2830

DESIGN FOR LOW/MODERATE SEISMIC RISK

Paolo E PINTO[1]

SUMMARY

The increase of knowledge about seismic hazard worldwide, associated to an increase of awareness of the potential losses due to earthquakes, and economic progress allowing for policies of risk reduction, have led several countries in which seismic hazard was formerly ignored to pay greater attention to seismic design. The paper deals with a number of aspects related to this increased interest. It is first discussed, with reference to the solutions adopted in a few countries, the problem of where to put the lower limit to seismic intensity for which explicit seismic design is worth being carried out. It is shown that the limit varies considerably among countries, depending on economic conditions and on the general quality of the constructions. Based on the results of analytical simulations illustrated in the paper, it is concluded that modern RC buildings can have a substantial capacity to resist earthquake motions, if they have a regular configuration and are correctly designed for gravity loads only. In terms of PGA, this capacity may go up to 0.15 ÷0.25g for structural damages still far below the ultimate state. This stresses the greater importance , especially in regions of moderate seismicity, of providing the profession with documents of good practice, rather than with analytically sophisticated codes. Looking at the future, ample space is given in the paper to discuss about which one of the two design methods, the established force–based or the emerging displacement–based is best suited to the needs specific to L/M risk zones. In their present state, they are both less effective for L/M areas than in ones of high seismicity. The DBD approach offers better perspectives for being extended to the L/M case on more rational bases.

INTRODUCTION

As a consequence of the general increase in material well being occurring in most parts of the world, a change in attitude towards risks is taking place, be these of natural origin or man-made. Even in countries which are still in the process of emerging economically, the need of employing part of the available resources to prevent or to reduce the risks is now explicitly recognized.

Seismic risk represents an interesting case apart. While in high seismicity places with an advanced economy, as for ex. in California, Japan, etc., the seismic threat is a constant concern which regulates all human activities that can be affected by it, there are many parts of the world where earthquakes do occur, but either rather infrequently or with a moderate intensity, so that they tend to be perceived more as "accidents" than regular, if rare and relatively minor, physical phenomena, and therefore considered not worth of a systematic effort for reducing their effects.

Yet the situation in these low seismicity places is evolving visibly in the last few years, in a way which depends on the particular combination of a number of factors. The first one is the large amount of efforts which are being spent to improve the knowledge on seismic activity worldwide. The most important project in this field is the well known "Global Seismic Hazard Assessment Program" [GSHAP, 1993], endorsed by the UN/IDNDR, whose goal is to assist especially developing countries in the evaluation of their seismic hazard in a regionally coordinated fashion and with the most advanced methods. Implementation of the GSHAP is entrusted to 9

[1] *Department of Structural and Geotechnical Engineering, University of Rome, Italy e-mail: pinto@uniroma1.it*

Regional Centers, hosted by geophysical institutions in all continents: 1) North and Central America (UNAM, Mexico City), 2) South America (CERESIS, Santiago), 3) Central and Northern Europe (GFZ, Potzdam), 4) Mediterranean (CNPCRST, Rabat), 5) Continental Africa (University of Nairobi), 6) Middle East (IIEES, Teheran), 7) Northern Eurasia (IPE, Moscow), 8) Central-Southern Asia (SSB, Bejing), East Asia-Oceania (PHIVOLCS, Manila). A first result of GSHAP will be a computer-based model of earthquake potential that can be utilized to produce seismic hazard maps at any regional and national scale, and it is natural to expect that once these maps will be actually diffused, public awareness of the seismic threat in each particular country will become more concrete.

A second factor which is slowly though inevitably producing a change in attitude towards earthquakes is the uncontrolled growth of megacities in areas of low-to-moderate seismicity, growth which is normally associated with poor and seismically unsafe constructions. Moderate and even small earthquakes may turn catastrophic in areas with poor building construction practice, as shown for ex. by the 1960 event in Marocco (M = 5.8, 12,000 casualties), and by the event of 1992 on Dahshour, Egypt (M = 5.7), which caused considerable shaking in Cairo, the heavily populated capital, leading to hundreds of life loss and thousands of injured and homeless. According to one estimate [Elnashai et al., 1994], in the whole Nile region where the rather moderate seismicity of Egypt has been observed historically (west of the Sinai peninsula), the number of old buildings susceptible to earthquake damage due to ageing and poor state of maintenance is in the range of 130,000, while the more recent constructions of 4 stories or less, designed with no intentional lateral resistance is in the range of 400,000.
Cases similar to those described abound around the world: there the risk is contributed more by the high vulnerability of the buildings than by the intensity of the hazard, and is therefore the situation in which an appropriate seismic design/retrofit code is the most effective means for reducing the risk.

Finally, a serious incentive towards the use of a seismic code even in countries whose seismicity is definitely low comes from their state of affluence. The hazard in these countries is not such as to pose a life threat to the buildings occupants, especially considering the generally good quality of the construction. The authorities however, with the consent of the owners, are willing to impose an additional initial cost for an increased protection against even small damages, not least in order to reduce discomfort following small shocks. In Europe, a situation of this type can be found in Germany and Switzerland, were seismic design is required in certain parts of the territory. Figure 1 [Mayer-Rosa, 1993] reports the histograms of the seismic intensities of all the events contained in the national catalogues of Germany, Switzerland and Greece, updated to 1985. The comparison with Greece clearly suggests that in the former two Countries seismic hazard should be given a different treatment than in Greece.

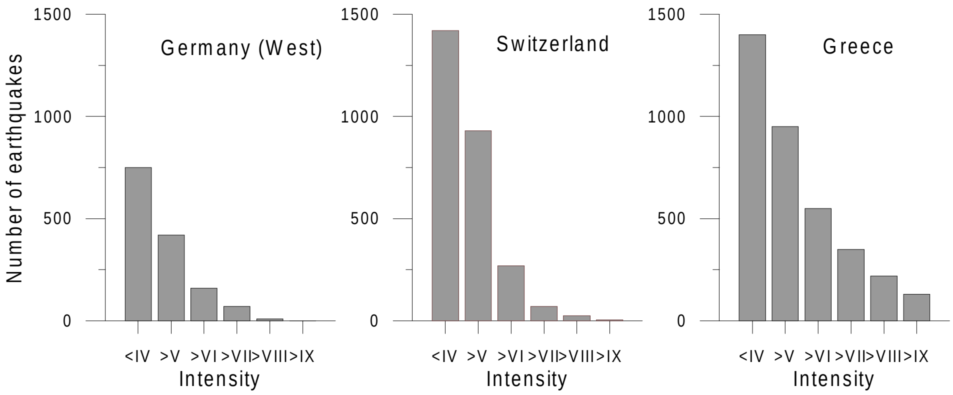

Figure 1 Histograms of the intensities in three European Countries

In summary, the three main factors entering into the decision of whether and how to deal with seismic risk when this latter is not obviously significant are: (1) physical, quantitative knowledge of the hazard and awareness of it from the public, (2) capacity on the part of the authorities to regulate the construction process and to impose the use of antiseismic standards, (3) availability of the resources necessary for ensuring protection against different levels of damage. Developing countries are often deficient on (1) and (2); rich countries remain with the problem of assessing the optimum amount of resources to be used in order to minimize overall cost and inconveniences in the use of buildings.

For oversimplified and incomplete the above list may be, it is sufficient for realizing that the problem of seismic design in low/moderate hazard regions cannot have a single, rational solution applicable to all cases. While the principles of seismic design are universal, their implementation requires a compromise to be reached based on the situation as defined by the combination of factors (1) to (3).

In those regions where hazard is low, the question that comes first in whether it is worthwhile to account for it in the design, or to ignore it altogether. It is clear that an answer to this question cannot be given without proper consideration of the prevailing type of construction in the regions concerned: it is again a matter that can only be solved on a case by case basis. The solution given to this problem by a small sample of countries involved is briefly described in the following: interesting differences can be noted.

MINIMUM HAZARD LEVEL FOR SEISMIC DESIGN

In all countries, design seismic actions and procedures are specified in the so–called zonation maps, which divide the territory into areas in which uniform criteria apply. In principle, the actions to be used in each area should be calibrated to provide the target protection levels with respect to life safety and collapse. In practice, many if not most of the existing zoning maps have not yet been updated with incorporation of the latest hazard studies. Even in those cases where updating has been carried out, the revisions have generally been minor, especially when the revisions would have involved changes in consolidated tradition, even if this latter could no longer be justified on a rational basis.

A few cases when the zoning, proposed or already in force, is more or less strictly related to the estimated hazard, are briefly mentioned below, with specific reference to the way the deal with the question of the minimum threshold for applying seismic design.

United States [FEMA 303, 1998]

The last revision of the NEHRP *Provisions* [FEMA 303, 1998] contains rather innovative proposals for the definition of the design ground motion. A major point is the decision to base the definition of the design action on the hazard having a probability of exceedance of 2 percent in 50 years ($T_R \cong 2{,}500$ years), instead of the previous and customary 10 percent in 50 years ($T_R = 475$ years). The hazard is described by two parameters: the 5% damped spectral ordinates at 0,3 sec (high frequency: S_h), and at 1 sec (low frequency: S_e). In many situations, the design ground motion is obtained by multiplying the spectral ordinates having a $T_R = 2{,}500$ years by a factor of 2/3. The underlying justification of this procedure is not going to be discussed here. What is relevant here is to point out that according to the new *Provisions*, all places where the $T_R = 2{,}500$ years S_h is less or equal to 0,25 g and S_e is less or equal to 0,1 g should be considered as places of negligible seismicity, for which seismic design is not required. The only requirements are a minimum lateral resistance of 1 percent of the dead load of the structure, and the adoption of a few simple detailing and connection rules. (those applicable to Seismic Design Category A, not intended for ductility).

