CODE-PRESCRIBED SEISMIC ACTIONS AND OBSERVED PERFORMANCE OF BUILDINGS

R. Meli [1]

ABSTRACT

The inconsistency between the code-prescribed seismic actions and the forces presumably induced in buildings by the ground motions recorded during severe earthquakes is pointed out. The influence of ductility and overstrength in the performance of buildings is discussed in general terms and through the analysis of the response of two buildings affected by strong earthquakes: one in San Salvador and the other in Mexico City. The capacity available in excess of that assumed in the design was significantly different for the two cases. Possible reasons for these differences are discussed and the need for a more rational procedure to derive the seismic actions for the design of buildings is emphasized.

GROUND MOTION RECORDS AND DESIGN SPECTRA

The earliest requirements for seismic design were limited to the specification of a base shear coefficient, i.e., the fraction of its own weight that a building should be able to resist as a lateral load distributed along its height. This coefficient was typically set at 0.10 for zones of high seismic risk. Later, the codes have evolved toward more sophisticated requirements and formats; nevertheless the base shear coefficients resulting from their application have remained similar to those initially stipulated. The lateral capacity required for the structure is based more directly on the observed performance of buildings with different characteristics during severe earthquakes, than on a rational derivation from the theoretical response of structural models to ground motions corresponding to extreme events. Modern regulations are aimed at obtaining safer structures by precluding features that have proved to originate inadequate behavior, such as irregularities in the structural system and structural detailing that leads to non-ductile behavior. Most recent codes do not place the emphasis on achieving a great capacity to lateral loads.

Nowadays, design response spectra constitute the most common way for specifying earthquake actions. Theoretically, they should represent smoothed envelopes of the most unfavorable ground motions that could affect the site, computed for linear systems with 5% damping. Actually, the design spectra are drastically lower than those computed from strong motions recorded at sites wich have the levels of seismic hazard considered by the codes. The reductions of design ordinates were formerly justified by the "ductility" of the structure. Broadly, the reduction for ductility imply that part of the energy induced by the earthquake can be dissipated by non linear behavior of the structure that is not required to remain linearly elastic for exceptional events such as the design earthquake.

[1] Research Professor, Institute of Engineering, National University of Mexico, Director for Research, National Center for Disaster Prevention, Mexico City

As the number of strong motion instruments has dramatically increased in the last two decades, records of severe events have become available, showing that the ground accelerations, and the corresponding spectral ordinates, can reach values much greater than those assumed as possible. On the other hand,in many instances, buildings designed for resistance much lower than that theoretically necessary to withstand the recorded ground motions, have showed an acceptable performance under such earthquakes, thus inducing code writers not to increase the design seismic forces to the extent that would have been required according to the strong motion records.

Design codes of different countries use distinct ways to arrive at the design base-shear coefficient for the computation of the equivalent lateral forces that buildings are required to withstand. Some of them start from linear elastic spectra with rather high ordinates, from where very large reduction factors are allowed (between 5 and 12 depending on the ductility of the structure for the UBC-SEAOC codes, widely used in the United States). Other codes start from "linear elastic" spectra that have been already significantly reduced from those of the maximum expected events. Moderate reductions factors are then applied to the spectral ordinates depending on the available ductility of the structure (Mexico City new building code specifies a maximum reduction factor of four). Other codes specify directly a reduced design spectrum (or base shear coefficient) with some magnification factors for structures with limited ductility.

Some examples that illustrate the difference between response spectra of actual strong motions and design spectra specified by the codes for the same area and type of soil where the records were obtained, are shown in Figure 1a. All response spectra assume linear behavior and five percent amping. Design spectra have been reduced as allowed by the codes for well-detailed ductile concrete frames. In Fig. 1a, the spectrum for the record of the Loma Prieta earthquake (1989) obtained at Corralitos is compared with the design spectra specified by the UBC code for California. It must be pointed out that the magnitude of the Loma Prieta earthquake is well below the maximum that can be expected in the area. The maximun peak of the response spectrum of that record is more than 20 times greater than the design ordinate. In Fig. 1b, the SCT-EW record obtained in the lake-bed area of Mexico City, for the 1985 earthquake, is compared with the design spectrum of the 1987 code. For the critical period, a tenfold difference is shown. The Llolleo record of the 1985 Chilean earthquake is extraordinarily severe, its response spectrum reaching a peak of 2.4 g for a period of about 0.3 sec. The maximum ordinate of the design spectrum for this country is 0.1 g, as can be seen in Fig. 1c. Finally, the 1986 San Salvador earthquake was a local event of moderate magnitude (Ms = 5.4); nevertheless the response spectrum, shown in Fig. 1d, has a peak of about 2g for a 0.3 sec period. The maximum base shear coefficient for the area is 0.12.

Common to all the afore-mentioned cases is the extremely large difference between the ordinates of the response spectra of events recorded at the sites and those of the corresponding design spectra.

