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Some relevant aspects of the seismic design codes: Lessons learned from earthquakes and impact on practice and research

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ABSTRACT: The object of the present paper is to review and to discuss some central aspects of current seismic Codes in the light of the newest research acquirements and from the lessons learned after recent destructive earthquakes. First, a brief comment is presented on the basic importance which the control and supervision of the design, construction and maintenance has on the probability of failure of buildings. A selected number of topics related to EQ Resistant Design such as: LEDRS, Soft Soil Effects, EQ Inputs for Ultimate Limit States, Reduction or Modification Response Factors, Increase in Damping due to large deformation, IRS and Reduction for Ductility and the problem of Overstrengths, are discussed and illustrated through many examples.
Then, some concepts of the capacity design methodology are indicated and the opportunity of their aplication is outlined. In the conclusion a procedure for improving the EQ Resistant Design of building structures is suggested.

1 INTRODUCTORY REMARKS

It has already been emphasized that the seismic response and damage of a structure depends on the state of the whole building system soil-foundation-superstructure and non structural components when earthquake shaking occurs.

But a sound preliminary design and a reliable analysis are not sufficient to ensure a good seismic performance of structures during earthquakes. Strict Control and Inspection in the construction process are also needed, and later on a good maintenance is also very important.

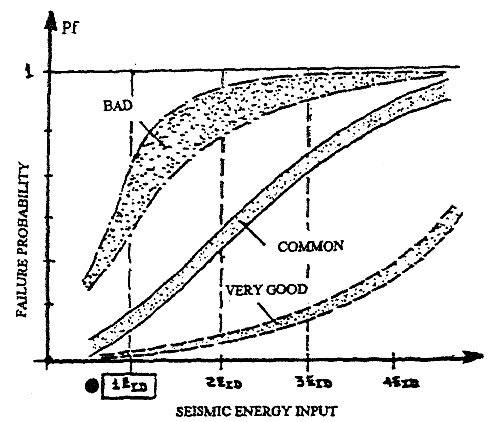
In fact the evaluation of observed damage during actual earthquakes points to a high vulnerability of buildings where the supervision of the construction process was ineffective or non existent.

Moreover alterations carried out during the lifetimes of the building have often resulted in unsatisfactory performances.

Although these two aspects have been recognized for some time, very little has been done to improve the practice so as to remove such negative effects (Bertero, 1986).

Another factor which has resulted in a disappointing structural behavior arises from building configuration which has proved to be ineffective, such as soft stories, short columns, sudden variations of strength and stiffness, and so on. Therefore a supervision of the engineering design process would be very appropriate in order to remove such inadequacies.

Fig.1 shows how the Probability of Failure varies with the Earthquake Energy Input E_I for buildings where the control on the previously mentioned factors (Configuration - Preliminary Design - Analysis - Construction and Maintenance) was respectively very good, according to current practice (common) and very poor. It may be seen that if the Earthquake Energy Input increases by a factor of 2 or 3, the probability of failure is still acceptable for very good practices. However, even for the Design Energy Input the Probability of Failure is unacceptable for poor practices.



VERY GOOD: Sound Preliminary Design (Capacity Design) and Reliable Analyses. Configuration Effective. Supervision Engineering Design Process. Supervision and Inspection in Construction. Maintenance. BAD: Unrealistic Analyses. Configuration Irregular. Poor Construction and Inspection. Not Maintenance.

Fig. 1 - Design and Construction of Earthquake - Resistant Buildings

2 CRITICAL EARTHQUAKE INPUTS

The first and perhaps the most difficult step in the design process is the specification on the Design Earthquake. The Design EQ should be that ground motion (among all the possible earthquakes at the site) which conducts the structure to its critical response. However, generally, the application of this concept involves many difficulties because: (1) there are serious problems in establishing the basic characteristics of the ground motions which may occur at the building site. In many sites current predictions of the characteristic of critical earthquake input are crude. (2) the Design EQ depends on the Design Criteria and specifically on the Limit State controlling the design. The critical response of a structure will also vary with the different limit states that may control design such as: Serviceability, Damageability and Collapse. Nevertheless, generally, current Codes explicitly specify the Design EQ only for one limit state (collapse); while the remaining ones are only indirectly considered and may or may not be adequate.

Presently Design EQ are commonly specified through the Design Response Spectra. Each seismic Code uses its own way to arrive at the Design Seismic Forces.

Some, such as ATC 3-06, EUROCODE 8 (EC8), ARGENTINEAN CODE INPRES-CIRSOC 103 (IC 103), 1988 SEAOC, 1987 MEXICO CITY BUILDING CODE, etc, start from Linear Elastic Design Response Spectra (LEDRS) and allow for large reduction to account for Ductility, Overstrength and Increased Damping.

Other such as: the Italian Code, the Chilean Code NCh 433 and the old Argentinean Code "Concar 70" specify directly a reduced Design Spectrum and account for specific structural system with limited ductility through an amplification factor.

2.1 Linear Elastic Design Response Spectra

The LEDRS recommended by the Codes have been proved to be inadequate by recent seismic events because they were derived by statistical analyses based on limited data.