In order to assess the threshold adopted by the *Provisions*, and also for the sake of comparison with other codes, the value $S_h = 0{,}25$ g, corresponding to the range of the maximum amplification of the spectrum, is first divided by a factor of 1,5, so as to obtain roughly a value with a return period of 475 years. One gets: $S_h = 0{,}25/1{,}5 \cong$ 0,l7 g. According to prevalent opinion, a rather stiff building with a period around T = 0,3 sec., capable of resisting an horizontal force just equal to 0,01 W, and with little intended ductility, should not be able to resist such a high elastic spectral acceleration, which in terms of ground acceleration could be in the order of ~ 0,07 g, without considerable structural damage or even collapse, and this would be in explicit contradiction with the fundamental safety objectives of the Provisions. These latter, however, are said to be based on recent evidence during the Northridge event in 1994, where buildings underwent spectral accelerations of that order of magnitude suffering only minor damages. The way of reconciling theory with reality might be that those buildings did possess unintentional lateral strength much larger than 0,01 W. A minimum lateral strength of this order is actually specified in a number of European codes as a general requisite of "robustness", unrelated to any specific action.

Switzerland [SIA 160, 1989]

Switzerland is kwon to be a Country of low seismicity, with the only exception of a small zone in the upper Rhone Valley. In spite of this, a seismic hazard map for a return period of 400 years is included in the Swiss code [SIA 160, 1989] and consideration to seismic design is mandatory in the whole Country. The territory is divided into 4 intensity zones: the elastic response spectra for each zone are shown in Figure 2. Considering the generally low level of the hazard, limitation of damages, rather than avoidance of collapse, represents the main objective of the code. This is reflected in the low values adopted for the ductility-related force reduction factor (K), which from a ceiling of K = 2,5 for ordinary buildings goes down to 1,4 for more important structures.
Looking for the lower threshold of the seismic action to be considered in design, one sees in Figure 2 that for Zone 1 the peak ground acceleration is 0,06 g, similar to the value of 0.07g implied in the FEMA *Provisions* for

excluding seismic design. With the value above, SIA 160 requires instead structures to be designed for a force given by the spectral ordinates divided by the factor K and multiplied by a factor 0,67 (corresponding for ex. to the structural performance factor of NZ). For an ordinary building (K = 2,5) with first natural period in the range 0,33 ÷ 0,5 secs, depending on soil conditions, one would get a design horizontal force of:

$$F = \left(0.13\frac{0.67}{2.5}\right)W = 0.035W$$

that is, three and half times what is required by FEMA 303, for the same objective of preserving the structures from even minor damages, and starting from the same value of the peak ground acceleration.

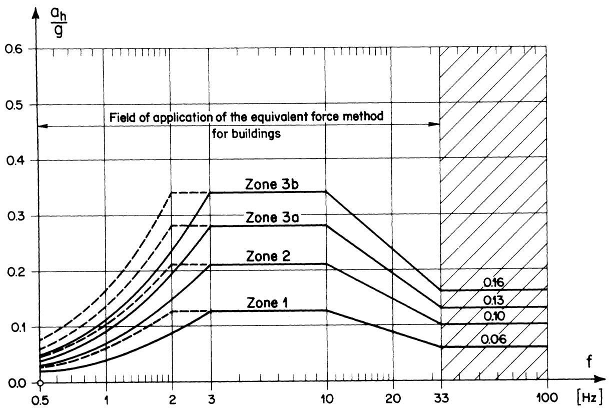

Figure 2 Elastic design spectrum (mean values, 5% damping)

Italy [Pugliese et al., 1997]

A further example of the latitude of the criteria adopted worldwide to define the lower threshold of seismic intensity requiring explicit consideration in design, a recently completed proposal [Pugliese ed al., 1997] for seismic zonation of Italy is presented.

Italy comprises areas of greatly varying seismic activity, ranging from practically zero in well defined regions to moderate-high magnitudes and frequencies of occurrences in other, equally well defined, areas. In these latter regions seismic events of magnitude in the order of M ≅ 6,5, causing typical values of PGA around 0,30 ÷ 0,35 g, are to be expected every 50 years on the average (although events with M > 7 have occurred in the past, the last one in Sicily in 1906), while the same return period can be associated to Magnitudes M = 5÷5.5 in less seismic areas. A well documented earthquake catalogue covering the past millennium and more, a wealth of isoseismal maps of Mercalli intensities for the events of the last century, and instrumental attenuation data for the events occurred after the 1975 Friuli earthquake, included, allow the seismic hazard of the Country to be evaluated with sufficient confidence. A new 475 years return period map, relative to the 8100 municipalities of Italy has been recently computed using the latest refined data, with the main purpose of checking the consistency of the present subdivision of the territory into three zones of high, medium and low seismicity, in which seismic design is mandatory, with the remaining portions exempted from it.

The intensity measure chosen for the map is the Housner intensity, defined in the study as the area under the 5%, damped pseudovelocity response spectrum between the periods of 0.2÷2 secs. The criterion proposed for the classification is actually multiparameter, including the Housner intensity having a $T_R \cong 100$ years and maximum felt macroseismic Intensity, but only the dominant indicator: H(475) will be referred to here for the purpose of illustration.

The 8100 H(475) data have been arranged to yield a distribution function, qualitatively shown in Figure 3. With m and σ indicating the mean and the standard deviation of H(475), respectively, the limits of the three zones have been selected as indicated in the Figure 3. Zone 1 includes values of H(475) comprised between (m + 1,9 σ) and the fractile at 99.5%, zone 2 the values between (m + 0,5 σ) and (m + 1,9 σ), and zone 3 the values between (m - 0,5 σ) and (m + 0,5 σ). The PGA values corresponding to the limits of the three zones are: 0,08 g, 0,14 g, 0,23 g, and 0,30 g

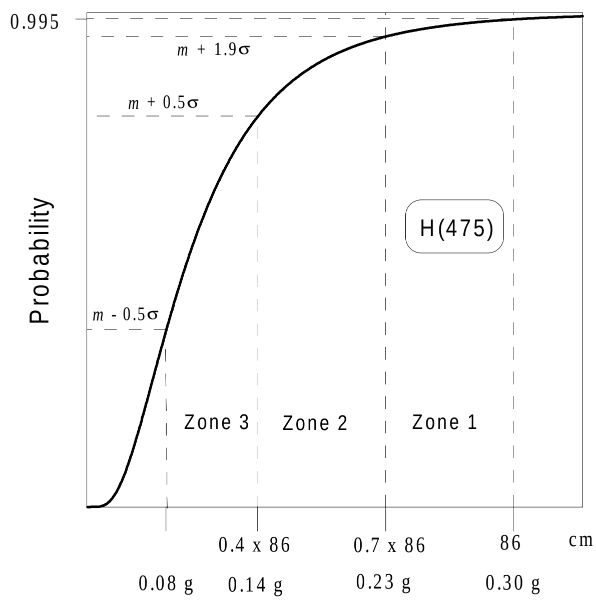

Figure 3 Distribution of historical Housner intensities and proposed zoning

The cut-off at the 99,5% fractile implies acceptance that 5 out of 1000 municipalities may experience seismic intensities larger than those adopted for the design of structures in zone 1. There is no particular logic in the choice of the separations between zones, except that the one between zone 2 and 3 involves the least modification of the present zonation. Worth of some comment is the width of lowest zone, and in particular of its lower limit, which corresponds to a PGA of 0.08 g, below which seismicity is assumed to be "zero", and design is made for gravity loads only. This contrasts neatly with what has been commented previously about USA and Switzerland (and with what is done in many other countries). The justification is of pragmatic nature. In the present zonation, zone 3 extends for 3,300 km^2 with less than 3 million people, while the proposed one would cover areas of more than 100,000 km^2 with 21 million of people, which is about a third of the total area and of the population of Italy. Had the threshold been put at a lower level, for ex. 0,06 g, almost all of the Country would have become "seismic", a too drastic change from the present situation where more than half of the Country is not considered as seismic.

Slovakia [Sokol, 1999]

The case of Slovakia has similarities with that of Switzerland: a low general seismicity level with a few small "islands" of stronger activity scattered within the territory, a tradition of ignoring the earthquake action in the design, at least for common buildings, due to the known predominance of wind action, a re–evaluation of the seismic hazard, and a tendency towards harmonization of the design procedures as embodied in the Eurocodes, in Eurocode 8 in particular.
The hazard map of Slovakia, shown in Figure 4, has a “background” seismicity covering the whole country, with a very modest PGA value of 0.03 g, and a few mostly circular areas where the PGA raises to 0,10 - 0,15 g.

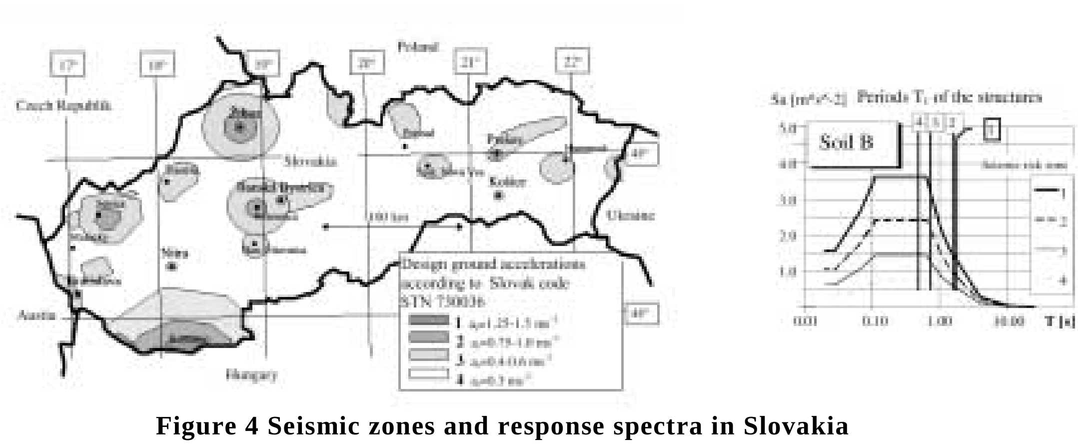

Figure 4 Seismic zones and response spectra in Slovakia

Associated with these values are response spectra shapes depending on soil conditions. With the exception of soil class D, which corresponds to a rather special situation of soft (Vs $\leq$ 180 m/sec.) upper layers, the other three classes represent soils of progressively decreasing mechanical properties, whose main effect on the spectral shape is that of extending towards longer periods the portion of constant response acceleration. In a recent study [Sokol, 1999], four hypothetical buildings, with number of floors ranging from 10 to 24 and natural periods from T = 0,5 sec. to T = 1,90 sec. have been analyzed to see under which combination of seismic zone and subsoil class the seismic action would become more important with respect to other variable actions. The most unfavourable case occurs for a wall frame building eleven story-high with a period of T = 0,495 g. The result is summarized in Figure 5.

