a) elastic case; b) inelastic case

The explanations for this relatively well known phenomenon are mainly two: the first one is that once the system becomes fully plastic in the longitudinal direction, its torsional stiffness becomes infinite with respect to the translational one, and this fact leads to a relative reduction of the torsional response; the second one is that with only two resisting elements in the y direction, the center of stiffness jumps instantaneously from its original position to the location of one of the two elements, according to which one is not plastic at that moment, and this fact leads to a partial cancellation of the torsional excitation.

The magnitude of the effect just described depends on several factors, in particular on the yield factor (the greater the effect the lower is c), on the presence of stiff and strong elements in both directions, and on the elastic period of the structure (short period structures tend to undergo large torsional plastic deformations).

Referring to the original publication for a detailed examination of the separate effects of the various parameters, in fig. 7 are collected some results which might be considered to have a rather general and conclusive character.

The ordinates of the curves give in all cases the maximum values of the displacements among all elements due to the superposition of the translational and rotational components, normalized to the peak ground motion. This latter consists of the record of the S00E component in El Centro, on May 18, 1940.

It is seen that the presence of an eccentricity $e_s/r > 0$ in all cases increases the maximum response of the most stressed element, but also that the amount of this increase is not really very significant. Comparing the three diagrams, one may note further that the increase

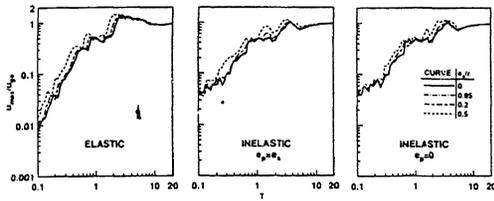


Fig. 7 - Maximum absolute displacements for different eccentricities

becomes appreciable for eccentricities in excess of, say, $0,2r$, while below that value the differences between dotted and full lines tend to vanish. Comparing the elastic case with the inelastic ones it is seen that, independently of the eccentricity, the peak responses of the latter are larger than the former ones in the low period range (say for $T \leq 0,3+0,4$ secs), a long recognized fact for simple oscillators, while in the medium to long period range the peak response are not appreciably dissimilar and one might say also not very dependent on e_c/r , especially in the case of zero strength eccentricity. Therefore, for the particular input and range of variables considered one could conclude that as concerns the effects of the eccentricity alone, their evaluation on the basis of an elastic behaviour should provide a reasonably approximate and conservative estimate.

A slightly more realistic case is treated in the study due to Dolce-Ludovici (1992) presented in the following. The authors examine a building three storey high, geometrically symmetrical, composed by frames in both orthogonal directions, spaced at 5 m interval with total dimensions in plan of 15 x 25 m.

The only source of asymmetry lies in the position of the center of the mass with respect to the geometrical center in plan: three values of eccentricity have been considered. Since the cross sections of the frame elements are symmetrical and remain constant (only the reinforcement varies from case to case), the center of stiffness is also fixed and coincides with the geometrical center.

The design input is given by the Response Spectrum of Eurocode 8 (both type A and type B Spectra have been used) with a peak ground acceleration of $0,35 g$; the behaviour factor has been taken in all cases equal to 4. For the subsequent non linear analyses, 10 spectrum-compatible simulated accelerograms have been employed with a total duration of 15 secs.

Two criteria have been adopted for the design of the building, for each value of e_m : in the first case the mass eccentricity has been introduced in the design analysis and the frames dimensioned accordingly, in the second case the eccentricity has been ignored at the design stage, but then in both cases the non linear analyses have been conducted with the mass located at the intended value of e_m .

This was done essentially to assess the potential of an elastic design analysis of adequately accounting of

the dynamic amplification of the torsional response.

The results of the study are summarized in the figs. 8a and 8b.

Figure 8 compares the results for the two types of design earthquakes, for the three different eccentricities (including zero): the buildings have been designed using a 3D model accounting for the actual position of the masses.

Since the particular building considered is more sensitive to the type B earthquake, we restrain our comments to this case. It can be seen that the greater the eccentricity of the mass, the lower is the peak ductility required to the columns. Not shown in the figures, the required amount of reinforcement in both the beams and columns increases in the most stressed external frame and decreases in the one at the opposite end, the total amount remaining practically unchanged.