RATIONALE FOR THE REDUCTION FACTORS IN DESIGN SPECTRA

No quantitative explanation is given by the design codes for the large reduction factors implicitely or explicitely involved in the seismic actions they require to design for. As already mentioned, it was originally assumed that ductility alone could explain the whole reduction. More recently, at least two other factors have been identified: additional damping and overstrength (see commentaries to the UBC code, Ref. 1, and to the Mexico City Code, Ref. 2).The influence of the three factors will be discussed next.

a) **Ductility** Analytical and experimental studies have shown that very large ductility factors can be developed by well-detailed steel and concrete members (in the range from 10 to 20). Nevertheless, the amount of energy that can be dissipated by non-linear deformations of a structure as a whole, is limited by the deterioration of the structural behavior and by the concentration of ductility demands at some sections. The analysis of the behavior of full scale structures tested in shaking tables shows that the maximum reduction attributable to this factor is of about four, for well-detailed concrete and steel frame structures (Ref. 3). Furthermore, the amount of structural and non-structural damage in buildings that undergo large non-linear deformations is such that this kind of behavior should be allowed only for truly exceptional events with a low probability of ocurrence during the life span of the buildings. Therefore, a reduction factor of about of four seems to be the maximum that should be allowed for very ductile structures. The rèduction factor should be smaller for buildings with short fundamental periods of vibration, as specified for instance by the Mexico City Code (Ref. 2). For this range of periods, non-linear ductile behavior has limited effect reducing the required lateral capacity.

b) **Damping** Design spectra are typically derived from linear response spectra for 5% damping. The major source of damping in the response of buildings subjected to very strong ground motions is related to inelastic behavior and its effects are accounted for in the "ductility" reduction factor. Therefore, only internal, friction and radiation damping must be considered separately. Radiation damping could be very significant in the response of stiff, massive buildings on soft soils. The structural damping that has been computed from the response of buildings to earthquakes, varies with the level of shaking, the fundamental period of the building, its structural system and the amount and type of non-structural members. For moderate earthquakes, damping coefficients have been recorded between 2 and 3% (Ref. 4). For stronger ground motions, higher values have been measured, often between five and ten percent for common buildings. Therefore, the reduction that can be made due to this factor to the spectral ordinates corresponding to linear behavior and five percent damping, cannot be very significant.

c) **Overstrength** It has been widely recognized that a basic reason for the good performance of many buildings during severe earthquakes has been that their actual capacity was well above the minimum required by the code and the value assumed in their structural design. The main sources of overstrength can be grouped as follows:

I) Code provided safety factors. These are not only the explicit safety factors, as load factors and strength reduction factors. Other sources of conservatism are: the difference between the actual strength of materials and its nominal values, the difference between the actual capacity of structural members and that computed with design formulas wich involve some degree of conservatism; the overstrength provided by the minimum requirements regarding geometrical properties and reinforcement. All these safety factors are intentionally introduced by the code-writers to cover for variability and uncertainly involved in the design process.

II) Conservatism in the model. The idealized structure used for the analysis and design is a model that is simpler and usually weaker than the actual building. Several parts of the buildings wich are ignored in the model provide for additional strength: infills, cladding, lintels, covering, etc. The idealization of a building by linear, plane models often ignores favorable tridimensional effects. Typically, the contribution of the slab to the flexural strength of the beams in monolithic concrete floors is ignored. Modelling of the complex phenomenon that imposes actions to the building, through a simplified set of lateral forces, is another significant source of conservatism.

III) Overdesign. Aiming at simplifying the computations and at obtaining more practical solutions, designers tend to uniform the structural members, adopting the properties of the most critical sections, and to round up member dimensions and bar diameters. Furthermore, the properties of many members are governed by load combinations not involving earthquake forces, thus conducting to an earthquake resistance well above that required by the code, especially in low-rise buildings. Finally, the limit states considered in the design usually corresponds to the "first yielding" of the critical structural members. There can be a significant reserve of capacity regarding the loads needed to form a true mechanism of collapse of the buildings.

There has been a considerable interest in recent years for evaluating the amount of overstrength available in different types of buildings, through analytical computations and laboratory testing of model structures, and through the evaluation of actual buildings.

An interesting example of evaluation of the contribution of the different sources of overstrength has been made by Shahrooz and Moehle (Ref. 5), for a six-story concrete frame structure designed according to the UBC and ACI codes. The results are summarized in Table 1. It can be seen that a structure designed for a base-shear coefficient of 0.092 could theoretically resist 7.65 times as much. The main sources of overstrength were: the excess strength needed to resist gravity loads, the effect of the minimum requirements for the reinforcement, the difference between actual and nominal properties, and the contribution of the slabs. It should be mentioned that the model was tested in a shaking table and resisted maximum lateral forces corresponding to a base-shear coefficient of 0.68, versus the 0.092 specified by the code and the 0.706 of the theoretical maximum capacity.

In other model structures tested in the laboratory, the ratio between the actual capacity to lateral load and that corresponding to the code specified base shear coefficient has been widely variable, often attaining values of the same order as those in the former example. Nevertheless, it must be taken into account that these results cannot be directly extrapolated to actual buildings and that some structural systems could not count on levels of overstrength of the same order as those of these models.