Because the number of strong motion accelerometers has drastically increased in the last twenty years and the information gained from the previous statistical base, *changes in the spectral values are required in the Code specifications.*

The most significant new samples of strong ground motions are provided by records from: 1985 Mendoza EQ (Argentina); 1985 Mexican EQ; 1985 Chilean EQ; 1986 San Salvador EQ; 1987 Whittier Narrows EQ (USA); 1988 Armenian EQ; 1989 Loma Prieta EQ; 1990 Southern Sicily EQ.

Many of these records of severe ground motions show that the Peak Ground Accelerations and the Linear Spectral Responses may reach much greater values than those previously determined.

The Loma Prieta EQ was the largest EQ that has occurred in the San Francisco Bay Area since the great EQ of 1906 (Housner et al., 1990).

The ordinates of the 5% damped LERS for several ground motions recorded on rock sites at Corralitos, Gilroy N°.1 and at U.S. Santa Cruz within 20 Km from the source reach values larger than those recommended by ATC 3-06 and EC8 for firm soils in regions of high seismic risk, in the period range from 0.1 sec to 1 sec (the extreme values are more than two times larger).

The instrument closest to the epicenter was at Corralitos, 7 Km, and perhaps just 1 Km from the fault. The recorded horizontal PGA was 0.64g and the vertical PGA was 0.47g. The maximum Spectral Ordinate was around 2g.

Relevant from a design point of view is the fact that the average spectral values obtained from those records exceed by up to thirty percent the Code design spectra for periods in the range from 0.2 to 0.6 sec. (Fig. 2). These observations are more relevant if it is realized that the Loma Prieta EQ is smaller than the maximum expected earthquake in the area.

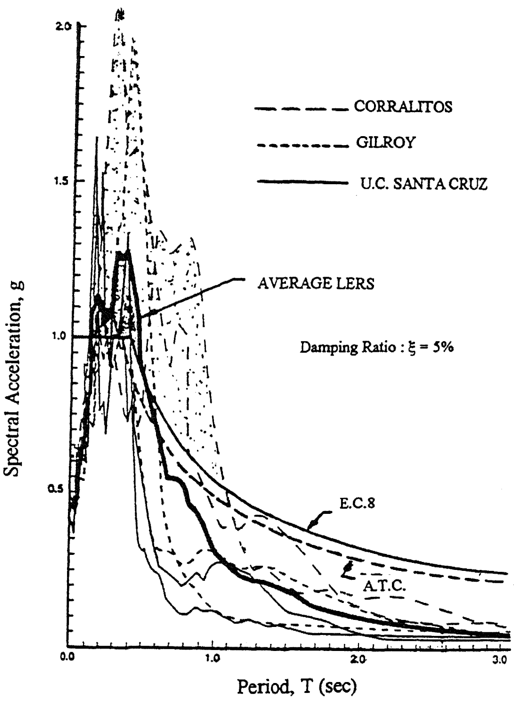


Fig. 2 - Comparison of LERS for motion recorded during Loma Prieta EQ with LEDRS recommended by EC8 and ATC 3-06 (firm soils)

Comparison of 5% damped LERS for N10E Component recorded at Llo Lleo during 1985 Chilean EQ with LEDRS recommended by EC8, ATC3-06 and 1988 SEAOC for soil type 1 (rock and stiff soil) and for the regions of the highest seismic risk, reveals that for periods less than 1.6 seconds the recommended spectra are significantly smaller than the values corresponding to the spectrum obtained from the N10E Component recorded at Llo Lleo.

The LERS recorded reaches values more than two times larger than those recommended. The maximum spectral ordinate is around 2.4g. This strong motion is considered to have the greatest damage potential of all the strong motions previously recorded or considered by any Code for rigid buildings located on stiff soil sites. Duration of strong motion, nearly 50 seconds, PGA: 0.67g (Saragoni, 1985).

For short period structures ($T \leq 0.7 \div 0.8$ sec) the 5% damped LERS for the strong motion with maximum intensity recorded during 1986 San Salvador EQ significantly exceeds the LEDRS recommended by ATC3-06, EC8, and 1988 SEAOC (Decanini et al., 1988).

2.2 Amplifications due to Soft Soil Site Conditions

The main characteristics of some of the ground motions recorded, the observed performance of buildings and recent research clearly demonstrate the basic importance of the site conditions (soil profile and topography) in establishing the design EQ and in the structural behavior.

The recorded PGA at the soft soil sites for the Loma Prieta EQ are significantly higher (2 to 3 times) than those on adjacent rock or stiff soil sites.

Records at stations located at the same distance from the rupture zone reveal substantial differences in the dynamic characteristics of the ground motion by effects of soil conditions. For example, the PGA was 0.12g at Foster City (stiff soil, epicentral distance 66 Km) and 0.28g at Redwood Shores (soft site, epicentral distance 63 Km), and significant variations in frequency characteristic of ground motions are revealed by spectral ordinates.

Soil factors were perhaps the most dominant feature in the Loma Prieta EQ. Most of the damage in buildings occurred on soft clay sites many kilometers away from the rupture zone.

During the 1985 EQ in Mexico City the ground motions recorded showed very different characteristics depending on the site condition. The PGA was greatly amplified (by a factor of more than 4) on the soft and deep deposits of an old buried lake.