Judging from these results, it would appear that if the presence of an eccentricity is duly accounted for at the design stage using a 3D model of the structure and the conventional elastic dynamic analysis with the

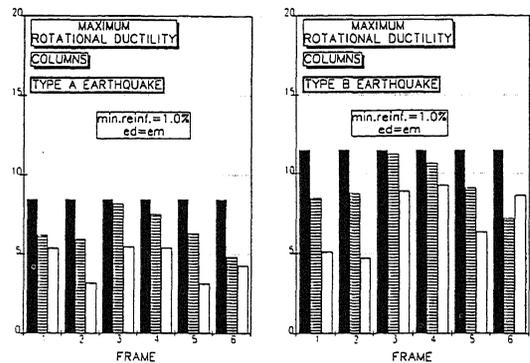


Fig. 8a - Maximum ductilities in frames. Black: $e_m = 0$; dashed: $e_m = 0,15$ D; white: $e_m = 0,30$ D; design made accounting for the eccentricity (Dolce-Ludovici, 1992)

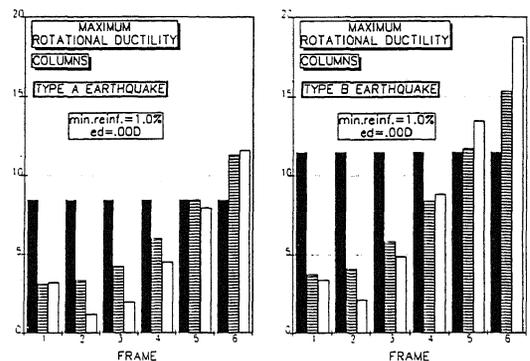


Fig. 8b - Maximum ductilities in frames. Black: $e_m = 0$; dashed: $e_m = 0,15$ D; white: $e_m = 0,30$ D; design made ignoring the eccentricity

forces reduced by the behaviour factor, the resulting structure has at least the same level of protection against the U.L.S. of a similarly designed symmetrical structure.

Ignoring [†]in the design the presence of an eccentricity, on the other hand (which might occur in practical cases if one would forget the possibility of an accidental eccentricity) can have serious consequences, as illustrated in fig. 8b. It is seen in fact that for the peripheral frames the ductility required (either peak or average) increases considerably.

The third study, due to Duarte-Costa (1988) is one of the few available investigations on the effects of the infills on the global structural behaviour, and its results are rather instructive.

The structure examined is geometrically and structurally symmetric, and it is made up by 6 three bay frames in one direction, and by 4 three bay frames in the orthogonal one. The spacing between the frames is 5 m. All the frames are infilled at all the four storeys: the stiffness and strength properties of the infills have been taken from test data available in the literature, conveniently modified to account for the openings. The structure loses its symmetry and its regularity along the height in the course of the dynamic response, since it is excited simultaneously by two orthogonal translational motions plus one rotational component. The asymmetry of the time history motions causes unsymmetric deterioration on the infills in plan, while the larger shear forces at the lower storeys affect predominantly the infills at these levels, leaving the upper storeys almost undamaged.

The design of the buildings has been made for the Eurocode 8 spectrum (Type B) with a peak ground acceleration of 0,1g applied in both principal directions, plus a rotational spectrum (rotation about the vertical axis) derived assuming all the motion as consisting in S-waves propagating along a vertical and a horizontal direction, with the power equally subdivided between these two directions. A constant value of the behaviour factor: $q = 3$, has been used in all cases described subsequently.

Four criteria have been considered in the design of the building. Case 1 is a reference case, in which the infills are not considered, neither in the design nor in the dynamic verification. Case 2 represents current practice, in which the presence of the infills is ignored at the design stage, but their presence has been accounted for in the nonlinear verification. Case 3 represents an extreme departure from current practice, in that the total strength of the infills has been deducted from seismic design force, and the remaining part (divided by q) assigned to the frames. Case 4 includes two intermediate choices, in which 70% and 30% of the total force has been attributed to the frames, respectively, and then divided by q .

For the nonlinear analyses, stiffness-degrading shear elements have been used to simulate the interstorey behaviour of frames, and stress and stiffness degrading

| CASE | MAX DUCTILITY(γ) | | COLLAPSE ACC. (A_c) cm/s ² | STRENGTH(α) RATIO (FRAMES) |
|------|---------------------------|-----------------------|--|---|
| | 100 cm/s ² | 200 cm/s ² | | |
| 1 | 3,1 | 4,9 | 380 | 0,25 |
| 2 | 1,6 | 5,8 | 374 | 0,25 |
| 3 | 6,3 | 2,3 | 157 | 0,25 |
| 4a | 3,2 | 7,3 | 276 | 0,22 |
| 4b | 2,9 | 3,1 | 143 | 0,16 |

shear elements for the infill panels, coupled in parallel with the former to give the resultant F- δ relationship.