The most reliable source of information for evaluating the actual contribution of the different factors that affect the earthquake resistance of buildings is constituted by the records of the seismic response of buildings subjected to severe earthquakes, so that the observed performance can be compared to the theoretical response. Two significant examples in this regard will be discussed in the following sections.

EVALUATION OF THE RESPONSE OF A HOTEL IN SAN SALVADOR

The El Camino Real Hotel is an eight-story reinforced concrete frame building that was subjected to a ground motion of very short duration but with high accelerations. The building has four symmetrical frames in the longitudinal direction with a drastic change of beam span between the first floor and the upper stories. A sketch of the structure is shown in Fig. 2. The seismic design was performed through a static analysis with a base-shear coefficient of 0.12. The structure was designed according to the 1967 ACI code, which did not include special provisions for ductile behavior. Good quality of the design and construction could be perceived. A large number of masonry partition walls existed especially in the upper stories, as well as many decorative masonry elements in the facade. All the masonry was carefulluy isolated from the concrete structure by a flexible joint 1 cm thick. The structure is founded, through isolated footings, on stiff deposits of silty sand.

The building was instrumented with three tridirectional strong motion instruments placed in the basement, first floor and roof. The building was struck by the October 10, 1986, San Salvador earthquake, suffering only minor non structural damage, as cracks in partitions and in facade walls.

The earthquake was of magnitude Ms = 5.4 with its epicenter at about 7 km from the site of the building. It produced great damage, especially in non-engineered low-rise constructions, but also in several modern concrete structures, including some collapses. For a description of the seismological aspects of the event see Ref. 6; for its consequences on the building structures see Ref. 7. A network of strong motion instruments was placed in the basements of about a dozen buildings spread throughout the city. The records showed maximum horizontal accelerations between 0.4 and 0.65 g. The record at the basement of the El Camino Real Hotel is representative of what obtained in other sites. As shown in Fig. 3, the record has only three cycles of severe motion. A malfunction of the instrument at the roof of this building did not allow to obtain a reliable record of the motion in the transverse direction. Therefore, the evaluation is limited to the response in the longitudinal direction. For a detailed description of the instrumentation and records, see Ref. 8. A complete report of the analyses performed on the El Camino Real building can be found in Ref. 9.

The maximum acceleration at the roof was 0.91 g, which is 2.67 times the maximum at the basement. The ratio of amplitudes of the Fourier spectra at the roof and at the basement is shown in Fig. 4. The transfer function allows the identification of the peaks corresponding to the first three natural modes as 0.9, 3.1 and 5.1 Hz respectively. The history of displacements at the roof obtained by integrating the accelerogram is shown in Fig. 5.

A tri-dimensional analysis of a linear model of the building was performed using conventional assumptions about structural properties and masses. The vibration periods of the model matched very closely those derived from the transfer function of their measured response. The history of lateral displacements in the longitudinal direction computed at roof level for the model subjected to the ground motion recorded at the basement, reproduced almost exactly that obtained from the record, see Fig. 5. On the other hand, a step-by-step linear analysis of the same model indicated that the ground motion should have induced in the structure lateral forces equivalent to a base-shear coefficient of 0.36, and bending moments at beam ends which drastically exceeded their capacity, computed with standard design formulas. The theoretical capacity for bending moments was exceeded in 69 sections with a maximum ratio of applied to resisting moment of 4.6 for positive bending, and of 2.2 for negative bending.

In order to take into account the overstrength of the structure, the load and strength safety factors were eliminated; expected average material strengths were used instead of nominal values. A 25% increase in concrete strength was assumed to account for the high rate of stress application, and the contribution of the slab reinforcement to the flexural capacity of the beam was considered. Figure 6 shows the moment-curvature relationships for a typical beam section for positive and negative moments, computed with and without consideration of the overstrength. The increase in bending capacity was 23% for positive and 34% for negative moments, for a section of an exterior frame; for interior beams the increases were 27% and 44%, respectively; the difference is due to the greater contribution of the slab at both sides of the beam in interior frames.

When the applied forces from the linear analysis were compared to the capacity computed considering the overstrength, the resistance was still exceeded in 29 sections at beam ends, with a maximum ratio of applied to resisting moments of 3.7 for positive and of 1.5 for nagative bending.

A non-linear analysis of an exterior longitudinal frame was performed assuming elasto-plastic behavior with yielding moments computed considering overstrength. The displacement history at the roof is shown in Fig. 5. As it can be appreciated, initially the displacements are the same as those computed considering linear behavior, but after the second peak of vibration the non-linear analysis shows a reduction of displacements, originated by the energy dissipated through yielding, and a markedly non-symmetrical response. This behavior does not correspond to that derived from the records, which indicates that the response had remained essentially linear elastic, despite the large number of sections in which the theoretical capacity was significantly exceeded. The non-linear analysis indicated a maximum ductility demand of 2.0 for curvatures due to negative moments and of 4.0 for positive moments.