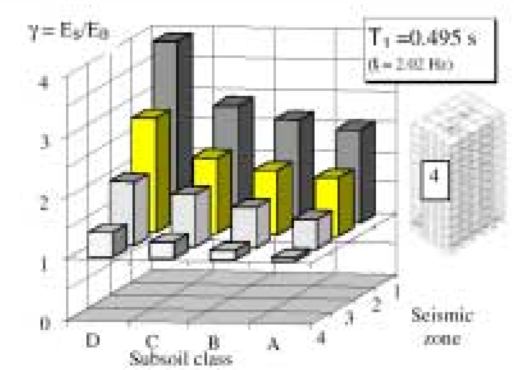

Figure 5 Relative influence of seismic and other actions as a function of seismic zones and soil classes

The ratio $\gamma = E_S/E_B$ is the ratio of the maximum effects due to the seismic action (E_S) to those due to basic actions (E_B), evaluated at some critical element of the structure. One sees that for all combinations of soil class and seismic zone γ is larger than 1, going up to a maximum of almost four, implying that for such a building any site in Slovakia would require seismic design, including the "background" seismicity area where PGA = 0.03 g. For the other buildings having larger periods the dominance of the seismic action over the other variable ones occurs essentially in the zones 1 and 2 of Figure 4.

Colombia [Garcia, 1995]

A seismic design code has been introduced in Colombia in 1984. Code provisions are given with reference to the seismic hazard map in Figure 6. The main seismogenetic features are the subduction of the Nazca Plate in the Pacific Ocean, and a series of internal faults far from the coastline with a general orientation North-South.

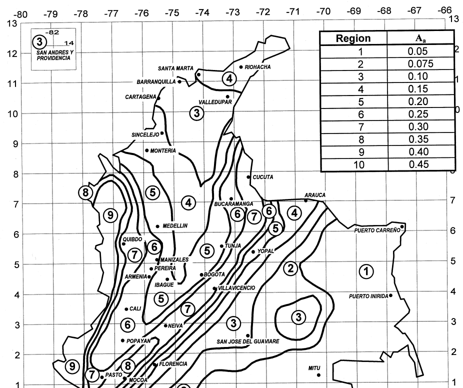

Figure 6 Seismic hazard map of Colombia

The territory of Colombia includes areas of high seismicity, with peak ground accelerations in excess of 0,40g, as well as areas of moderate and low seismicity, with the whole of the Country being defined as seismic. Although the design seismic intensity is attributed individually to the various cities, a formal subdivision of the Country in three zones is made for the purpose of differentiating seismic requirements applicable in each of them. The subdivision is as shown in Table 1.

Table 1. Seismic Risk zones in the Colombian Seismic Code

Seismic Risk Zone	PGA (%g)
HIGH	> 0,20
MODERATE	0,20 ÷ 0,10
LOW	≤ 0,10

Design values of PGA as low as 0,05 g are given in the Code for sites of low seismicity. Similar to what is done in other international codes, RC structures can be designed according to different levels of energy dissipation capacity (Ductility Classes, as they are called in Europe). The groups are defined as: Minimum (DMI), Moderate (DMO) and Special Energy Dissipation Capacity (DES), and the corresponding force reduction factors are, for frame structures, R = 2,4 and 6, respectively.

In zones of low hazard design can be done according to any of the three groups, while only DMO and DES are admitted in the intermediate hazard zone, and DES only can be used in the high hazard zones. The low value of R for DMI structures practically implies, if one considers the unavoidable sources of overstrength to be discussed later, an elastic response of the structure, and hence, by definition, a level of damage almost irrelevant. The same cannot be said about DES structures, whatever is the zone in which they are constructed, since for them the use of inelastic and, hence permanent, deformation is explicitly aimed at. It would therefore seem, that, for a design PGA having the same return period, a DMI in a low hazard zone is better protected from damages than a DES structure in moderate or high hazard areas.

SEISMIC CAPACITY OF NON SEISMICALLY DESIGNED BUILDINGS

Reinforced concrete buildings designed with little or no consideration to lateral forces have nonetheless some inherent capacity to resist earthquake actions of some intensity. This capacity is sometimes expressed in terms of the ratio between the base shear at ultimate and the weight of the building (seismic coefficient). Numerical investigations on the seismic coefficient available in existing RC buildings systematically show considerable scatter, with values ranging from as little as 1% to as much as 15 ÷ 20%, the higher values easier to be found in low-rise buildings. The quasi random nature of the seismic capacity in terms of lateral strength is the obvious consequence of the almost infinite possible geometric configurations of buildings, of the freedom in the choice of the dimensions of the members, of the reinforcement percentages above the minimum values, etc.
The lateral strength of a building, however, is not a sufficient indicator of its ability to survive earthquakes in general, since two buildings with the same coefficient but different natural periods will behave in rather different ways. Assuming, as it is widely held, that damage is directly related to displacements and distortions, the period is a more significant indicator of possible damage than strength is.

An early investigation on the risk to buildings designed primarily for gravity loads is reported in [Hoffmann et al., 1992]. Simulated designs of buildings, followed by nonlinear static and dynamic analyses to assess their seismic response, were carried out on typical frame structures with symmetrical floor plans, having identical layout in plan but different heights. The elevations of the analyzed frames are shown in Figure 7. The designs had all the detailing deficiencies of gravity load dominated structures, i.e., insufficient amount and discontinuity of bottom beam longitudinal reinforcement at joints, absence of transverse reinforcement across the joints, lack of confining hoops at the beams ends, and no adequate provisions for shear resistance. The features were at least in part modeled in the analysis.

A first result is presented in Table 2, showing fundamental periods and maximum base shear coefficients for the three buildings.

Table 2. Fundamental Periods and Base Shear Coefficients

Building Type	Fundamental Period	Base Shear Coefficient
3 Story	0,79 s	0,056
6 Story	1,15 s	0,037
9 Story	1,48 s	0,028

It is noted that the base shear coefficient of the 9 story building is exactly half that of the 3 story building, due to requirements on minimum columns dimensions and minimum column reinforcement playing a larger role for the latter.

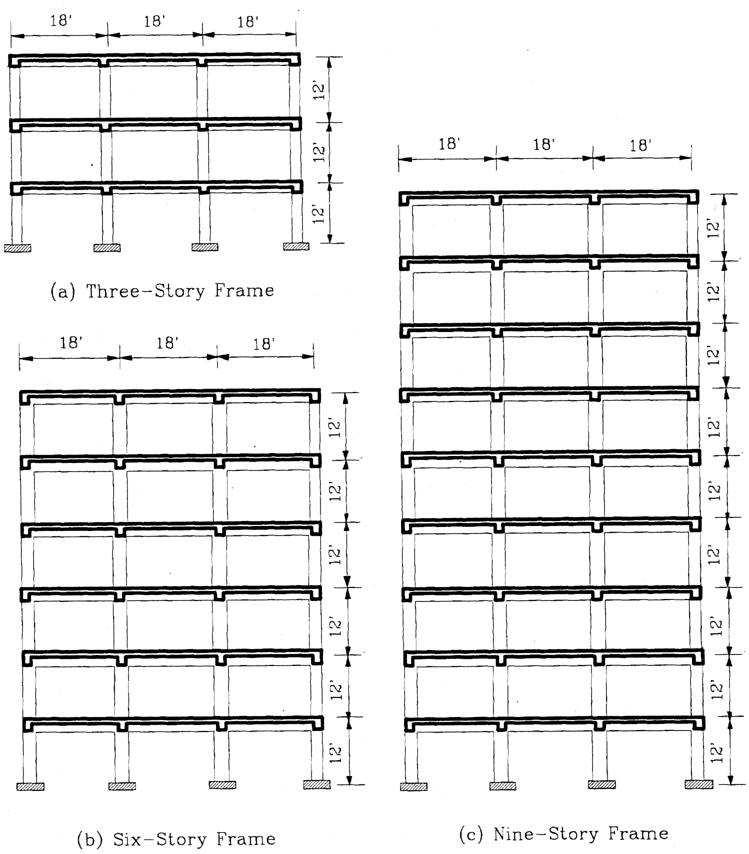

Figure 7 Frame structures analyzed by Hoffmann et al. (1992)

The analyses were performed using a number of different records, the two most significant being considered here. The first one is an artificial accelerogram matching the UBC spectrum [UBC, 1998], scaled to a PGA of 0,15 g, while the second one is the 1952 Taft (N21E) record scaled to a PGA of 0.20 g. The response spectra of the two accelerograms are shown in Figure 8.

It is immediate to note that the Taft spectrum is much more severe than the UBC one, independently of the (small) difference in the PGA. For example, at T = 1 sec., the ratio of the spectral ordinates is a little above 3, and for T = 1,5 sec. is roughly 2,6. Needless to say, the same ratios apply to the corresponding displacement spectra, so that one would expect also the response displacements and the story distortions of the frames to be approximately in the same ratio for the two accelerograms. Maximum inter-story drifts obtained from the analysis are presented in Table 3.

Table 3. Maximum Inter-story Drifts (%)

Building Type	UBC PGA = 0,15 g	Taft PGA = 0,20 g
3 Story	0,53	3,06
6 Story	0,95	4,47
9 Story	0,63	1,83

Although the figures in Table 3 refer the maximum values of drift (which is a local response measure) occurring at different floors, the ratios of the responses are not far from the anticipated values, with the exception of the 3 story building for which the Taft record greatly amplifies the inelastic response.