The main results obtained from the analysis are collected in the following table. The behaviour was essentially similar in all 4 Cases, consisting in the progressive destruction of the infills at the base floor and partially at the first floor with increasing intensity of the excitation, with the final formation of a story hinge mechanism.

For a peak ground acceleration equal to the design one ($A = 100 \text{ cm/sec}^2$), Case 2 shows that the infills are still beneficial since they assist in reducing the maximum ductility demand in the frames, while for example in Case 3 it is evident that the stage of destruction of the infills is more advanced and therefore the frames are already more called into play.

By doubling the ground acceleration, the differences among the various cases become evident: the best behaviour is exhibited by Case 1, where no infills are present; in Case 2 there is already an indication that, by preserving the upper floors, the infills tend to create a concentration of ductility demand at the base, and this phenomenon becomes patently clear in the Cases 3 and 4, though in different measures.

The next column at right contains the values of the collapse accelerations in the various cases, collapse being defined either as an interstorey ductility demand in excess of 15 or as an interstorey drift in excess of 3%, whichever occurs first.

Looking at the values, it is easy to conclude that it is not safe to count in the infills for the safety of the buildings: what matters is the strength of the ductile elements, i.e., the reinforced concrete frames. This statement is confirmed almost by the letter by the figures in the last column, which represent the ratios between the shear strength assigned to the frames at the design stage and a force given by the product of the mass of the building and the acceleration at collapse: the figure is practically identical in all cases.

The problem treated in the study of Shahrooz-Moehle (1990), i.e., the effect of the setbacks along the height of the buildings, is mentioned in almost all seismic codes in the world, yet the solutions offered are still based in difficult to support "expert judgement". The parametric analysis conducted by the authors refers to the case illustrated in fig. 9: ten story buildings with "towers" of different height and contrast of slenderness with the lower portion. Though the examined cases clearly do not represent an exhaustive catalogue of the possible, or even typical, situations, the results obtained

allow, at least qualitatively, to extend the scope of the conclusions to a wider category of geometries.

A first phase of the study is dedicated to see whether a better behaviour of the buildings with setbacks could be obtained by using in the design a dynamic method of analysis instead of a static one. To this purpose, the structures in fig. 9, characterized by different values of n_1/N and m_1/M , have been designed twice, according to the UBC static method and to the ATC spectrum and modal analysis procedure, respectively, for the same base shear resultant. Subsequently, they have been analyzed in the non linear range using four well known recorded accelerograms, and for each element the averages of the computed responses were compared for the two designs. The resulting distributions of rotational ductility demands indicated a similar pattern and amount required ductility, so that it has been concluded that a dynamic method of analysis does not seem sufficient to account for the change of behaviour that takes place in the inelastic range and which causes a concentration of the ductility demands in the areas immediately below and above the base of the towers.

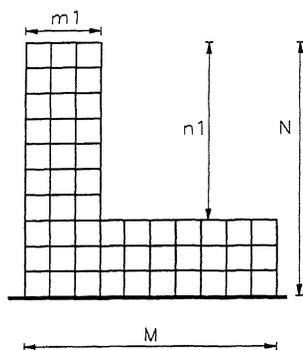


Fig. 9 - Geometry of the setbacks examined (Shahrooz-Moehle, 1990)

The indications from preliminary non linear analyses were that the "irregular" behaviour is essentially related to the difference in the lateral deformability between the towers and the base, the deformability of the former depending on the combination of their "width" with their height.

In order to evaluate this relative deformability in a simple manner, the authors propose to model the actual building as a 2 DOF system, one stiffness and one mass for the tower and the base, respectively.

The generalized stiffnesses K_i and mass M_i are easily derived assuming for ex. a linear modal deformation for each of the two parts.

From the maximum response of the 2 DOF model to the pertinent seismic input the quantity of interest is the maximum value of the ratio between the drifts of the upper and the lower parts. The parametric investigations indicate that when the drift ratio is less than about 2, the design can be made based on conventional static or

dynamic method of analysis, and the resulting behaviour is satisfactory, otherwise the tower has to be designed for increased forces, so as to avoid a disordered demand of ductility.

The specific proposal made in the paper is to use the maximum deformation of the 2 DOF system as mode shape of the building and to distribute the total shear force according to this bi-linear shape; this procedure automatically assigns a greater portion of the total force to towers that are more tall and deformable, and has proved to be effective in eliminating the concentrations of ductility demand.