The same frame was analyzed for a set of lateral loads applied with a triangular distribution along the height of the building. The loads were increased monotonically until a mechanism of collapse appeared. The load-displacement curve are shown in Fig. 7 for the structure with and without considering overstrength. It can be appreciated that, ignoring overstrength, the maximum capacity to lateral loads corresponds to a base shear coefficient of 0.23. This coefficient is raised to 0.28 when overstrength is considered. It must be remembered that the structure was designed for a seismic coefficient of 0.12.

It can be concluded that the actual capacity of the building was well above that computed taking into account all the foreseable sources of overstrength. The structure showed no damage and an essentially linear behavior under the effect of a ground motion that induced forces up to 3.7 times greater than what the critical sections are assumed to be able to resist. Possible explanations for the additional capacity are the following:

a) The contribution of non structural members. The masonry infills and the facade walls were ignored in the analysis. The appropiateness of this assumption seemed to be confirmed by the good matching of the measured and the computed response. Nevertheless the cracking of some walls indicates that they must have had some contribution to the structural capacity.

b) The effect of the high rate of strain applied only in a few cycles. This effect was taken into account only by increasing the strength of the concrete, therefore it did not lead to a significant increase in the bending capacity of the beams. It is possible that, for the kind of shaking withstood by the building, some increase in the bending capacity of the concrete members could exist.

EVALUATION OF THE RESPONSE OF AN OFFICE BUILDING IN MEXICO CITY

This is a ten-story office building, regular both in plan and elevation, whose lateral strength was essentially provided by four robust end walls in the transverse direction and by four frames in the longitudinal direction. The building was designed for a base shear coefficient of 0.078 in the longitudinal direction (frames) and 0.104 in the transverse direction (shear walls). The building was designed in the early 70's, and although the reinforcement does not comply with all present requirements for ductility, it was carefully detailed with a good distribution of transverse reinforcement in beams and columns and it showed good quality of the construction. The main features of the building are summarized in Fig. 8. It is founded on a 40m deep deposit of very soft clay, through friction piles. A more detailed description of the building and of the analysis of its response can be found in Ref. 10 and 11.

The performance of the building in the 1985 Mexico earthquake was characterized by the plastic hinging at the ends of several longitudinal beams, from the first to the sixth floors. The concrete was crushed at the top and bottom of the section and the longitudinal reinforcement buckled in several cases. Some diagonal cracking in the columns from the third to the sixth floors and some evidence of hinging at the column bases at the ground floor are found in the longitudinal direction. No sign of damage attributable to the shaking in the transverse dire.ction could be found.

Although the building was not instrumented to record its seismic response, it showed a very clear pattern of structural damage which allows a comparison between observed and computed behavior. Fundamental vibration periods measured by ambient vibration tests, after the earthquake, were 2.1 sec and 1.3 sec for the longitudinal and transverse directions, respectively.

The large period in the longitudinal direction reflects the flexibility of the frame system, but also the loss of lateral stiffness due to the damage. Measured periods are compared in Table 2 with those computed assuming linear behavior and considering the soil-structure interaction. A good coincidence is found in the transverse direction, whereas for the longitudinal direction the measured period exceeds the computed one by 15%, presumably due to the loss of the stiffness of the damaged building. First the response of the structure was computed by a tri-dimensional linear analysis and then by a non-linear analysis of planar frames in each direction. The ground motion recorded at another site of Mexico City with similar soil conditions was imposed to the base of the structure. Only the most significant part of the record was used (between seconds 30 and 80); the graph is shown en Fig. 9. The response was first computed assuming nominal member strengths derived from code-specified formulas, in which the strength reduction factors had been eliminated. Displacements and rotations at the base of the structure due to soil deformation were taken into account.The analysis in the longitudinal direction indicated the formation of plastic hinges (for both positive and negative bending) at the ends of most beams.

For example, Fig. 10 shows the history of ductility demands, related to the curvatures of a section at the end of a beam in the first floor. The maximum ductility demand was 17. For the transverse direction the analysis indicated that the response remained linear along the whole duration of the shaking.

The same analyses were performed considering the most probable values of the capacity of the structural members taking into account all the sources of overstrength, similarly to what already described for the El Camino Real Hotel. Fig. 11 shows the history of displacements at roof level obtained from the analyses for the longitudinal and the transverse direction. It can be appreciated that the response is much more severe in the longitudinal direction, with a maximum displacement of 48 cm, compared to 21 cm for the transverse direction. The long duration of the shaking and the large number of cycles with an almost armonic vibration must be highlighted. For the longitudinal direction the response was largely non-linear, even when overstrength was taken into account. Yielding was exceeded more than 10 times with a maximum ductility demand of 16. The distribution of plastic hinges in a longitudinal frame is shown in Fig. 12 and compared with the plastic hinges which actually appeared in the frame as derived from the observed damage. A close ressemblance can be perceived. As shown in Fig. 10, the number of inelastic excursions and the maximum ductility demand at a critical beam end are significantly reduced with respect to those computed without considering overstrength. Nevertheless the amount of inelastic response is still very large.