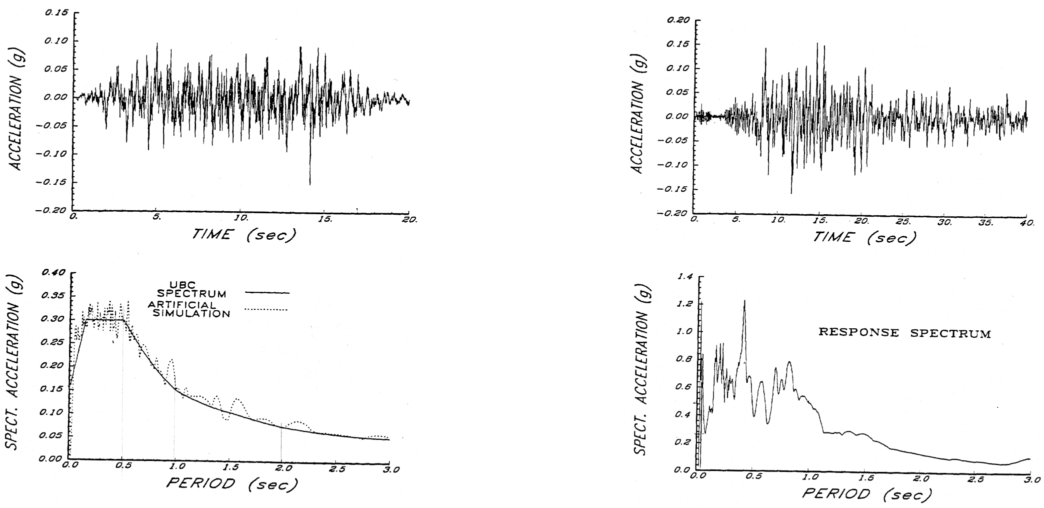

Figure 8 Accelerograms and associated spectra used for the analyses

The conclusions of the authors are that the three structures would all have survived a seismic event like the UBC for a PGA = 0.15 g, although serviceability would be totally compromised and reparability in doubt. On the other hand, none of the three would have survived the Taft event, not even if scaled to the same PGA = 0.15 g of the UBC event. It should be remarked that the UBC spectral shape is a standard one (linearly increasing spectral displacements after T = 0.5 sec.) and is similar, when not equal, to those adopted by a good number of design codes, while the Taft record is known to have a power density of rather exceptional width.

One might therefore tentatively generalize the conclusions of the authors by saying that while structures correctly designed and detailed according to modern seismic codes could have the extra properties of deformability and dissipation capacity for surviving an exceptional event, gravity load designed structures obviously lack this tolerance. Nevertheless, these latter are likely to remain standing, though damaged, for an action of significant intensity (PGA = 0,15 g) they were completely unprepared to resist.

If the above conclusions were of general validity, they would be supportive of the previously mentioned proposals advanced in the US and in Italy of disregarding seismic design where the hazard is of the order of PGA ≅ 0.10 g. It should be noted, however, that the structures examined by [Hoffmann et al., 1992] were of the most regular possible configuration, and of a type (moment resisting frames) naturally suited for distributing the inertia forces among the internal elements. Other very common structural types, one for all: the buildings with open ground floor and heavy infills at the upper stories, have proven in so many other studies to be fatally vulnerable to a PGA = 0,10 g associated with a standard spectrum.

So, the question of the minimum threshold of the hazard for which seismic design is warranted cannot be separated from the indication of the structural type which is going to resist it. If no such indication is given, then the prudent approach taken for ex. by the Swiss code, the Slovakian code, etc. i.e., to design explicitly for earthquakes even in the presence of a low hazard in order to prevent damages, would be the rational, though expensive, choice.

Additional interesting information on the seismic performance of buildings not specifically designed to resist earthquake actions can be found in a recent study by J.P. Browning [Browning, 1998]. The purpose of the study is actually that of developing a practical method for controlling buildings deformations due to future earthquakes, with particular target Central and Eastern US (CEUS), where uncertainties associated with large future events are especially large and hence do not warrant a high level of sophistication in the design method. For a better appreciation of the results produced in the study an outline of the method is given below.
In a loose sense, the method could be classified within the category of the displacement based approaches, since the starting point is an elastic displacement spectrum, which is taken to be of the simple linear form:

$$D = \alpha T \tag{1}$$

where D is the response displacement, T the fundamental period of the structure, and α, the slope of the straight line representing Eqn. (1), gives the severity of the spectrum. In view of the results to be seen later, it is of interest to note that, for all the applications meant for the CEUS, the value of α has been calculated such that the 2% damped acceleration spectrum corresponding to Eqn. (1) takes on the value of (0.25 g)×3,75 = 0,99 g for a period of T = 0,55 sec. (corner period of the acceleration spectrum). The procedure includes the steps described below.

a) a target displacement (derived from a target drift ratio) is selected for the building to be designed, and the corresponding necessary value of T is obtained from Eqn. (1);
b) members are proportioned so as to meet the target period, evaluated on the basis of their geometry and of the masses present;
c) the base shear strength of the structure is determined/checked and the same is done for the shear strengths at all stories;
d) the strengths of columns and beams are determined/checked for flexure and shear;
e) the details are provided to the members so as to allow for the selected drift.

Step b) can be accomplished in two ways. The first one is simply to design the building for ordinary loading cases, assuming typical depth to span ratios (e.g.1/10) for beams, and deriving column dimensions from axial stress limitations (ex.: $N/f'_cA < 0.3 \div 0.4$). For drift values of the order of 1,5% and α quantified as described, the condition on the period has been found to be satisfied in most cases. Otherwise, modeling the building as a SDOF system characterized by known values of T and M, the necessary generalized stiffness is obtained directly from: $K = 4\pi^2 M/T^2$, and story stiffness are derived therefrom by means of suitable assumptions on the distribution of these stiffnesses along the height of the building.

As regards steps b) and c), the basic premise is that the lateral-load capacity of a building is not a significant parameter for controlling drift, unless it is below a certain minimum. In the study, the minimum adopted for the base shear coefficient corresponds roughly to the spectral acceleration corresponding to period of $\sqrt{2}T$ divided by a force reduction factor R = 8.

The reinforcement in the beams is that required by gravity loads, (with half of the top reinforcement provided also at the bottom at beam-column joint regions), while the reinforcement on the columns is a fixed percentage of the column cross-sectional area (1% ÷ 1.5%).

Having determined the moment capacities of beams and columns, a push-over analysis with a triangular distribution of forces is carried out, and the base shear capacity of the structure is the resultant of the applied forces when the average drift ratio reaches the target value (1,5% in the study). The calculated value of the base shear strength must exceed the minimum threshold given above.

For what concerns the relative flexural strength between columns and beams, the ratio of this strength necessary for ensuring a beam-sway mechanism for all possible frame configurations is known to vary between 1 and a rather large number, say 2,5 or 3. It is shown in the study, however, through a number of representative example cases that by making this ratio equal to one a still acceptably uniform distribution of the drifts along the height of the building is obtained, so that this value is recommended in areas of low/moderate risk (L/MR).

Finally, in order to accomplish step e) of the procedure, expressions are given in the work to check whether confinement and shear reinforcements actually provided are compatible with local, i.e., story drift demand values larger than the average demand, of the order of 2%.

A total of 42 frames satisfying the requirements of the proposed method have been analyzed to determine their non linear dynamic response under a series of ten recorded accelerograms, scaled so as to produce the same spectral displacement at their own characteristic period. For the characteristic period of the El Centro 1940 record (Tg = 0,55 s) the peak ground acceleration was 0,25 g.

All frames had three bays with girder length to depth ratios of 12, the number of stories was varied from 5 to 17 and the initial periods ranged from 0,6 sec. to 2,6 sec. The reinforcement ratio in the columns was 1%, whereas beams had 1% negative and 0,5% positive reinforcement ratios. Frame members were proportioned for gravity loads, with columns satisfying the criterion $P/A_g(0.85f'_c + \rho f_y) < 0.45$. All members dimensions were checked to be representative of actual proportions in existing buildings.

The target (or limiting) periods were determined so as to achieve a mean drift ratio of 1.5%: all frames designed for gravity loads satisfied the stiffness requirement under the earthquake action. Figure 9 shows the initial periods of the frames normalized to their respective target periods, plotted versus the number of stories: it is seen that gravity load proportioning leads to a stiffness increase with the number of stories larger than that required for earthquake effects.

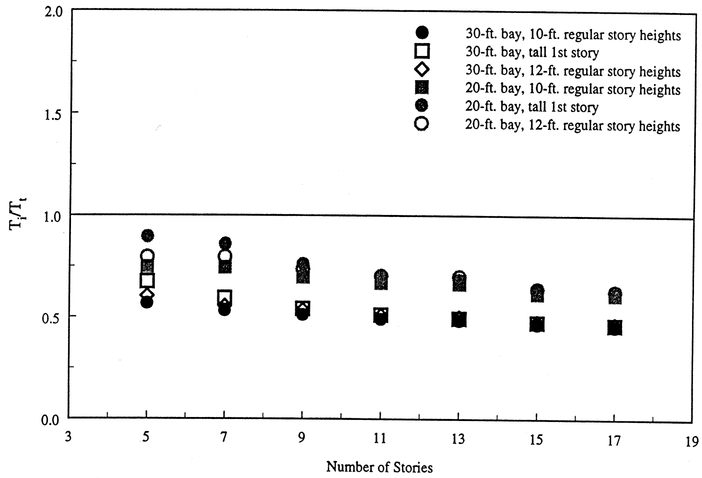

Figure 9 Normalized initial periods of frames

Results of interest from the analyses start with the values of base shear strengths of the buildings, evaluated by means of static incremental analysis. In all cases the values found did exceed the minimum values required. The shear coefficients decrease with the number of stories: approximately half of the values are below 0.04, with an isolated maximum of 0.l6 for a low-rise buildings and several values in the order of 0.02.