Comparison of 5% damped LERS for EW component recorded at SCT with LEDRS recommended by EC8, ATC3-06 and 1988 SEAOC for soil type 3 (soft soil) and for the regions of the highest seismic risk, shows that for periods range from 1.7 sec to 3 sec the values recommended by Codes are significantly smaller than those recorded. For T = 2.0 sec the spectral ordinate of 5% damped LERS for EW SCT is around 2 times the recommended values by Codes.

On the other hand, the strong motion recorded at this soft soil had an extremely long duration (about 3 minutes of perceptible motion) with nearly 8 cycles of reversals exceeding 0.1g. This EQ has shown the possibility that structures can undergo a large number of yielding reversals with very high ductility demands.

Therefore, the results from the Mexico City records should be watchfully considered with regard to LEDRS for sites having soil profiles similar to that in Mexico City (soft clay profiles).

The case of Leninakan City during 1988 Armenian EQ is another clear example of the ground motion amplification due to soft soil site conditions.

No records of strong motion instruments were obtained at Leninakan. Unfortunately four stations of accelerographs were in a building that collapsed and no useful recordings could be salvaged. Other four stations produced only seismoscope and seismograph records.

The measurements indicate that Leninakan experienced a ground motion with a PGA around 0.40g. The motion due to the main shock consisted of about 50 sec of strong shaking followed by more than 60 sec of ground vibration owing to the response of the local soil structure (Wyllie et al. eds, 1989).

The duration of the motion was much larger than that observed in Ghoukasian and the low frequency components of the motion on the bedrock may have been amplified significantly because of the local soil response. These observations are confirmed by recordings of the aftershocks.

An analysis of measurement of relative ground response for Leninakan and the spectral ratios computed with respect of different rock sites indicated that ground motion in the period band 0.5 to 2.5 seconds were amplified significantly by the local geological setting.

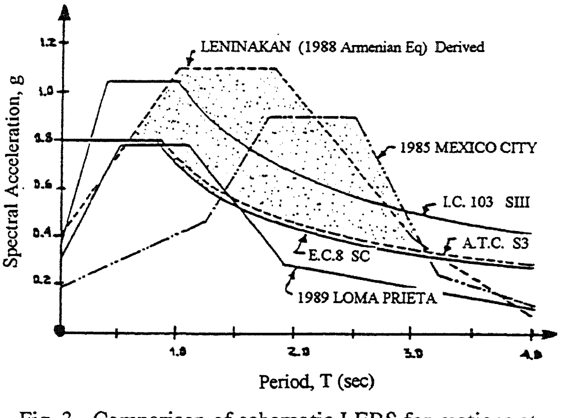


Fig. 3 - Comparison of schematic LERS for motions at soft soil sites with LEDRS recommended by EC8, ATC 3-06 and IC 103

On the basis of the observations made from recordings of aftershocks in Leninakan, the observed damages and the local geological condition, a schematic LERS (5% damping) has been deduced (Fig. 3) so to be representative of the response in Leninakan during the main shock.

In the same Fig. 3, schematic LERS have been represented for motions at soft soil sites, for the 1985 Mexican EQ and 1989 Loma Prieta EQ.

Comparison of these schematic LERS with LEDRS recommended by EC8, ATC3-06 and IC103 for soft soil and regions of highest seismic risk, shows significant difference.

In the light of these observations *new Code provisions seem to be required for soft soil sites.* Microzonation of urban areas seems to be a useful tool to be implemented in seismic Codes. Perhaps, an other way to minimize future earthquake damage may be a "strategic ground use".

3 DESIGN EQ FOR ULTIMATE LIMIT STATES (SAFETY AGAINST COLLAPSE AND DAMAGEABILITY)

It is necessary to identify what information is needed for reliable Design EQ to improve hazard reduction.

LERS does not represent a complete description of the ground motion damage potential in the case of inelastic behavior in which most of the input energy is dissipated through plastic deformations. The lessons learned from recent EQs and researches indicate that *Elastic Spectra Ordinates are not directly related to structural damage*. Extremely important factors such as the duration of the strong ground shaking and the sequence of acceleration pulses are not taken into account adequately by LERS. *The number of yielding reversals (NYR) and/or the number of yield events significantly affect the structural behavior.*

Among all different intensity parameters proposed for defining the damage potential, perhaps the most promising is Earthquake Energy Input (E_I) that was examined by Bertero and Uang (1988). This parameter considers the inelastic behavior of a structural system and depends on the dynamic features of both the strong motion and the structure.

The importance of and the need for considering additional information for the establishment of Design EQ, is well illustrated by comparison of the LERS (5% damping) and the spectrum for E_I (ductility ratio u = 2) for the 1986 San Salvador EQ and 1985 Mexican EQ, that are shown in Fig. 4.

The San Salvador record (CIG-H2) exhibits a PGA of 0.71g and a strong phase duration of around 4 seconds. On the other hand, the Mexican record (SCT-EW) shows a PGA of 0.17g and a very large duration.

LERS shows high spectral values in different period bands with larger values (maximum around 1.8g) for the San Salvador EQ at lower periods, and larger values (maximum around 1g) for the Mexican EQ especially in the range of long periods. However, a considerable different result is obtained when the values of E_I are examined.

From the above comparison, it is clear that the damage potential of the Mexican EQ is several times (about 10 for respectively dominant frequencies) greater than that of the San Salvador EQ in spite of the fact that maximum elastic spectral ordinates for dominant frequencies are larger for the San Salvador EQ.