The capacity of the building, both in the longitudinal and transverse direction, was computed with a static analysis for a set of lateral loads proportional to the one obtained from the modal analysis with the spectrum of the SCT ground motion. As shown in Fig. 13, for both directions an almost perfect elasto-plastic behavior was obtained. When nominal properties are considered the maximum lateral capacity corresponds to a base-shear coefficient of 0.14 for the longitudinal direction, and of 0.21 for the transverse one. When overstrength was taken into account very similar curves were obtained and the resistant base-shear coefficient raised to 0.22 and 0.33 for the longitudinal and transverse direction, respectively.

From the tri-dimensional linear analyses, the base-shear coefficients necessary to exceed different limit states were computed. Values obtained without consideration of overstrength are shown in Table 2.

The afore-mentioned results indicate that for both directions the computed response corresponded closely to 'the observed performance. The patterns of plastic hinges in the longitudinal direction matched closely, and the ductility demands were compatible with the damage at the beam ends.

This structure did not show a seismic capacity in excess to that computed with established procedures taking overstrength into account. It must also be pointed out that the ratio between the maximum theoretical capacity and the design values was of 2, for the longitudinal direction, and of 2.9 for the transverse one. These values are lower than those computed for the El Camino Real building.

The comparison between the two cases indicates that the available overstrength can have drastic variations in different buildings and for different ground motions. The extremely large overstrength shown by the El Camino Real was not apparent in the Mexico City building.

The most evident difference between the two cases is the type of ground shaking. It seems that a structure could resist forces well above the theoretical capacity, if it is subjected to a short-duration motion with high dominant frequencies and that this overstrength is largely impaired for large duration motions with long periods.

PRACTICAL IMPLICATIONS

Response spectra do not contain all the information about the damage potential of a ground motion; therefore, they do not constitute a complete representation of the seismic action.

Important factors such as the duration of the shaking and its frequency content are not properly accounted for by the response spectra. The number of cycles of vibration significantly affects the structural capacity. The same spectral ordinate is more harmful when it corresponds to a long period than to a short one.

Despite the afore mentioned shortcomings, design spectra appear to constitute the most viable way for specifying seismic actions for building design. They can be derived from envelopes of linear response spectra of the ground motions which are critical for each range of period. The modification factors affecting the ordinates of such envelopes should take into account both the characteristics of the strong motions that had generated them and those of the structure they are meant to be applied. Therefore the modification factors must vary for different ranges of periods according to the duration and frequency content of the strong motion which governed each part of the spectrum.

In addition to the so called ductility, other structural characteristics should affect the modification factors which reduce the linear response spectra, mainly damping and overstrength.

Most building codes assume a 5% damping for deriving the design spectra. This amount of damping roughly corresponds to measured values for common buildings subjected to moderate earthquakes. Nevertheless, for some special structures, such as bare frames with stiff connections, a significantly lower damping could be expected, hence the design actions should be increased. On the other hand, stiff massive structures on soft soil can count on an additional source of damping due to the radiation of energy between the structure and the soil. There is strong evidence that this effect significantly reduces the actual shaking of the buildings, but no reliable rocedure of its quantification seems to be available, yet.

Reduction factors related to energy dissipation through non-linear response (ductility factors) should be limited to values which do not demand extreme local deformations that could lead to unrepairable damage or to deteriorating behavior. Ductility reduction factors should be significantly lower than local available ductilities. From tests of complete structures it has been concluded that ductility reduction factors of more than four can hardly be acceptable even for very ductile, well-detailed and highly redundant structures. Moreover, for buildings with short fundamental periods, ductility is of little help for the reduction of the necessary structural capacity. Therefore, ductility reduction factors should be small in this region of the design spectrum. On the other hand, quasi-harmonic ground motions as those that occur over thick deposits of very soft soil due to earthquakes originated at very long distances, produce response spectra with very high peaks for the resonance periods. These peaks are drastically reduced by non-linear behavior; therefore, ductility reduction factors greater than the average should be allowed in this situation.

Overstrength, i.e. the reserve of structural capacity over the minimum value prescribed by the codes, is a key factor for the reduction of linear elastic spectra. Available overstrength varies widely depending on the type of structure and on the characteristics of the strong motion. For some low-redundant simple structures, overstrength corresponds roughly to the safety factors required by the codes, therefore no increase in the reduction factors should be made for this concept. For other buildings, the actual capacity could be up to ten times that coresponding to the base-shear coefficients prescribed by the codes, then its effect should be reflected in the overall reduction.

The scope of this work was to point out the most important factors involved in the problem and to quantify the influence of some of parameters for two cases representative of different situations. The results do not allow to arrive to general recommendations. Nevertheless, they indicate that the extremely large difference between ordinates of response spectra from actual records and those of the design spectra cannot be explained by the foreseable contribution of additional damping, ductility and overstrength.