Finally, the study by Costa-Oliveira (1990), provides an integrated approach in which the effects of irregularity are related to the values of the behaviour factor necessary to maintain an uniform reliability level.

The population of the considered buildings, shown in fig. 10, is characterized by the same overall dimensions in plan and internal spacing of the resisting elements: these, however, may consist of frames and core elements in two different layouts in plan, or of frames and external walls, while the regularity in the vertical direction is described through five different configurations. The buildings can be 12, 16 and 20 story high.

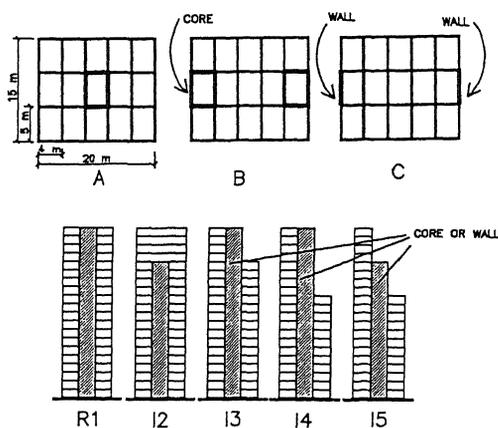


Fig. 10 - Typologies in plan (upper) and in elevation (below) (Costa-Oliveira, 1990)

The end purpose of the study is to obtain for each of the $3 \times 3 \times 3 = 45$ example buildings the appropriate values of the behaviour factors, derived in terms of constant reliability with respect to the U.L.S.

The buildings are designed elastically according to the Portuguese Code (zone A, type I, soil type 2) with a spectrum scaled to a peak acceleration of 0,175 g. Spectrum-compatible accelerograms have been used for the non linear analyses.

The variable adopted for the definition of the U.L.S. is the peak ductility demand, independently of the storey height and of the member where it occurs. This choice is evidently conventional and conservative, since

for the actual collapse to occur the formation of a mechanism is required; it could also be observed that a single peak variable might be excessively dispersed and this would add to the global conservatism. Other choices, however, could be easily implemented.

A first step in the reliability analysis is the evaluation of the vulnerability functions, that is the variation of response variable of interest as a function of the peak ground acceleration. A number of these functions are represented in fig. 11 (average values over 4 accelerograms).

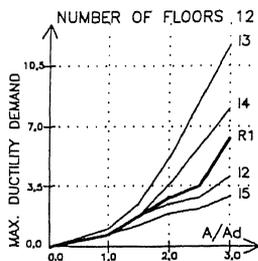


Fig. 11 - Average vulnerability curves for the various buildings

The curves are instructive per se, since they show how difficult is to approach the problem of irregularity on the basis of qualitative "good reasoning": for example, it would have been difficult to predict that the two "irregular" buildings I2 and I5 are *less* vulnerable than the perfectly regular building R1.

It is not necessary for the purpose of this presentation to enter in the details of the subsequent reliability analysis. It suffices to say that the probability of "collapse" has been evaluated by convoluting the probability of exceedance of the ductility capacity, which is a random variable, with the hazard proper of a given Portuguese site. The transformation of the action into the action effect is accomplished via the vulnerability functions.

Some of the final results are shown in the fig. 12, which gives for each of the 45 buildings the values of the "force behaviour factor": v required in order not to exceed with an annual probability of $P_f = 10^{-5}$ the ductility value of 4. The force behaviour factor is defined as the ratio between the maximum forces obtained in the elastic case and in the elastic-plastic with strain hardening case, respectively: the values of v are smaller than the values of the more commonly used q factor of about 15%.

As regards the interpretation of the result shown, no systematic trend is discernible. In qualitative terms, referred to the particular typologies used in the study (frames + walls), a general result is that irregular distributions and increased demands of ductility occur in the proximity of the interruption of the (stiffer) wall elements.