Comparison of the amount and type of damages observed in both EQs are consistent with the last observation. *Earthquake Energy Input reflects clearly the effects of the duration of the strong ground motion.*

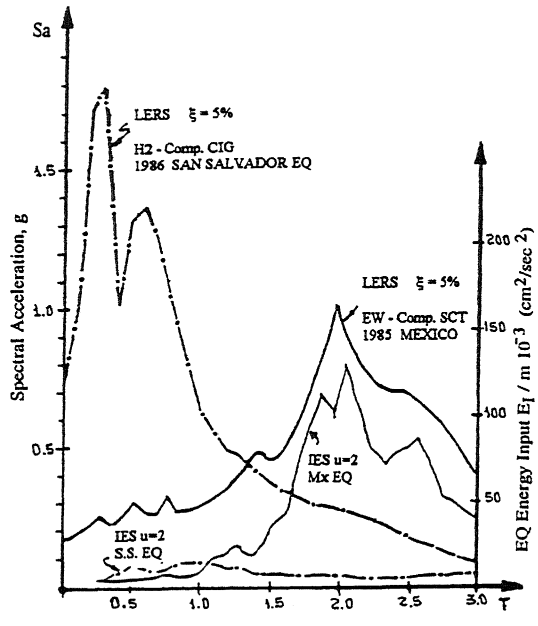


Fig. 4 - Comparison of LERS and Input Energy Espectra. 1985 Mexico EQ vs. 1986 S. Salvador EQ

The Inelastic Response Spectra (IRS) obtained directly from strong motion records give another picture of the damage potential. The informations provided by IRS are necessary for the design at safety level, but they are not sufficient because they do not give a precise description of the quantity of the energy that will be dissipated through hysteretic behavior; they give only the value of maximum ductility requirement.

An example of the shortcomings of the LERS and IRS, is provided by the comparison of two records of strong motions from Argentinean EQs (Decanini et al., 1986). The first one is a low duration high peak accelerogram recorded at Las Heras (Epicentral distance 35 Km) during the 1985 Mendoza EQ (M_L = 5.7). The second is a long duration strong motion recorded at San Juan (Epicentral distance around 65 Km) during the severe 1977 Caucete EQ (M_S = 7.4).

Both accelerograms have clearly different characteristics. The Las Heras record shows a pronounced peak of 0.41g and a strong phase duration no greater than 2 seconds. In spite of the high peak acceleration recorded, the building where the instrument was installed, suffered only slight non structural damage. The San Juan record presents a duration greater than 20 seconds and the damage produced was considerably higher than that caused by Mendoza EQ.

Both Elastic and Inelastic Response Spectra show larger demands for the Las Heras record. IRS for 5% damping and EPP behavior computed for different values of the ductility factor indicates the spectral ordinates of Las Heras higher than that of San Juan in the period range of 0.05÷1 seconds. The observed damage is not reflected by these results.

From analyses of the values of PGA, Maximum Elastic Spectral Ordinates and required yielding strength coefficient C_y (given in Table 1), it might be concluded that the requirements of Las Heras record are larger than those of San Juan.

However, *a very different conclusion is obtained when the values of the number of yielding reversals and the number of yield events are considered.* Much larger NYR and NYE are shown by the San Juan record as compared to the Las Heras one. This result is in good agreement with the amount and type of observed damages.

Table 1 - Las Heras vs. San Juan Records Argentinean Earthquakes

Record		Las Heras Transv. Comp.	San Juan E-W Comp.
PGA	(g)	0,41	0,19
PGV	(cm/sec)	26,98	20,60
tD	(sec)	2,0	22,0
MM intensity at the site		VI	VII
Arias intensity I_A	(m/sec)	0,90	1,33
Maximum spectral ordinate (g) 5% damping		1,70	0,70
Maximum required	u = 2	0,73	0,38
yield strength	u = 4	0,44	0,25
coeff. C_y	u = 6	0,38	0,18
Number of yielding reversal NYR u = 4		6	45
Number of yield events NYE u = 4		11	98

4 THE REDUCTION OR MODIFICATION RESPONSE FACTOR

In their provisions many Codes (EC8; ATC 3-06; IC 103; 1988 SEAOC Mexico City Code; etc.) have adopted a simple method for obtaining IDRS directly from the LEDRS using a global reduction factor, which, for a certain structural system, may be a constant value independent of T (EC8; ATC 3-06; 1988 SEAOC) or may be dependent on T at the lower periods (IC 103; Mexico City Code).

The reliability of this approach for the design at safety level sometimes is questionable (Bertero, 1986). It is not general, but it can only be applied to particular structural systems. If Codes maintain this procedure it will be necessary to indicate clear limitations on the type of structures that can be adopted and to give adequate recommendations in the design of the critical regions of the structural members and their connections, for large ductility and stable hysteretic behavior.

At the present, the Design Seismic Forces specified by the Codes are drastically lower than those (LERS 5% damping) obtained from strong motions recorded at different sites during recent EQs. The reductions of the design values were only justified by the "Ductility", "Overstrength" and "Increase in Damping due to Plastic Deformation".