The results from the dynamic analyses are shown in Figure 10, which gives all maximum (in time) mean (along the height) drift ratios plotted versus the number of stories of the frames. All of the values are less than 1.1%, a result that could have been qualitatively anticipated since the initial periods of the frames were less than the target values. The distribution of maximum story-drift ratios over the height is rather uniform in all cases, and becomes nearly constant for frames with more than 13 stories.

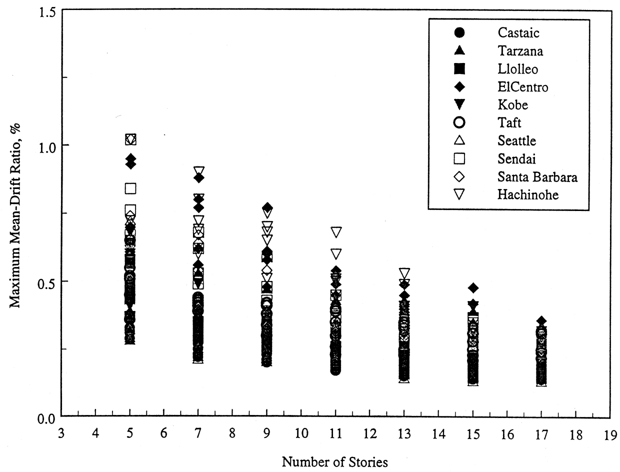

Figure 10 Peak drift ratios averaged over the buildings height

FLEXIBILITY ALLOWED BY TRADITIONAL SEISMIC DESIGN

The term traditional refers here to the approach that has been evolving since the 1970's, starting in the US and in NZ, and has by now won universal acceptance in seismic codes worldwide [IAEE, 1996]. Two really innovative features, worth being recalled lest they might be forgotten or taken for granted, characterized that new generation of codes, one related to the definition of the seismic action, the other one consisting in the development of the capacity design (CD) concept. Contrary to the previous codes, the design action was no more given apodictically, justified by tradition, but in a format where seismic hazard and structural response to it were clearly separated. The hazard was in fact given by an elastic response spectrum, often defined as isoprabable (for ex. 50% fractile), while the capacity of the structure to dissipate energy upon entering the inelastic range, and/or the willingness of the designer to exploit this capacity, is represented by a so-called reduction factor R.

The use of this factor, coupled with the adoption of CD rules in order to force the desired inelastic mechanism, has shown over the years a surprising ability to control the behavior of structures.

Initially, the values of R were established, for each structural typology and for each material type, as functions of the global ductility expected and of the period of the structure. Over the years, and continuing until the present day, a number of refinements have been introduced, by making R dependent on soil conditions [Miranda, 1993, Riddell, 1995], on deterioration due to cyclic loading [Fajfar, 1992], on higher modes effects for MDOF systems [Seneviratna and Krawinkler, 1996], etc.

Of course, the values of R also depend on the intensity of the hazard: for low intensities it may not be sensible or even possible to exploit much ductility, thanks to the intrinsic resistance of the structures against lateral loads.

Notwithstanding these refinements and in parallel with them, the limitation of having to use a single global parameter for controlling the response of a structure in the whole range of its post elastic behavior, has started to appear as a serious impediment, especially since the ideas on multiple performance objectives have taken firm roots in the earthquake engineering community. On theoretical grounds, it is in fact difficult to accept that with a single parameter applied to the design forces one could control compliance with several performance levels, defined in terms of displacements.

The search for a more intelligible and transparent metodology for achieving a better control on the objectives of the design is the driving force behind the development of the so-called displacement based design (DBD).

In the context of the present work, devoted to seismic design in L/MR areas it would highly desirable to be able to provide a comparison of the relative merits and demerits of the R-factor and the DB approaches. Conceptually, the question is open, since in L/MR areas the performance objectives to be achieved are less in number and involve less inelastic action, thus giving the strength of the structure a larger role than in high risk areas, where deformability matters more.

Unfortunately, a comparison of this sort is not yet available, if for no other reason that no single well established and recognized DB procedure exists. What will be done here is to present sample cases of use of the two procedures with no ambition of proving anything more but their applicability to L/MR areas. This paragraph is devoted to the R–approach, as implemented in the Eurocode 8 [CEN TC 250/SC8, 1994]; the following one derives from recent works [Fardis and Panagiotakos, 1997, Priestley, 1999] where specific DB procedures are developed.

Basic Structure of Eurocode 8

As it is widely known, the European Union (EU) has promoted, starting from about fifteen years ago, the preparation of a set of documents covering the design of all types of ordinary structures under all types of loading. Out of the set of nine documents, Eurocode 8 (EC8) is the one dealing specifically with seismic design. The countries involved in the updating of the Eurocodes are those of the EU, with the recent addition of those belonging to the EFTA, for a total of 19. Other eastern European Countries like Slovenia, Croatia, Rumania, Bulgaria, Slovakia, etc., who are in the list of future members of EU, are also showing interest in EC8 and are since some time comparing and trying to harmonize their national codes to it.

EC8 belongs to the category of traditional (i.e. force–based) codes, whereby the design forces are obtained from the elastic spectral accelerations (i.e. probabilistically defined hazard) by dividing them by a "behavior factor" q (henceforth called R), held constant through the three branches: constant acceleration, velocity and displacement of which the spectrum is composed. The application rules, and in a particular the CD rules, are designed so as to link as closely as possible the values of R to the preferred inelastic mechanism, its global displacement ductility, then to the corresponding local curvature ductility and finally to the detailing rules necessary to meet the ductility demand.

Given its nature of a regional code, EC8 must have sufficient flexibility for adapting itself to widely different conditions: differences in tradition, technical background, construction practice and, more to the point, in seismic activity. The main instrument to achieve this flexibility is the faculty of choosing between three so-called Ductility Classes: high (DCH) medium (DCM), and low (DCL). In the intention of EC8, the three Classes should be equivalent in terms of safety with respect to the ultimate limit state: all factors and applications rules for each Class have been calibrated with this aim. The choice between Classes should therefore be dictated, only by the convenience of the particular user, although the DCL has been included in EC8 having in mind especially the case of regions of moderate or low seismicity.

The R factors corresponding to the three Classes are in the ratios of 1 : 0.75 : 0.5, indicating that design forces for a DCL structure are double than those of a DCH structure, for the same peak ground acceleration and hence elastic spectrum. In particular, for RC frames of regular configuration the R-factor is equal to 5, 3.75 and 2.5 for DCH, DCM and DCL, respectively. Assuming in approximation $R = \mu_g$ ($\mu_g \cong$ global ductility), in a perfect beam-sway mechanism, which is the one sought in EC8, chord-rotations of all beams and at the base of the bottom story are equal to μ_g. For a hinge length of 0,5 m, which might be typical of a building element, the required curvature ductility for the three DC levels would then be: 10, 7,50 and 5. The values above, even if they were increased by 50% or more to account for the simplifications made, can be easily accommodated by means of proper ductile detailing.

CD rules are used in EC8 to prevent flexural yielding in columns, and shear failures both in beams and in columns. These rules are obviously more stringent for DCH than for DCM, while they are not required at all for DCL. This decision has been taken in consideration of the low ductility demands expected in DCL structures, demand that can be met through a moderate ad hoc detailing. As a final point worth mentioning, EC8 takes explicitly into account the possibility of incomplete moment reversal in the beams due to the dominant effect of gravity loads: when this is the case the CD magnification factor for the design of columns is appropriately reduced.

Sample numerical calibrations for low and moderate seismicity

A great deal of calibration studies have been performed in the Universities and within specific EU-funded research programs to test consistency and viability of EC8 procedures and parameter values. For the purpose at hand, a minuscule extract of one of the largest and more systematic numerical analyses [Fardis & Panagiotakos, 1997] undertaken for checking the results obtainable from EC8 is briefly illustrated in the following.

The structure chosen for illustration is a 12 story frame structure whose layout in plan is shown in Fig.11.

The structure has been designed for four different conditions: with a PGA = 0,15 g (scaling the EC8 spectrum) for DCL and DCM, and with a PGA = 0,30 g for DCM and DCH.

The results from standard EC8 design are summarized in Table 4.

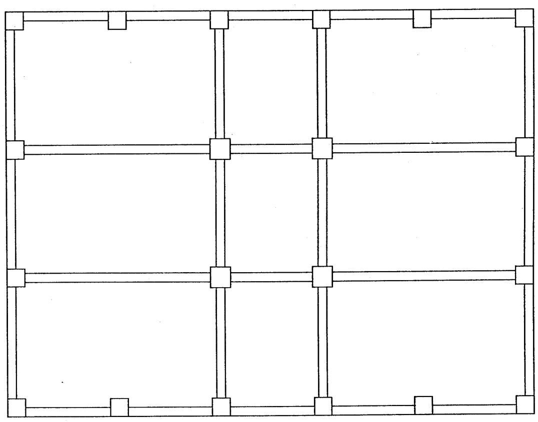

Figure 11 Layout in plan of the EC8 frames

Table 4. Global Design Results for the 12 Story Building

Type	Ag/DC	R	T (sec)		V_b/w (%)		Max Drift (%)		Concrete	Steel		
			X	Y	X	Y	X	Y	(m^3)	Total (t)	beam / col	long./ trasv.
12/Frame	0,15/L	2,50	0,66	0,72	17	16	0,24	0,29	643	97	37/63	80/20
“	0,15/M	3,75	0,66	0,72	11	11	0,24	0,29	643	94	32/68	74/26
“	0,30/M	3,75	0,58	0,63	23	23	0,47	0,56	682	117	42/58	78/22
“	0,30/H	5,00	0,58	0,63	18	18	0,43	0,52	712	126	36/64	64/36

From this and from the other examined cases the following trends are detected.

a) the values of the interstory drifts under the design seismic action, unconservatively evaluated using the uncracked stiffnesses of the members (as required by EC8) appear quite low;
b) for a given ground acceleration the DC has very little systematic effect on the quantities of concrete and steel;
c) in the passage from DCL to the higher ductility levels there is a noticeable (and explainable) shift of the steel from the beams to the columns, and from the longitudinal to the transverse direction.
d)

Following their design, the frames have been subjected to nonlinear dynamic analyses, using accelerograms consistent with the design spectrum, and progressively scaled up until a PGA twice the design value was reached. The main results from the analyses are described for the particular case chosen, but their validity extends to the whole population examined. There are two major aspects worth commenting. First, under the design action extensive cracking of the elements takes place, as expected, and this has effect of lengthening the predominant period of the structures, with respect to the value used at the design stage, of an average factor of 2,5, which is increased by a further 25% when the excitation goes up to twice the design one. This lengthening of the periods reduces the force demands on the structures: on the average, and with little dispersion, the reduction amounts to 1,8 and 2,0 times for the cases of the design and of twice the design actions, respectively. The result is equivalent to having used R values reduced by the same factors. At the same time, the lengthening of the periods has the effect of increasing the deformations of the structures: drift ratios at the design intensity are from 2,5 to 4,0 times the elastic values calculated using the stiffnesses of the gross uncracked sections.