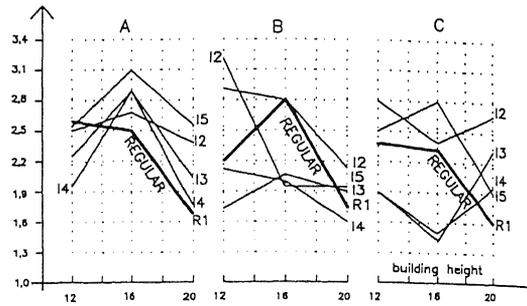


Fig. 12 - Isoprobable force behaviour factors for the various buildings

4.2 Behaviour factors

For all the theoretically well founded criticism that has been, and from some parts still is, leveled at the "wrong" idea of determining the seismic effects by coupling the intrinsically elastic concept of modal analysis with seismic actions reduced because of the post-elastic behaviour, and after that of superimposing these effects with those elastically computed due to gravitational loads, the procedure has irreversibly gained the favour, when it has not been yet implemented, of practically every seismic code in the world. It is clear, however, that this success proves only the following point, i.e., that, for "wrong" that it may conceptually be, the procedure is capable of being calibrated to give "correct" results: from the point of view of the codes there is no need to push the theoretical debate any further.

But then the question arises: is the background supporting the values of the q factors adopted in the various codes based on common, or at least comparable, criteria, and can it be considered as sufficiently probative?

The answer requires some distinction to be made, since the situation is not uniform across the countries. In the area falling within the philosophical sphere of influence of New Zealand, for example, the question might even be regarded as of modest relevance. According to the N.Z. philosophy, the principal aim of the design process is that of proportioning the relative strengths of the elements of a structure such that it will be 'forced' to behave according to a well definite, desirable inelastic mechanism, and this aim is achieved by means of extensive recourse to capacity design procedures.

Additionally, members are detailed to possess ductility properties which can tolerate possible local demands in excess of the expected one.

In this situation, q values can be established confidently without the need for sophisticated calibrations. Not so in other areas, as for example in Europe, where the tendency is to leave the options regarding the balance between strength and ductility more open and possibly with less "redundancy" than in

N.Z., a fact which makes it necessary a finer and explicitly rational quantification of the q's. And then we assist to a situation analogous to the one previously described regarding the effects of structural configuration (no wonder, because they are two aspects of the same problem): too many possible cases to be treated, the analytical effort too onerous, lack of harmonized models and criteria, etc.

For the moment, we register the fact of a continuous flow of intelligent but isolated contributions that keep accumulating: their synthesis is a task for a near future.

To exemplify the nature of the problems encountered in the quantification of the q factors, in addition to the previously described work of Costa-Oliveira (1990) a case study developed by Kappos (1991) is summarized and commented. Two typical reinforced concrete structures, a frame and a dual (frame + shear wall), shown in fig. 13, have been designed elastically using the Spectrum of the CEB Seismic Model Code (1985),

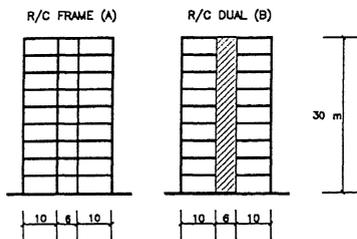


Fig. 13 - Reinforced concrete structures examined (Kappos, 1991)

scaled to a peak ground acceleration $A_g = 0,25g$ and reduced by behaviour factors $q = 3,5$ for the frame and $q = 2,1$ for the dual, following the indications of the Code for the intermediate class of ductility. For the assessment, inelastic dynamic analyses have been carried out by using three recorded accelerograms: El Centro (1940, S00E), Pacoima Dam (1971, S16E) and Thessaloniki (1978, N30E), chosen for having the corresponding spectral peaks close to the periods of the example structures, and scaled to have the same spectral velocity intensity. In the analyses the elastic stiffnesses of the elements were varied between lower and upper bounds, and important sensitivity of the results to this aspect was found.

Two criteria, whichever resulted to be the most severe, were considered for defining the limit state of collapse, a): failure of the most stressed member for the exceedance of its plastic rotation capacity, this latter evaluated accounting for the unfavourable effect of shear, b): storey failure, whenever the maximum drift exceeded the fixed value of 2%.

For the frame structure, the Pacoima Dam record resulted as the most severe for both the failure criteria: figure 14 (at left) shows the distribution of the drifts along the height for the design value of A_g , and for the three records. To reach a drift of 2% at the 7th floor the