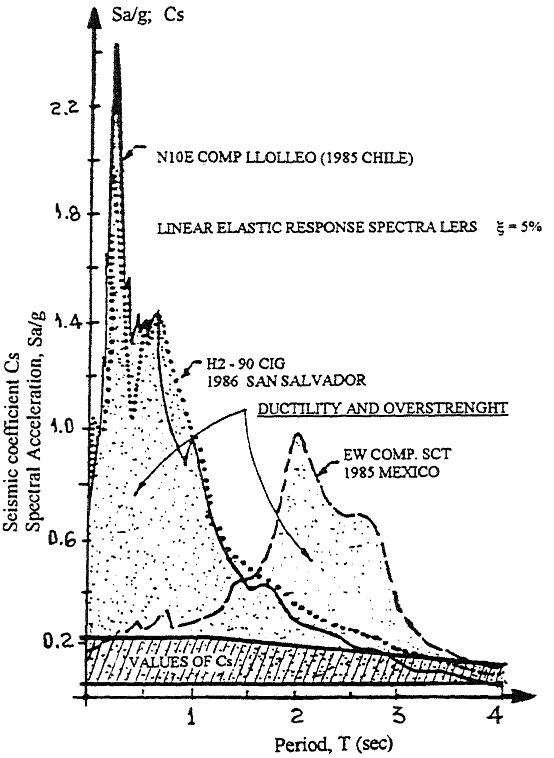


Fig. 5 - Comparison of LERS for severe ground motions with Seismic Coefficient C_s

Fig. 5 shows the differences between the LERS of three EQ in Latin America (1985 Chilean EQ, 1985 Mexican EQ and 1986 San Salvador EQ) and the range of variation of the Design Seismic Coefficient specified by many seismic Codes in the world. It may be seen that the maximum spectral values from those EQs are between 7 to 20 times larger than the corresponding Codes values. Overstrength and Ductility should make up for the differences.

The Reduction or Modification Response Factors, R, which are recommended by many current seismic regulations to obtain the IDRS directly from the LERDS *are essentially empirical values.* Generally, codes show a lack of any quantitative explanation about R or how it is derived.

Though considerable analytical and experimental researches have been devoted to its evaluation and application, *even today R continues to need an explanation and quantification.*

In certain cases the presently recommended values for R may be too high, together with the fact that LEDRS presently adopted by Codes might be a underestimated representation of the intensity and response of the major strong ground motion that can occur at a specific site, and the resulting design will not be reliable.

If overstrength should not be present the actual behavior of structures would be much worse than the observed one. In fact if the demand is too large the sole reduction for ductility would not be sufficient, OVS does allow for the difference.

For assessing a more precise definition of R, it may be interpreted as a combination of various factors affecting the actual response (Bertero, 1988). Then it is necessary to evaluate and to calibrate each of these factors as accurately as possible.

The first of such factors is connected to the increase in damping due to large deformation and is termed R_d. The second arises from ductile behavior (dissipation of energy through inelastic deformation) and is indicated R_u. While the third is connected to the Overstrength (OVS) and is termed R_s. So it is possible to express R as the product:

$$R = R_d \cdot R_u \cdot R_s$$

4.1 Damping Effects

LERDS currently are specified for 5% damping if the building is subjected to very strong ground motion with large deformation over initial yield; in certain cases structural damping may be greater than 5%.

Records of moderate and major EQs obtained at different levels of the buildings (USA, Japan, S. Salvador) and test results from the experiments and associated analytical studies conducted on scale model on shaking table (Bertero, 1986) clearly indicate an increase of the critical damping ratio with the accumulation of damage (cracking, yielding, crushing, etc.) during EQ response of the R/C buildings.

In the case of moderate intensity of shaking the equivalent viscous damping ratio ranges between 3% and 5%. For major EQ, greater values have been recorded ranging between 5% and 10% for R/C buildings. Then, the reduction due to this cause cannot be very important (Meli, 1991).

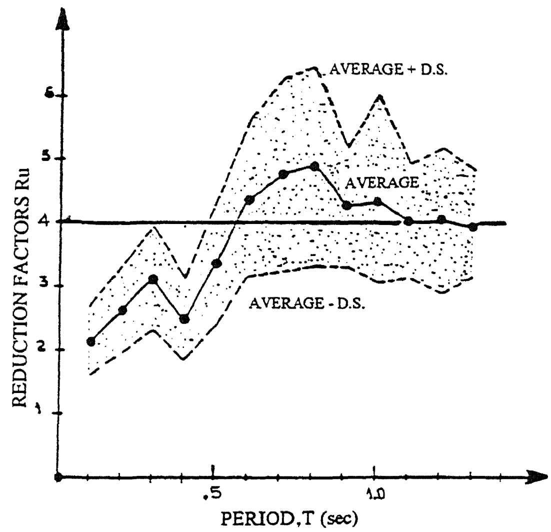
4.2 Some Remarks on the Reduction Factor for Ductility

The first step for an improvement of the values of R_u is to analyze the IRS of structures sujected to available recorded or expected critical ground motions.

There are numerous studies on the IRS, but here only some relevant aspects are pointed out.

IRS for a specific ductility ratio gives the spectral ordinates for which a SDOFS should have been designed to obtain the maximum available ductility when subjected to a given ground shaking. The ratio of ordinates of elastic to inelastic spectra provides the values of actual R_u for a given value of the ductility factor.