It is in the author's mind that a more clear and rational procedure for the determination of the design seismic actions is needed. The evaluation of the response of a wider set of buildings subjected to severe ground motion will be of great help in establishing such a procedure.

AKNOWLEDGMENTS

Freddy Zurita performed the analyses of the El Camino Real Hotel, and Jorge Avila those of the Office Building in Mèxico City. The careful review of the manuscript by Sergio Alcocer is greatly appreciated.

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Table 1. Base shear strength of a concrete frame building computer with different hypoteses (Adapted from Shahrooz and Moehle, Ref. 5).

Assumptions for the computation of the lateral load capacity of the structure	Base Shear Coefficient *	Ratio from computed to design strength
Strength of each section exactly equal to forces from static analysis with UBC lateral load	0.092	1.00
Same, except forces are those obtained from modal analysis for code values	0.108	1.16
Strength to resist gravity load condition and seismic load condition, including load factors	0.148	1.60
Actual beam dimensions and reinforcement according to ACI-83 requirements	0.244	2.64
Actual column dimensions and reinforcement according to ACI-83 requirements	0.274	2.97
Strength computed removing strength reduction factors	0.305	3.30
Expected material properties considered instead of nominal values. Effect of confinement included	0.429	4.65
Contribution of the slab to flexural strength of the beam included	0.706	7.65

* Factors mentioned for each case are additional to those of the previous ones

** Maximum capacity for the model tested in a shaking table corresponded to a base shear coefficient of 0.68

Table 2. Properties of the office building in Mexico City

Direction	Fundamental Period (sec)		Limit State	Resistant Base Shear Coefficient *
	Measured	Computed		
Longitudinal	2.1	1.84	Beams, Bending Columns, Shear	0.11 0.15
Transverse	1.3	1.31	Walls, Bending Walls, Shear Coupling Beams, Bending	0.15 0.20 0.15