The second major aspect emerging from the results of the nonlinear analyses is the well known [Park, 1996] problem of the unintentional overstrength due to rounding-up of bars, to minimum measures and detailing rules for gravity loads, to the continuation of bars required by neighboring sections, etc. Attention is called to the fact that in considering the sources of overstrength one should not include the presence of safety factors, like the partial safety factors for steel and for concrete, the DC magnification factors, etc. These are in fact safety elements intended to cover possible dispersions and uncertainties, which exist in reality but cannot be accounted for in a deterministic non-linear analysis. Considering only the actual physical sources of overstrength one would find values in the range 1.2 to 2.0, the last value mainly in the columns, and especially in the upper stories.

If the two effects, of the reduction of the demand and of the increase of the supply, are combined, one may reach the conclusion that, at least for DCL structures, the actual value of R is close to 1,0. This is confirmed by the results of the analyses, which indicate that DCL structures come out with very little damage (less than 5%) from the design earthquake and with not much more (less than 10%) for the doubled intensity. But in all cases of DCM and DCH the damage seldom exceeds 10%, whatever the intensity.

Summing up, this small but representative sample of simulated designs and subsequent analyses shows that the R-factor approach keeps providing consistent results when applied for low design intensities at least when these do not go below a PGA of 0,25 g. The illustrated examples confirm however the weak points of the method, in particular its failure to yield realistic values of the displacements. The gross overestimation of the design forces due to the use of unrealistic values of the members stiffnesses, and hence of the period of the building is another weakness, one that could be at least partially remedied with the use of reduced values of the stiffnesses. This a priori reduction, however, could only represent a rough estimate, whose approximation would vary from case to case (reduction factors to account for cracking found in the literature range from √2 to 3). So, it seems possible to conclude that the R-factor approach remains a simple and, in the ways it is implemented in the codes, generally conservative design method, but its simplicity (the use of a single factor governing all post elastic behaviour) limits the control on the response variables which are important for checking the performance of the structures.

PROMISES FROM DISPLACEMENT–BASED METHODS

Based on the results one normally gets from force-based designs (FBD), as exemplified by the small but typical sample of simulated designs presented above, it would be difficult for the common code user to formulate harsh judgments on the method as a whole, the sole exception being possibly a request for a better approximation in estimating the deformations.

Yet if one starts thinking on the conceptual bases of the FBD, one is forced to admit that these bases are anything but sound and, even if the results from the method are acceptable, the search for a more rational and effective alternative appears as an imperative. The reasons, although well known and at least partly already anticipated, are worth being exposed in full.

gravity loads and earthquake actions are orthogonal between them, and so are their effects on structures. Reasoning in terms of the forces induced by each system, if the intensity of the seismic action is reduced for the purpose of design in view of the post-elastic reserves of the structure, while the gravity loads remain constant, the actual ratio between the two effects is altered: when or if the seismic action exceeds the design value the variation of the combined effect is not proportional and not quantifiable a priori, in the sense that it may lead to reversing the sign of some of the action effects (bending moments and normal forces), as it often does.

The R-factor approach has been conceived with reference to a SDOF nonlinear oscillator, for which vertical loads, if present, have no other relevance than that of modifying its constitutive law. Consequently, the approach maintains a practical validity in those cases where seismic forces dominate over gravity loads, while it becomes almost irrelevant when the opposite is true, as is the case in the L/MR zones.

in the FBD approach, the lateral strength of the structure is circa proportional to the intensity of the seismic forces (disregarding the intrinsic, or “natural” fraction of this strength), and the lateral displacements are also proportional to seismic intensity. This obvious fact implies, as a less noted consequence, that it is not feasible to satisfy, as a direct design objective, any fixed code limit to the displacements. Yes, the structure can always be stiffened/strengthened in a second phase to respect these limits, but this would certainly not be called an effective approach to a performance-based design. Further, it has been recently shown [Priestley, 1999] that designing for fixed, code-prescribed deformation limits requires a base shear strength increasing in proportion to the square of seismic intensity, not just to the intensity, as done in the FBD. Practically, this implies that satisfaction of code limits, if not originally achieved, requires adjustments of no small importance.

in the FBD approach as adopted by present codes, displacements are evaluated based on an elastic analysis of the structure under the set of forces deriving from the unreduced response spectrum. Generic warnings, or empirical suggestions, are given in order to approximate the actual stiffnesses of the members taking into account of their respective probable amount of cracking. This procedure leads to rather erratic estimates of the deformations.

In a formal definition, a DBD method is one that, *given* a precisely defined target limit state configuration of the system (≅ target performance level (PL)), and *given* the seismic action associated with it, provides the stiffness and strength distributions necessary to achieve the objective.

A direct and accurate implementation of this definition into a practical design method of general applicability is still a distant goal, though promising attempts are available.

Difficulties of various nature exist, leaving aside the one which is common to all design methods, i.e., that a given objective can normally be achieved by means of a multiplicity of structural solutions, so that even “direct” methods need to start from some arbitrarily chosen initial condition, one that can be intrinsically sub-optimal for the objective.

We start by discussing a difficulty which couples the definition of the PL with the method to achieve it. For all levels who are not related to heavy damage and near collapse conditions, as for example the “operational” level (i.e., the ex serviceability LS), there is not a corresponding global configuration of the system. These PL’s actually consist of conditions on strains in concrete and in steel to be attained, but not exceeded, in an undefined number of sections. The stress here is in the words *attainment* and *number*, since a structure satisfying by equality the conditions on strains in just a few sections and by inequality (below the limit values) in all the others could not claim to be designed for a particular PL.

But even if the question of the number of sections would be solved in some pragmatical way, the point is that in dynamics the maxima of local quantities, as the strains are, are attained at different times in different locations, and it is hard to imagine a *practical direct* method capable of ensuring the fulfillment of all of the conditions.

This limitation is perhaps more severe in the L/MR zones than in high seismicity ones, since in these latter emphasis is on the ultimate levels of response, for which the idea of a global mechanism is more realistic.

Even in this case, though, a possible objection against starting with a target displacement pattern, which is the distinctive feature of a direct DBD, is that imposing, or guessing, a vibration shape, is an operation likely to succeed in the case of regular structures, but it may well fail to do so for irregular structures, whose dynamic behavior, especially in the post elastic range, is not easy to anticipate.

Two procedures for DBD will now be briefly presented, chosen within a still small group of other proposals (for ex.: [Mohele, 1996]) for their longer development life and larger amount of test applications. Their suitability for use in L/MR areas will be subject to specific remarks.

The procedure proposed in [Priestley, 1999] and in [Priestley and Kowalski, 1999] is the extension to building structures of a concept that has been under development since the early nineties, first for SDOF systems ([Kowalski et al., 1995] and then to MDOF bridge systems [Calvi and Kinsley, 1995]. It includes the following steps.

a) *target displacement.* As already discussed, to each PL there must correspond a design displacement. The problem associated with PL’s defined in terms of material strains has also been noted earlier: it requires further developments.

   Damage limit states are generally defined in terms of drift. The admissible values of drift depend on the specific structure considered, and must be evaluated; in any case they must not exceed the limits prescribed by the code, if any.

   On this last point, one might argue that for PL’s corresponding to extensive structural damage it would be more rational to evaluate the drift from fundamental principles based on the characteristics of the structure, rather than to have fixed limits imposed for all structures, which may well correspond to different levels of damage.

   The admissible drift limits refer to single stories. In [Priestley, 1999] a simple empirical expression is provided for evaluating the story drift at yield θ_y, which only requires the value of ε_y and the ratio between beam bay length and beam depth, while the plastic contribution to story drift: θ_p, is obtained by adding θ_y to the difference between ultimate and yield curvatures, multiplied by the length of the plastic hinge. The total design story drift is therefore: $\theta_d = \theta_y + \theta_p < \theta_c$, with θ_c = code limit, and is important to note that it can be defined with sufficient accuracy without prior knowledge of the reinforcement in the elements.

   Next comes the critical passage, i.e., from story drift to a displacement shape of the structure. Figure 12, taken from [Priestley, 1999], illustrates the proposed choices, which depend on the assumed behavior of the structure, whether of the shear or of the flexural type.