Fig. 6 shows the average Ductility Reduction Factor derived from two components of three different records (Vina del Mar, Valparaiso El Almendral and Valparaiso UTFSM) of the 1985 Chilean EQ for Ductility Displacement Ratio of 4 and EPP Model (Bonelli, 1986). A large scattering may be observed over the whole range of periods.

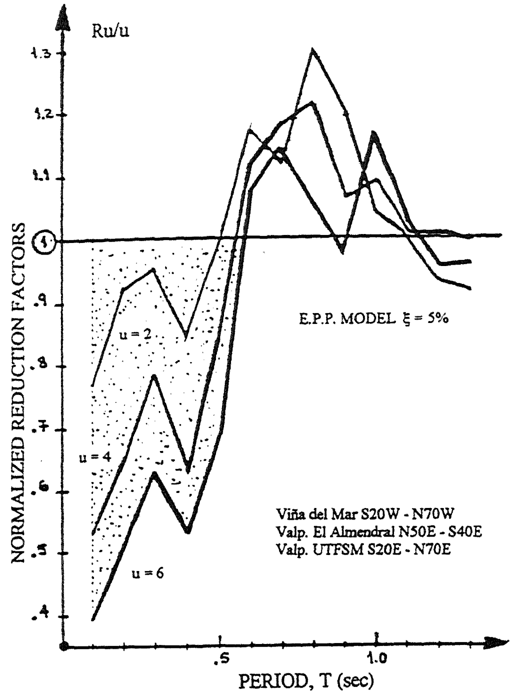


* Vina del Mar	S20W	-	N70W	PGA = 0.36g
* El Almendral	N50E	-	S40E	PGA = 0.29g
* U.T.F.S.M.	S20E	-	N70E	PGA = 0.18g

Fig. 6 - Scatter of R_u for selected strong ground motions recorded during 1985 Chilean EQ

Another representation for three different ductility ratios is given in Fig. 7, where the variation with period and with ductility itself may be clearly observed. *Therefore it is not clear why most of the current Codes do not account for variation of the R_u with the period.*

While the reduction equal to ductility ratio may be acceptable in the case of structures with a period similar or greater than the period of the predominant frequency contents of the earthquake ground motion, the well-known reduction by SQRT (2u-1) cannot be adequate in most cases of the structures subjected to EQ. The reduction should be smaller for systems with fundamental period lower than the dominant period of



* For $T \leq 0.5$ sec the reduction factor can be 1/2 of the reduction factor for $T > 0.6$ sec, depending on the value of ductility displacement

Fig. 7 - Normalized reduction factor for selected strong ground motion recorded during the 1985 Chilean EQ

the shaking. This fact is partially recognized only by the Argentinean Regulations IC 103 and by the Mexico City Code.

Soil condition play an important role in the dependance of R_u *from period:* for example, it can be seen from the Reduction Factor for Ductility drawn by Meli and Avila (1989) for the record at SCT obtained during 1985 Mexican EQ.

For narrow-band strong motions, as SCT, ductility reductions are more pronounced than for wide-band earthquakes, for periods close to that for which the peaks of elastic spectra ordinates occur. IRS of SCT record corresponding to EPP model, indicates that for long periods extremely larger reduction takes place and the high peak of elastic spectra ordinates tends to disappear for a ductility factor of 4. On the contrary, for periods lower than around 1.5 seconds, the reduction is smaller than for common wide-band shakings, which are typical of strong ground motion of firm soils. For these periods non-linear ductile behavior has only slight effect in reducing the elastic spectral ordinates.

For stiffness degrading behavior little differences are found with respect to EPP system.

However, if the inelastic response is characterized by a strength degrading behavior (whose yielding branch has a negative slope of 10% of the initial stiffness) the corresponding reduction factors are significantly smaller, and for periods lower than 2.3 seconds, R_u is less than u.

Moreover, the trend above mentioned has been observed in other records obtained on soft soils.

With respect to the aforementioned results, it is possible to remark: (a) the importance of soil condition, (b) the influence of the kind of inelastic behavior, (c) the quantity of energy that may be dissipated by non linear deformations *is seriously limited by the strength degrading.*

Accordingly to the above remarks on the effect of soil condition, *a possible improved procedure for the evaluation of* R_u *is as follows*: for periods larger than that where the peak elastic spectral acceleration is attained, R_u is assumed constant and equal to the ductility factor u, while for lower periods the variation between one ($T = 0$) and the ductility factor u is linear or less than linear.

For the practical purpose the peak of elastic spectral acceleration may be assumed in correspondence with the characteristic site soil period T_s, but not less than 0.5 seconds for soft soil deposits.

On the basis of all the information available (observed performances, structures and subassemblages tested in the laboratory, and associated analytical studies) *it seems that the maximum reduction associated to ductility is about 4 for well-designed R/C structures.* But a moderate decrease of the R values does not justify any relax in ductile detailing, because ductility demands during a strong ground motion may differ considerably from values anticipated by Codes.

4.3 Remarks on overstrength

Presently, the evaluation of the actual strength of the whole soil-foundation and superstructure system (including non structural elements), perhaps is one of the most important problems concerning the improvement of the Reduction Factors. *Overstrength is a key factor for the reduction of actual LERS.*

The actual strength of a building depends not only on the strength capacity of its bare superstructure, but also on the extra capacity that arises from the presence of the non structural elements. If the effects of partitions or walls infilling frames is neglected, the prediction of the stiffness and strength will be significantly inaccurate and unexpected brittle failure modes will take place.