* Not considering overstrength

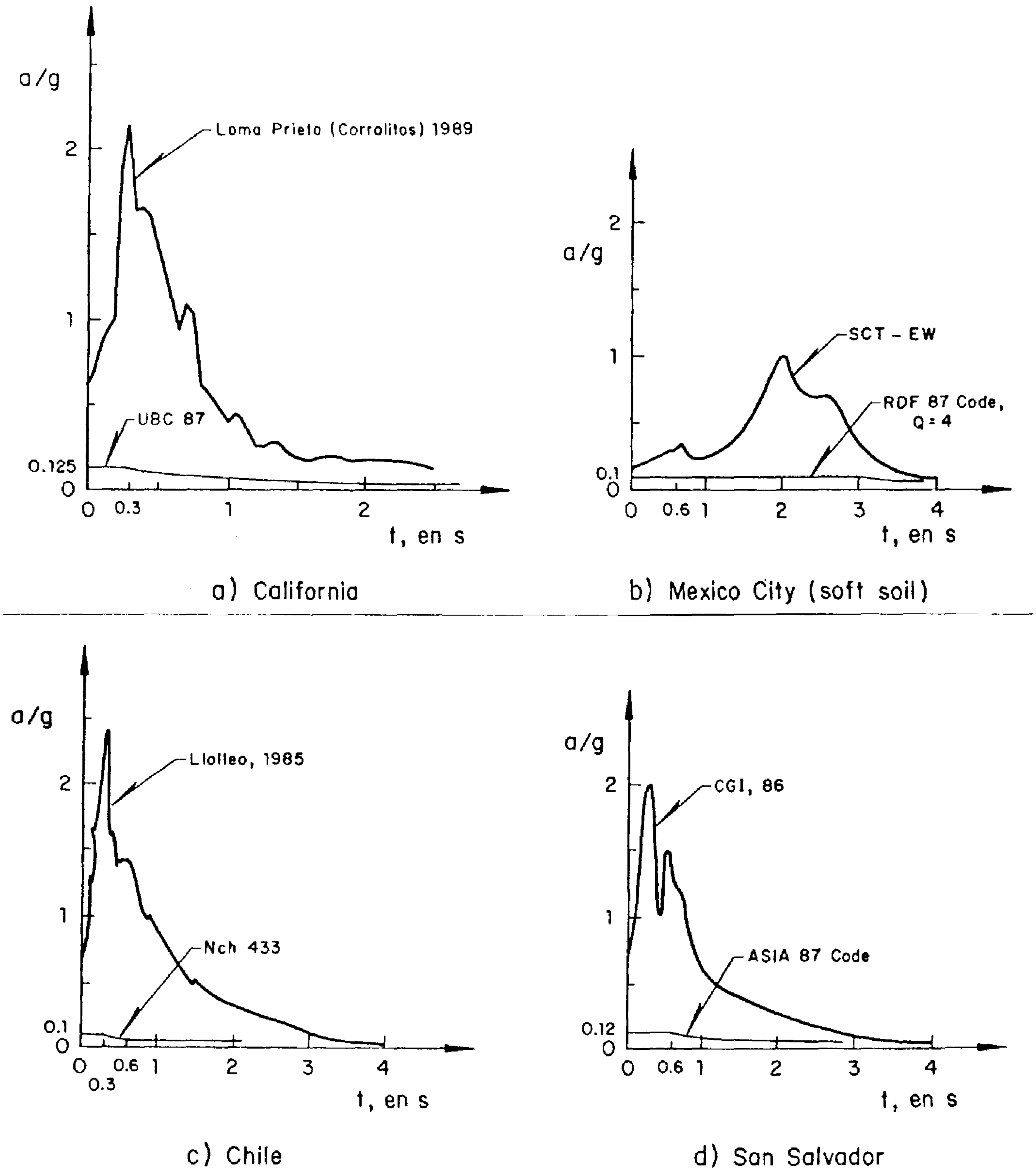
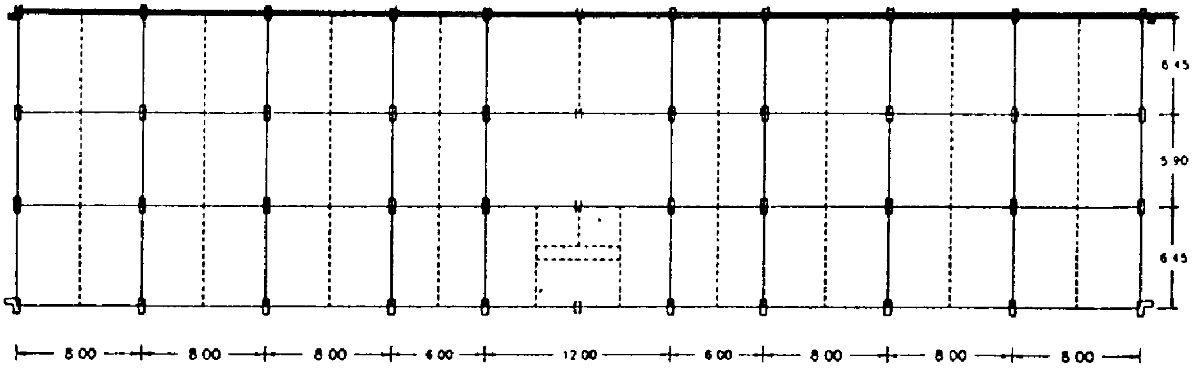
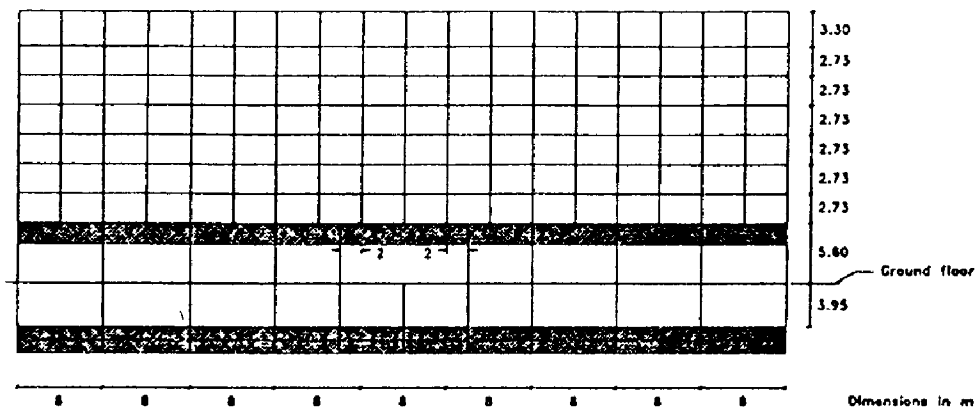


Fig 1 Comparison of response spectra of actual strong motions and design spectra specified by codes for ductile structures