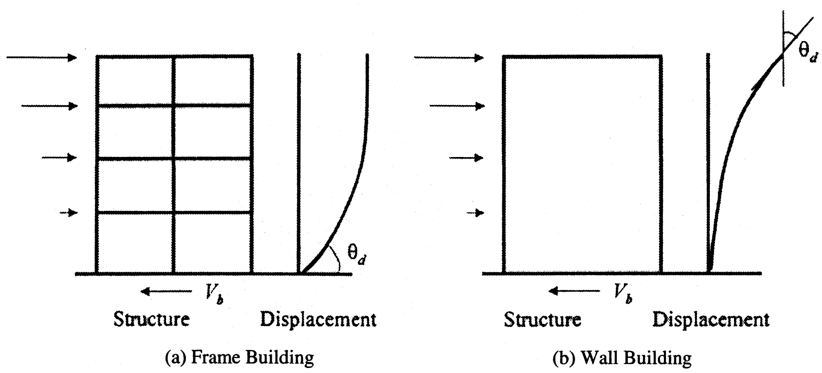

Figure 12 Critical drifts for building structures

In the first case the critical drift occurs at the base, in the second at the top: in both cases approximate, empirical displacement profiles can be constructed for the whole structure.

This construction is needed for two purposes: a) to provide, based on the "distance" between the critical and the yield displacement profiles, a measure of the *ductility* corresponding to design displacements, and b) to allow determining the parameters of an equivalent SDOF structure: the equivalent displacement Δ_d and the equivalent mass M. It is noted that the procedure works on the assumption that critical drifts occur at specific stories and only there. This is conditional to the deformed shape remaining unchanged during the inelastic dynamic response; there are no later steps in the procedure, however, for controlling that drift values larger than the admissible ones do not occur also at some other stories.

In fact, in all examples shown in the previous paragraphs maximum story drifts occur often at stories above the bottom one, depending, given the structure, on the particular ground motion used and, given the ground motion, on the specific geometrical and mechanical features of the structure. There can be design measures ensuring avoidance of upper softer stories as, for example, the adoption of constant column dimensions and reinforcement along the height of the building, but these measures become obviously uneconomical with increasing number of stories. They could be recommended in L/MR zones, however, since the design strength required in those cases is always rather low in absolute terms, and so would therefore be the extra cost involved.

b) *response of the SDOF equivalent structure* . Recourse to a SDOF oscillator "equivalent" to a MDOF structure, when the vibration shape of the latter can be assumed to be known and fixed, is expedient in many problems in dynamics and in earthquake engineering. The method is at the base of the increasingly popular push-over analysis, now being considered not only for the assessment of existing but for the design of new structures as well. The specific application of the criterion as proposed by Priestley consists in using the calculated displacement Δ_d as the response of the SDOF oscillator obtained from the pertinent nonlinear design displacement response spectrum. From the known spectrum, the period T corresponding to the displacement Δ_d is read directly: the structure must be designed for this value of T. This is done first by solving for the stiffness K: $K = 4\pi^2 M/T^2$, and then obtaining the required strength: $V = K\Delta_d$.

What has been called earlier nonlinear response spectrum is actually constructed by associating to the structural ductility: μ an "equivalent" hysteretic damping factor $\zeta = \zeta(\mu)$, and then by reducing the ordinates of the design, 5% damped, response spectrum according to the value obtained for ζ.

c) *design of the structure*. The base shear V is distributed along the height of the building in proportion to story masses and displacement values: $F_i = V(m_i\,\Delta_i)/(\Sigma\, m_i\,\Delta_i)$.

Under these statically applied forces the building is analyzed to obtain internal action effects. For frame structures, it is suggested to use in the analysis the (estimated) cracked moments of inertia for the columns, and the (estimated) cracked moments of inertia divided by the system ductility: μ, for the beams. Capacity design procedures are to be used for the final dimensioning of columns in flexure, and of beams and columns in shear.

d) *applications*. A number of applications are shown in [Priestley, 1999], for frame and for wall structures. An interesting result concerning frames is that if the code drift limits are of the order of 2%-3%, the amount of structural ductility that can be exploited is rather low, of the order of 2 or less. This is because for frames of ordinary dimensions, if one takes into account slip of reinforcement at the column forces, shear deformations in the (cracked) joints and shear deformations of the element themselves, the drift at the onset of yielding is already of the order of 1% ÷1,5%.

If further analyses and tests on real scale structures would confirm this fact, the implications on the current design methods would be significant. Many important codes use R-factors for ductile structures in the range from 6 to 8, and detail the elements so as to achieve local ductilities three or four times the global values. These provisions would become void, if drift at low ductilities would be the controlling factor. Then, either R-factors should be reduced, and with them ductile detailing accordingly, or drift limits should be relaxed.

Another approach to DBD, which is less direct than the one just discussed, in the sense that the desired displacements are *obtained* from the procedure, rather than representing a *known initial objective*, is that proposed in [Fardis and Panagiotakos, 1997]. It is a comprehensive procedure, only apparently close to a traditional force-based approach, but containing in reality a few ideas which, adequately developed through further experimental work and analytical modeling, may provide a significant improvement over existing design practice. The procedure consists of the concepts and of the steps described below.

a) *serviceability LS under seismic action as first dimensioning criterion*. It has been observed in previous sections that the lateral strength capacity of a structure is not a critical factor in determining its behavior under a strong seismic action, provided it is not disproportionately low. In this proposal, the basic strength capacity is obtained from the so-called "serviceability" seismic limit state, i.e., one in which the seismic action might have a return period of the order of the design life of the structure (ex. 50 years for an ordinary building). This level of seismic action, combined with the quasi-permanent value of the gravitational loads (in European notation: $G_K + \psi_2 Q_K$, where G_K is the mean value of the permanent loads, Q_K is the characteristic value of variable loads (mean return period of 50 years), and ψ_2 is a combination factor (typical values 0.2 – 0.3) such that $\psi_2 Q_K$, called quasi-permanent value, is the 50% fractile value in the distribution of loads "as sampled"), determines the main longitudinal reinforcement *in beams*.

The section flexural capacity is evaluated using non factored (characteristic) values of the material strengths, and *serviceability* values of concrete and steel strains (for ex.: 0.0035 and 0.01, respectively). For calculating the seismic response using the appropriate response spectrum, the period of the structure should better be obtained using cracked values of cross section inertia. Alternatively, a global increasing factor of 1,5 could be applied to the value of the period based on gross sections.

Of course, the reinforcement in the beams obtained from the seismic condition cannot be less than that necessary of complying with the other non seismic load conditions.

At the beam–to–column faces, the bottom reinforcement in the beams, if not required by the analysis, needs not to respect the classical 50% condition relative to the top one: a minimum percentage should be provided at this stage.

Capacity design calculations are used for determining design moments on *columns*, as well as shear forces in the beams, in the columns and in the joints: the reinforcement is thus known all over the structure.

b) *peak response under the design (damage LS) seismic action*. It is proposed that peak response be calculated elastically with reference to a fundamental period of the structure evaluated by taking for all members their respective secant stiffness at yield. This is to say that the method assumes equality of maximum elastic and inelastic displacements. The limits of this approximation are well known, together with the main elements having an influence on them: i.e., period of the structure, height of the building, amount of the inelastic displacements, shape of the F-Δ relationship and soil conditions being among the major ones. Correction factors depending on the elements above are available in the literature (see for ex. [Miranda, 1993]) to improve the approximation, if desired.

Consistently with the overall approximation of the method, it is suggested to evaluate the maximum displacement using a static set of linearly distributed lateral forces. The same set of forces, with arbitrary magnitude, can be used to calculate the period by means of the Raileigh quotient.

Given the period and the pertinent 5% damped elastic displacement spectrum, a displacement value: S_d, is obtained. This response is treated as the generalized response of a single mode structure whose shape is proportional to the displacement pattern calculated under the lateral forces of arbitrary magnitude. Accordingly, the displacements u_i from the arbitrary set of forces must be multiplied by the factor: $S_d\,(\Sigma\, m_i\, u_i)/(\Sigma\, m_i\, u_i^2)$ to obtain the correct values.

c) *estimation of member chord rotation capacity*. With the peak values of the expected deformations throughout the structure known, they now must be compared with the corresponding available capacities. This is the most original though difficult part of the method, one for which additional work is admittedly necessary.

With reference to linear frame members, capacity is expressed in terms of end chord rotation, i.e. the ultimate rotation at one end of an element in nearly antisymmetric bending. This capacity depends on the geometric characteristics of the element, its longitudinal and transverse reinforcement, nature of imposed loads and deformations, etc. Recognizing the limits of the present approaches in terms of the product of the ultimate curvature by an ill-defined plastic hinge length, the authors propose a purely empirical alternative, based on the results of tests in elements and subassemblages.

The variables considered for the regression expression for θ_u are: the shear span ratio l/h, the depth h of the member, the concrete strength f'_c, the mechanical reinforcement ratios of the tension, compression and confining steel: ω_1, ω_2 and ω_3, the ratio of the longitudinal bar diameter ϕ_L to the stirrup spacing s, the axial load ratio ν, and the ductility characteristics of the steel. A total of 320 tests have been used to derive an expression linking the variables above with θ_u. Figure 13 compares the experimental values with those predicted by the regression expression, in terms of average and low characteristic values (the latter being simply taken as: 0,4 θ_u).

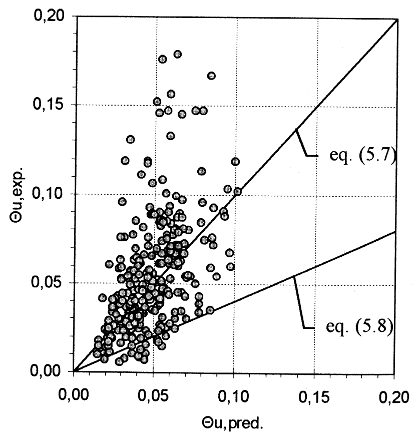

Figure 13 Comparison of experimental results with predicted values

The method just described has been applied to the design of a four story structure that has been tested at full scale at the ELSA reaction wall facility of the European Commission in Ispra. Calculated periods and displacements showed good agreement with experiments.

The obvious advantage of the final step of the procedure is the possibility of checking/improving the detailing arrangement of elements taking into account of the interacting effects of the relevant parameters on the ultimate deformation capacity, as opposed to present practice were provisions of partly analytical and partly of empirical origin are used additively, ending up frequently with an unnecessary amount of detailing.