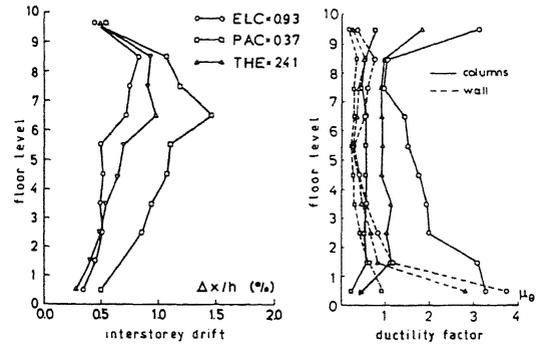


Fig. 14 - Interstorey drifts for the frame structure (left), and peak ductility demands for the dual structure (right) for $A_g = 0,25g$

intensity of the Pacoima record has to be raised to $A_g = 0,35g$, which constitutes also the collapse value of A_g , since the first member failure would occur (in the lateral columns at the base) for A_g as high as $0,58g$. Thus, according to the stipulated conventions, the "actual" value of q for the frame would result as:

$$q = 3,5 \cdot \frac{0,35}{0,25} = 4,9$$

It could be noted however, that the drift limit might, at least to some extent, be made dependent on the actual deformation capacity of the members, otherwise flexible structures, for which the drift limit is likely to be more critical than the ductility capacity, might result excessively penalized.

Figure 14 (at right) refers to the dual structures, and confirms the sensitivity of the response, in this case the ductility requirements, to the characteristics of the ground motion. It can be seen, for example, that base columns have a required ductility of 3,3 for the El Centro record scaled at $A_g = 0,25g$, while they remain elastic for the other two.

This results, however, is obtained using the upper bound values for the stiffnesses; if the lower ones are adopted the situation reverses and the Pacoima record becomes critical, leading to a collapse value of $A_g = 0,40g$ for the columns, and of $A_g = 0,52g$ for the central wall. For this stiffer structure the drift limit is of no concern: in order to reach it the Thessaloniki record should be scaled at $0,92g$. Using the most restrictive criterion for collapse, i.e. failure of the columns, the calculated value of q would result in this case:

$$q = 2,1 \cdot \frac{0,40}{0,24} = 3,4$$

The study presented is believed to be paradigmatic of the amount on inevitable subjectivism which still presides over the calibration of q values, and to provide a number of indications on where efforts to reduce it should concentrate.

4.3 Capacity design

The denomination "Capacity Design" (C.D.) refers to all procedures aimed at enforcing preferred inelastic response mechanism while at the same time avoiding the formation of brittle or otherwise undesired mechanisms. It represents an intelligent and effective technique, originated in N.Z. more than twenty years ago and subjected to continuous refinements since then, and its importance is recognized in all modern seismic codes. Does it still contain aspects which are worth further research efforts? The answer is positive, not for the concept itself but for its implementation with regards to the following three aspects: *rationalization*, *differentiation*, *simplification*. *Rationalization* involves explicit recognition of the nature of the problem, and of the techniques best suited to provide solutions, as contrasted with the purely empirical, though competent and reasonable, approach followed until now. C.D. is, in fact, an application of system reliability theory in presence of several and important sources of uncertainty, namely: *modeling errors*, due to the inadequacy of the structural models and methods used in design (ex.: the "equivalent" static linear analysis) and also of the models used for assessing the capacity of the elements; *pure randomness*, due to the nature of the input (predominant factor) and to the natural scatter of the mechanical properties of the materials, *systematic but unknown variability*, related to the unavoidable differences between the results from the analysis and actual construction. The proper theoretical framework of dealing with this problem is system reliability theory, with stochasticity present both in the input and in the system properties. It is readily conceded that a systematic coverage of all the factors above for a representative sample of the structural types included on the code is beyond practical reach, even if the available algorithms for the reliability analysis were computationally more efficient than they presently are. Sensitivity analyses on selected typical cases, on the other hand, to see the relative importance of the different factors at the scale on the system, not on a element-by-element basis, and to detect possible undue compounding and conservatism, might be undertaken right from now, and they would certainly be worth the effort.

The point for the need of *differentiation* is readily made: it comes from the fact that C.D. procedures are strictly related to the other aspects of design, most notably the amount of ductility which is exploited and the corresponding values of q . This reason makes C.D. procedures not directly transferable from one code to another without a check of their compatibility. For example, the future harmonized European Seismic Code (Eurocode 8, Draft 1988) foresees three classes of ductility; this is necessary in order to accommodate the widely different existing situations with regard level of seismicity, construction technology and economy. A graduation of the severity of the C.D. measures is

introduced in Eurocode 8, from the more stringent which refer to the higher ductility level, to the almost nominal ones for the lower level. The problem of adjusting the C.D. procedures would have to be faced in all cases in which modifications are introduced in the ductility requirements.

A more rigorous approach to the graduation of C.D. procedures than the pure common sense which is adopted presently would be obviously welcomed.

The aspect of *simplification* is equally clear as the previous ones, although the opinions here are more diversified.