Moreover the interaction of the whole superstructure system and the soil may have certain effect in the actual resistance of the building.

The actual strength of buildings is the result of a complex combinations of many factors. The specified code seismic forces used in design are *only one* of these factors.

There are numerous and different sources of OVS, the main may be grouped as follows:

a) Design Procedures:

* Safety Factors (load factors and strength reduction factors).
* Simplified and conservative models used for analysis and design (3D Effects; Contribution of the slab to the flexural strength; Conservatism in equations and empirical formulae for sizing and detailing of the structural elements; etc).
* Load combination not involving seismic forces.
* Interaction soil-structure.
* Minimum requirement for sizing and reinforcement.

b) Code Material Requirements:
* Difference between the actual strength of material and its nominal values.

c) Positive effects of Non Structural Elements:
* Presence of Infills, Partitions, Claddings, etc.

d) Overdesign:
* Size of structural members conditioned by the practice of construction and/or by the architectural functions.
* Designer tendency to uniform the dimensions of members and round up size and bars diameter.

Moreover, the construction technology (quality control of materials, inspection, workmanship, maintenance) has a significant influence on the EQ resistance of buildings.

As a consequence of the complexity of the actual strength of buildings it is clear that seismic resistance cannot be considerably improved merely by enhancing the specified design seismic forces.

It has been indicated that current design procedures generally produce structures that are stronger than minimum code requirement. For a well designed and constructed structure, the actual strength generally results 200% to 300% higher than the minimum code-specified yield resistance.

The large OVS beyond the first effective yielding resulting in experimentally tested structures (Bertero, 1986; Shahrooz & Moehle, 1990) corroborates the tendency of current design procedures to generate structures that are stronger than the minimum yield strength required by the Codes. In many model structures tested, the ratio between the seismic coefficient corresponding to the actual strength and the code specified seismic coefficient has been variable and comprised roughly between 2.5 and 6.

However it is necessary to point out that such values obtained in models cannot be merely applied to real situations and that certain structures can not have OVS of the same order as those experimental.

Data recorded on buildings during severe seismic events give another reliable way for evaluating the actual strength and for calibrating the OVS (Decanini & Giuffrè, 1992). Furthermore the observed performances and associated analytical studies constitute a good source of information.

An example is provided by the performance of the El Camino Real Hotel observed during 1986 San Salvador EQ. It is an eigth-story R/C frame building that was struck by a shaking of high acceleration but with short duration, suffering only minor non structural damage (Decanini et al., 1988; Zurita & Meli, 1991).

The design seismic coefficient was 0.12, the maximum acceleration recorded at the roof was 0.91g. It is estimated that the EQ should have induced in the structure lateral forces equivalent to a seismic coefficient of 0.42. The ratio between the seismic coefficient derived from the records and the design seismic coefficient, results around 3.5.

In other various cases of buildings subjected to strong ground motion which have been affected only by minor damages, values of the seismic coefficients derived from the records several times greater that those required by the codes have been found.

Another example is constituted by the very good performance of the majority of stiff shear walls structures in Vina del Mar, Chile, during the severe EQ occurred on 3 March 1985.

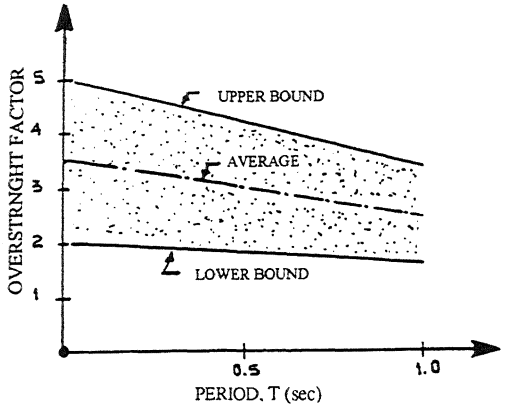
The basic cause of this excellent observed behavior is the type of structural layout and structural system which consists in a dense array of shear walls. The ratio of shear walls area to floor area has a average value of 3.5% in each direction. It is found that constructions observed in Vina del Mar generally possessed strength in excess of the Chilean Seismic Codes.

Analysis and comparison of all available information indicates that *the actual OVS may have strong variation in different buildings and for different EQ ground motions*. From these facts one of the most significant difficulties arises on establishing reliable values of the Reduction Factors R. Furthermore there is an urgent need to improve the knowledge and calibration of the possible OVS of structures when subjected to critical EQ.

Estimation of OVS Factors carried out for the San Juan City, Argentina, indicates that for actual buildings these value are in average range between 3.5 for lower periods and 2.7 for periods close to 1 sec, as illustrated in Fig. 8. In addition, a large scattering may be observed over the whole examined range of periods. The reference Code is CONCAR 70 (previous Argentinean Seismic Regulation) which specified directly a reduced design spectrum with some magnification factors for structures with limited ductility and working allowable stress method for sizing.