CONCLUSIONS

The subject of seismic design in L/M risk zones has indeed many facets, only a few of them having being dealt with to some extent in this overview. According to logic, not to their importance, the main facets can be listed in the following order: state of the knowledge on seismic hazard in all parts of the world, including those parts to which until now scientific attention has been scarce; diffusion of this knowledge among the society, the decision makers, the profession, and awareness of the expected losses in case of future earthquakes; existence of the socio–techno–economic conditions for ensuring adequate quality in the construction process, whatever the nature of the actions to be resisted; availability of rational and effective seismic design codes.

Seismic hazard studies have made great progress in the last decade or so, and few places in the earth are left where the potential for the occurrence of earthquakes of various magnitudes has not been assessed. This applies to really low seismicity areas, were the maximum expected intensities are bounded by relatively modest levels, as well as to areas where geologists feel confident of estimating the potential for very large magnitudes, albeit characterized by equally large return periods. These areas are generally located in intraplate regions and their historical seismicity is not such as to provide a statistical support to estimates based on geological evidence: Central and Eastern parts of the US are well known examples of this type of seismicity. There seems to be a tendency in the US to consider, as a basis for the design of even ordinary structures, events characterized by return periods of 2500 years, so that even those rare large magnitudes would be included. This would be a purely political decision (since cost-benefit considerations loose their credibility for such a long temporal basis), and one that only a very advanced economy and well organized society could take today.

With the hazard known, the next step is to establish a threshold for ground motion intensity below which no special attention needs to be paid to earthquake action in design. A look to the solution adopted in a number of countries shows that the threshold depends on the overall hazard level and again on economic factors. Countries with a feeble seismic activity may choose, when they can afford to do so, to account for it in design even at a rather low levels, so as to minimize non structural damages, while where seismicity is higher and resources less abundant the lower threshold is meant to be an action for which ULS verifications start to be carried out.

Clearly, however, the question of the threshold cannot be separated from the basic aspect of the quality of the design and of the construction. A wealth of numerical simulations, as well as of factual evidence, exists to show that reinforced concrete buildings may have a substantial capacity to resist earthquake motions if they have a regular configuration and are correctly designed for gravity loads only. Among the studies reviewed in the paper, a recent investigation targeted at the type of seismicity of CEUS shows, with reference to a rather wide population of buildings designed for vertical loads only, that these would be capable of resisting real earthquake motions with PGA in the order of 0,25 g with drifts not exceeding 1,1%, a figure that one relates essentially to a moderate state of damage.

It would therefore appear that in regions having a low to moderate seismic activity, unless the concern is the protection from non structural damages, the question of the lower threshold for seismic design is almost irrelevant. Correct conceptual design (and hence practical guidance documents and rules for selecting appropriate structural types) and quality of the construction (and hence quality control exercised by the authorities) have a dominant influence on structural damages and on life-safety control, and quantitative seismic design provisions of purely analytical nature are no substitute for the two fundamental requisites above. However, if a figure should have to be indicated for a design PGA worth undertaking analytical seismic design for a damage LS, this figure could be, according to the information reported in the paper, anywhere in the range from 0,07 g and 0,10 g.

Once it is clear that a specific situation calls for full seismic design, the question could be raised about which one of the two design methods, the traditional FBD or the new and developing DBD is best suited to the needs specific to L/M risk zones. Unfortunately, for different reasons both methods do not offer their best in this situation.

Starting with the FBD, it is well known that its inherent limits aggravate when lateral actions are small in comparison with vertical loads. In this case the force-reduction factor R, which is intended to exploit the inelastic dissipation capacity of the structure, is of difficult calibration, since the total lateral resistance of the building is contributed already in good part (but variable from case to case) by the structure as designed for vertical loads. Especially for low rise buildings of 4-5 stories, FBD leads frequently to overdesigns, which is an economic penalty, and in any case to uneven levels of protection, which is an undesirable characteristic for a design code

DBD, on the other hand, at the stage of development where it presently is, is more effective in dealing with cases where ultimate deformation demand and capacity confront themselves, rather than with LS's describable in terms of strains, or of moderately inelastic drifts, which are more appropriate for L/M risk areas. One should note, however, that while the weaknesses of FBD are at least conceptually unremovable, DBD has cleaner theoretical bases, which will allow it, with time, to overcome its present limitations.

REFERENCES

Browning, J.P. (1998) "Proportioning of Earthquake-Resistant Reinforced Concrete Building Structures" Ph.Thesis, Purdue University

Calvi, G.M., and Kingsley, G.R. (1995) "Displacement-based Seismic Design of Multi-Degree-of-Freedom Bridge Structures". *Earthquake Engineering and Structural Dynamics,* Vol.24, pp. 1247-1266

Calvi, G.M. (1998) "Performance-based Approach for Seismic Assessment of Existing Structures" *Proc. 11 ECEE,* Vol.2, pp. 3-19, Paris September 1998, Balkema, Rotterdam NL

CEN TC 250/SC8 (1994) "*Eurocode 8: Design Provisions for Earthquake Resistance of Structures*", ENV 1998-1 Brussels

Elnashai, A.S. and Soliman, M.M. (1994) "Seismic Risk and Mitigation Planning in Egypt" *Engineering Seismology and Earthquake Engineering* Research Report N. 94/5, July, Imperial College, London.

Fajfar, P. (1992) "Equivalent Ductility Factors, Taking into Account Low-cycle Fatigue" *Earthquake Engineering and Structural Dynamics*, 21, pp. 837-848

Fardis, M.N. and Panagiotakos, T.B. (1997) "Displacement-based design of RC Buildings: Proposed Approach and Application" *Seismic Design Methodologies for the Next Generation of Codes*. Fajfar & Krawinkler Eds., Balkema, Rotterdam, NL

Fardis, M.N. and Panagiotakos, T.B. (1997) "Seismic Design and Response of Bare and Masonry Infilled Reinforced Concrete Buildings. Part I: Bare Structures". *Journal of Earthquake Engineering*, Vol.l, N.1, January, pp. 219-256

FEMA 303 (1998) "*NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*" 1997 Edition, February 1998.

Garcia, L.E. (1995) "Moderate and Low Seismic Risk Zones Requirements for Reinforced Concrete Buildings" ACI 1995 Fall Convention, Montreal, Quebec, Canada, November. Technical Session sponsored by ACI Committee 368.

GSHP (1993) "Global Seismic Hazard Assessment Program". *Annali di Geofisica*, Vol.XXXV, N.3-4, June-July.

Kowalski, M.J., Priestley, M.J.N. and MacRae, G.A. (1995) "Displacement-based Design of R/C Bridge Columns in Seismic Regions". *Earthquake Engineering and Structural Dynamics*, Vol.24, pp. 1623-1643

Hoffmann, G.W., Kunnath, S.K., Reinhorn, A.M. and Mander, J.B. (1992) "Gravity-Load-Designed Reinforced Concrete Buildings: Seismic Evaluation of Existing Construction and Detailing Strategies for Improved Seismic Resistance". National Center for Earthquake Engineering Research, State University of New York at Buffalo, Tech.Rep. 92-0016

IAEE International Association for Earthquake Engineering (1996) "Regulations for Seismic Design. A World List". Central Office, Tokyo

Mayer-Rosa, D. (1993) "Towards Uniform Earthquake Hazard Assessment in Europe". *Annali di Geofisica*, Vol.XXXVI, N. 3-4, June-July

Miranda, E. (1993) "Site-dependent Strength Reduction Factors". *Journal of Structural Engineering*, ASCE, 119, N.12 pp. 3503-3519

Moehle, J.P. (1996) "Dispacement-Based Seismic Design Criteria". *Proc. 11 WCEE*, Acapulco, June, Elsevier Science Ltd., Oxford, U.K., Paper N. 2125

NZS 4203 (1992) “Code of Practice for General Structural Design and Design Loadings for Buildings”. Wellington, New Zealand

Park, R. (1996) “Explicit Incorporation of Element and Structure Overstrength in the Design Process”. *Proc. 11 WCEE*, Acapulco, June, Elsevier Science Ltd., Oxford, U.K., Paper N.2130

Priestley, M.J.N. and Kowalski, M.J. (1999) “Displacement-based Seismic Design of Buildings”. *Bulletin, New Zealand National Society for Earthquake Engineering*, (in press).

Priestley, M.J.N. (1999) “Displacement-based Approaches to Rational Limit States Design of New Structures”. *Proc. 11 ECEE*, Vol.2, pp. 317-335, Paris, September 1998, Balkema, Rotterdam, NL

Pugliese, A., Romeo, R. and Sanò, T. (1997) “Seismic Hazard in Italy. Proposal for a new Seismic Zoning” (in italian). Report SSN/RT/97/2, Roma, Italy.

Riddell, R. (1995) “Inelastic Design Spectra Accounting for Soil Conditions”. *Earthquake Engineering and Soil Dynamics*, N.24, pp. 1491-1570

Sabetta, F. and Pugliese, A. (1996) “Estimation of Response Spectra and Simulation of Nonstationary Earthquake Ground Motion”. *Bull. of Seism. Soc.of Am.* N. 86-2 (1996) pp. 337-352

Seneviratna, G.D.P.K. and Krawinkler, H. (1996) “Modifications of Seismic Demands for MDOF Systems” *Proc. 11 WCEE*, Acapulco, June, Elsevier Science Ltd., Oxford, U.K., Paper N. 2129

SIA (1989) “Actions on Structures”. Swiss Society of Engineers and Architects, Zurich, Switzerland

Sokol, M. (1999) “Seismic Effecrts on Buildings in Slovakia according to New Codes” *Proc. 11 ECEE*, Vol.1, pp. 1-6, Paris, September 1998, Balkema, Rotterdam, NL

Tassios, T.P. (1999) “The Seismic Design: State of Practice” *Proc. 11 ECEE*, Vol. 2, pp. 255-267, Paris, September 1998, Balkema, Rotterdam, NL

Uniform Building Code (1988) “International Conference of Building Officials”. Whittier, California