Capacity Design requires, as it is well known, the design of a structure to be made according to a rigid sequence, all the beams first, then the columns, the beam column joints, etc., a fact which is a departure from standard practice, and not one in the direction of more simplicity.

Much more cumbersome, however, is the fact that columns have to be designed individually for design actions evaluated passing through the capacity of the beams, as previously designed. This not only increases the total amount of work of the designer by orders of magnitude (this objection can be countered by noting that design is becoming informatized at a rapid pace), but it is also against the real practical advantages of a standardized production.

The question of whether some simplification alleviating the before mentioned aspects would be feasible is not therefore without cause. Among the few studies known to the author in this area, the one by Dolce-Evangelista (1991) offers an appreciable amount of information, due to the width of scope of the investigation made, covering a total of 720 cases.

The study refers to plane frames, with 2 bays, 4 and 8 story high. The other parameters are: the average axial stress in the columns (3, 4 and 5 MPa are used), the height of the beams (40 and 60 cm), three types of stiffness distribution along the height, two values for the minimum reinforcement ratios (1% and 0.3%), two values for the design acceleration (0,25g, 0,35g), and five values for the "overstrength factor": β , a factor which is applied to all the moments in the columns obtained from the analysis to obtain the design values. β has been varied between 1 and 3.

Some of the results from the study are illustrated in fig. 15, which gives the peak ductility demands and the steel ratio in the columns as functions of the variable β .

It is seen that a constant value of β of the order of 2 is already sufficient to reduce the peak ductility to a very low value and to cancel to a great extent the effects of the other design variables, while keeping the necessary amount of steel to acceptable levels. The values of β calculated accounting for the "overstrength" of the beams and of the "dynamic amplification factor" would have resulted of the order of 2,0+2,7. Although the evidence provided by this study is far from exhaustive, it gives an indication of possibilities that merit to be further investigated.

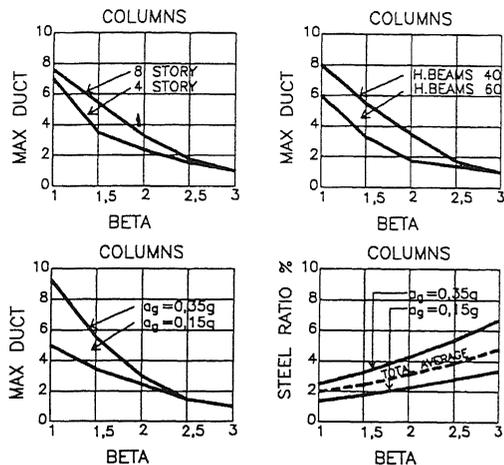


Fig. 15 - Peak ductility demands and required steel ratios on columns as functions of β , for different parameters (Dolce-Evangelista, 1991)

5 CONCLUSIONS

Conclusions are out of the scope of this presentation. The thesis that the link between structural norms and applied research is stronger today than ever in the recent past is also easy to justify with arguments and to verify in the reality. This being established, views are expressed on a selected number of normalization problems that are in need of support from fresh research work, and an opinion is given on the progresses likely to be achieved in the short to medium period.