During the 1977 Caucete EQ, a very good seismic performance was generally observed of the buildings which were designed according to the Code and constructed with accurate practice. Some of the main reasons for this behavior were: strict inspection and control, good workmanship and adequate structural engineering practice (overdesign, dense and resistant infills, stiffer structural system, confined masonry, etc.). Moreover, in San Juan City EQ safety is a priority for all public and private buildings and there is a good degree of preparedness.

Finally, it seems opportune to add some last considerations about the characteristics and distribution of OVS. Naturally, the larger lateral load capacity due to OVS carries with it the advantage that the required



$$* \text{ OVS Factor} = \frac{\text{ACTUAL STRENGTH}}{\text{MINIMUM CODE STRENGTH}}$$

Fig. 8 - Estimated Overstrength Factors in San Juan City, Argentina

global ductility may be reduced. However, if the actual structure have only higher axial-flexural OVS a brittle failure mode may occur (shear, buckling, etc.). For instance, the shear forces developed at the critical regions when yielding flexural capacity is attained may overcome the available shear capacity. *To avoid this, it is necessary to base the design against shear on the maximum axial-flexural resistance, evaluating probable OVS carefully.* Furthermore, it is well recognized that any lack of continuity of OVS along the height of the structural system may create serious problems of concentrated energy dissipation.

Observations regarding the OVS may be summarized as follow:

* Available OVS varies widely with: design procedures, structural systems and characteristic of critical ground motions.

At the present it is not possible to provide a general and reliable approach which may accurately predict the effective contribution of OVS (R_S) to the Reduction Factor R.

This problem requires urgently extensive and integrated analytical and experimental studies; and then to formulate their results in an easily understandable way that could be used in practice.

* There are important changes in building technology and certain tendencies of designers that will possibly produce in the future buildings with very little OVS beyond the minimum codes required resistance. If Reduction Factor should be relayed only on ductility built-up in the structures, the recommended values of R in certain current Codes seem much higher and the behavior of the resulting buildings when subjected to strong ground motion will be unsatisfactory.

* The seismic analysis of the bare structures are not sufficient for a reliable prediction of the EQ resistance. *It is necessary to include realistically in the model the effects of non structural elements (partitions, exterior walls, cladding, etc.).*

5 CAPACITY DESIGN

In many sites significant uncertainties are involved in the assessment of the seismic hazard. As a consequence, the prediction of the characteristics of critical design EQ are still very rough. Certainly it is necessary to remove these uncertainties but the available informations needed to improve the establishment of EQ input often are insufficient.

If it is not possible to define a precise and complete description of ground motions at the site, it seems appropriate to select a reasonable level of resistance to lateral forces (without jeopardizing the requirements of damage control) and then to ensure that the structural system may support considerable variations in the ductility demands without appreciable degrading of its strength. This concept is based on the fact that one of the most problematic aspects of the seismic response is the amount of inelastic deformation which may be attained when the structure is subjected to strong shaking.

Hence, the critical (plastic) regions of the structural members and their connections should be designed, and particularly detailed, with high ductility and stable hysteretic behavior; furthermore shear failure should be avoided through an appropriate strength differential.

The authors believe that the aforementioned conditions can be reached through the procedure of *Capacity Design*.

The philosophy of capacity design, developed primarily in New Zealand by Paulay and Park (1975, 1979), is well known.

At the present in New Zealand the seismic Code is based on a *Capacity Design* procedure. The Argentinean Seismic Regulation IC 103, since 1983 also have introduced some concepts and applications of capacity design principles (the design and detail of critical regions of structures is based on the probable supplied strength of members, and dynamic magnification factors are considered when equivalent static analysis is used).

With modifications, the strategy of capacity design has been adopted also by EC8 but certain aspects require improvement (i.e. dynamic effects on columns).

When a suitable and enforceable strength hierarchy between the different elements of the structure is ensured and the critical regions are adequately designed and detailed, the resulting structures (specially structural walls system and hybrid system) will be tolerant to strong ground motion in terms of inelastic deformations (Paulay, 1988, 1991).

6 IMPROVEMENT OF DESIGN PROCEDURE

As conclusion of the concepts and aspects discussed and identified in this paper it is possible to indicate that the current codes procedures for Design of Seismic Resistant Buildings could be improved in the following way:

First - Preliminary Design should be based on more realistic LEDRS and calibrated values of the Reduction Factors R for the derivation of IDRS and on the application of the principles and procedures of *Capacity Design* in a complete and rational way. Improved values of the dynamic magnification factors should be used.

By a choice of a reasonable forces level, *Capacity Design* will account for uncertainties on ground motion input. Good detailing and ductility provisions will allow for large inelastic deformations. Therefore a structural design tolerant to ground motion variations will result.

Second - Once a sound Preliminary Design has been completed, the expected maximum strength must be checked through a Non Linear Mechanism Analysis (Limit Analysis) for several feasible force distributions.

Hence the value for R previously adopted may be controlled with respect to available OVS and Displacement Ductility Ratio. An adequate value of OVS (depending on the R selected, structural type and fundamental period T) should be ensured.

Further, through Limit Analysis it is possible to check if the structural system may develop its maximum potential strength (summation of full resistance of all components) and if then it may move to a mechanism dissipating a large amount of energy.

The indicated procedure should be applied only to standard buildings because only for them calibration factors have been derived.

For buildings which cannot be classified as "standard", a 3D Non Linear Dynamic Analysis should be used for several feasible ground motion inputs.

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