REFERENCES

- Banon, H., Veneziano, D. 1982. Seismic Safety of Reinforced Concrete Members and Structures *Earth. Engng. Struct. Dyn.*, Vol. 10, 1713-1729.
- Chen, P., Powell, G.H. 1982. Generalized Plastic Hinge Concepts for 3-D Beam Column Elements. *Rep. EERC 82/80, Univ. California, Berkeley.*
- Chung, Y.S., Meyer, C., Shinozuka, M. 1987. Seismic Damage Assessment of Reinforced Concrete Members. *Tech. Rep. NCEER 89-0033, State University of New York at Buffalo.*
- Clough, R.W., Benuska, K.L., Wilson, E.L. 1965. Inelastic Earthquake Response of Tall Buildings. *Proceedings, Third World Conference in Earthquake Engineering, New Zealand.*
- Coelho, E. 1992. Comportamento Sísmico de Estruturas em Portico de Betão Armado. *Doctoral Thesis, Universidade Tecnica de Lisboa, Lisboa.*
- Comité Euro-International du Béton. 1985. Model Code for the Seismic Design of Concrete Structures. *Bull. CEB n. 165, Paris.*
- Costa, A.G., Oliveira, C.S. 1990. Influence of Vertical and Horizontal Irregularities in the Seismic Response of Buildings. *LNEC Report, Lisbon.*
- Dolce M., Evangelista, L. 1991. The Strength of Building Columns Required to Ensure an Acceptable Seismic Response (in Italian). *V Nat. Conf. in Earth. Engng., Palermo.*
- Dolce, M., Ludovici, D. 1992. Torsional Effects in Buildings under Strong Earthquakes. *Earth. Eng. and Struct. Dyn., Submitted for Publication.*
- Duarte, R.T., Costa, A.C. 1988. Earthquake Behaviour of Reinforced Concrete Frame Structures Infilled with Masonry Panels. *LNEC Report, Lisbon.*
- Fardis, M.N. 1991. Member-Type Models for the Nonlinear Seismic Response Analysis of Reinforced Concrete Structures. *Experimental and Numerical Methods in Earthquake Engineering.* J. Donea, P.M. Jones (eds.) EEC, Brussels.
- Fardis, M.N., Sfakianakis, M.G., Christopoulos, P.A. 1991. Models of Inelastic Cyclic Shear and Flexural Behaviour of Reinforced Concrete Members. *ICES 91 Int. Conf. in Comput. Engng. Science, Melbourne.*
- Filippou, F.C. 1993. Analytical Models. *Behaviour and Analysis of R.C. Structures under Alternate Actions Inducing Inelastic Response, Ch. 4. CEB Bulletin (to appear), Lausanne.*
- Filippou, F.C., Issa, A. 1988. Nonlinear Analysis of Reinforced Concrete Frames under Cyclic Load Reversals. *E.E.R.C. Report n. 88/12 University of California, Berkeley.*
- Filippou, F.C., Issa, A., D'Ambrisi, A. 1992. Nonlinear Static and Dynamic Analysis of Reinforced Concrete Frames *Report N. EERC/92/1X, University of California, Berkeley, in print.*
- Goel, R.K., Chopra, A.K. 1991. Inelastic Seismic Response of One Storey, Asymmetric Plan Systems: Effects of Systems Parameters and Yielding. *Earth. Engng. Struct. Dyn.*, Vol. 20, 201-222.
- Kappos, A.J. 1991. Analytical Prediction of the Collapse Earthquake for R/C Buildings: Case Studies. *Earth. Engng. Struct. Dyn.*, Vol. 20, 177-190.
- Park, Y.J., Ang, A. H-S, Wen, Y.K. 1984. Seismic Damage Analysis and Damage Limiting Design of RC Buildings. *Tech. Rep. SRS N. 516, Dept. Of Civ. Engng. University of Illinois at Urbana.*
- Park, Y.J., Reinhorn, A.M., Kunnath, S.K. 1987. IDARC: Inelastic Damage Analysis of Reinforced Concrete Frame-Shear Wall Structures. *Tech. Report NCEER 87-0008. State Univ. of N.Y. at Buffalo, N.Y.*
- Priestley, M.J.N., Seible F. 1991. Seismic Assessment and Retrofit of Bridges. *Rep. N. SSRP-91/103, Dept of Appl. Mech. and Engng. Sciences, Univ. of California, San Diego.*
- Sfakianakis, M.G., Fardis, N.N. 1991. Bounding Surface Model for Cyclic Bending of RC Sections. *ASCE Jnl of Engng. Mech.*, V.117, 12.
- Shahrooz, B.M., Moehle, J.P. 1990. Seismic Response and Design of Setback Buildings. *ASCE Jnl of Struct. Engng.*, Vol. 116, N. 5.
- Takayanagi, J., Schnobrich, W.C. 1976. Computed Behavior of Reinforced Concrete Shear Walls. *Struct. Res. Series N° 434, Civil Engng. Studies, Univ. Illinois at Urbana - Champaign.*
- Tseng, W.S., Penzien, J. 1975. Seismic Analysis of Long Multiple-Span Highway Bridges *Earth. Engng. Struct. Dyn.* V.4, 3-24.
- Vaz, C.T. 1989. Analise Estatica Nao Linear de Pilares de Betão Armado Sujeitos a Accoes Repetidas e Alternadas. *Res. Rep. N. 036/12/9376. LNEC, Lisbon.*
- Zeris, C.A., Mahin, J.A. 1988. Analysis of Reinforced Concrete Beam Columns under Uniaxial Excitation. *ASCE Jnl of Struct. Engng.*, Vol. 114, N. 4.
- Zeris, C.A., Mahin, J.A. 1991. Behaviour of Reinforced Concrete Structures Subjected to Biaxial Excitation. *ASCE Jnl. of Struct. Engng.*, Vol. 117, N. 9.