a) Plan view



b) Elevation view of an external frame

Fig 2 Schematic views of the structure of the El Camino Real Hotel

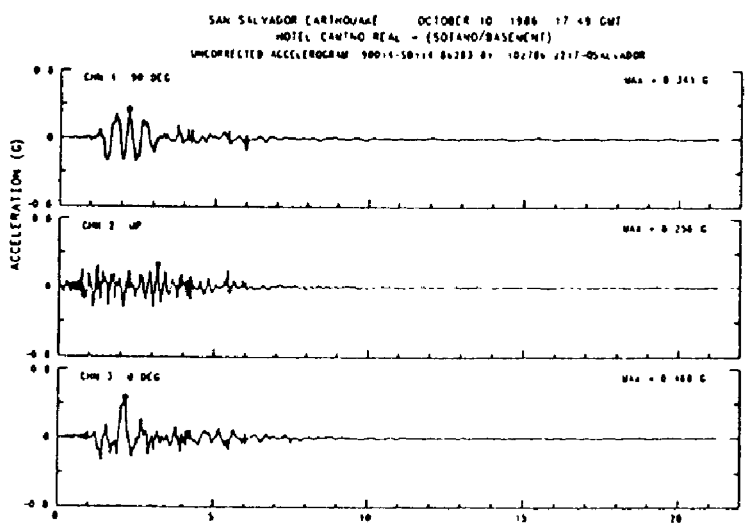


Fig 3 Accelerograms of the motion at the basement of the El Camino Real Hotel

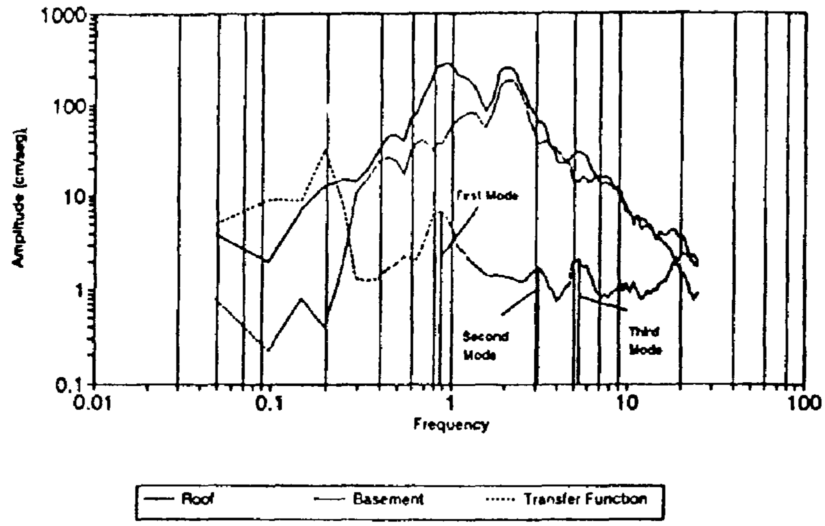


Fig 4 Fourier spectra of the ground motion records at basement and roof

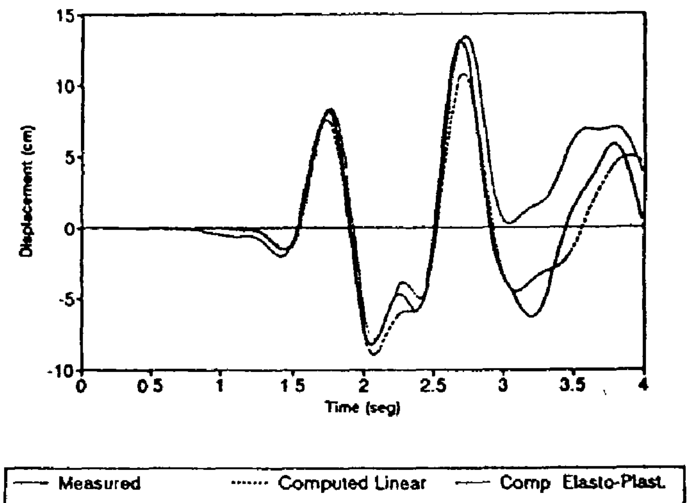


Fig 5 Computed and measured displacement histories at roof level

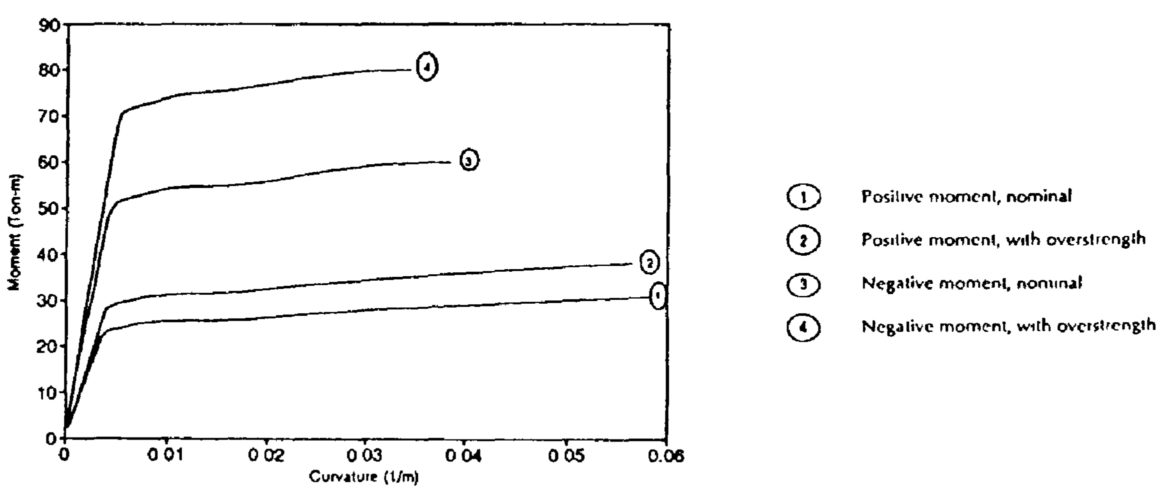


Fig 6 Moment-curvature relationships for typical sections of beams of the longitudinal frame

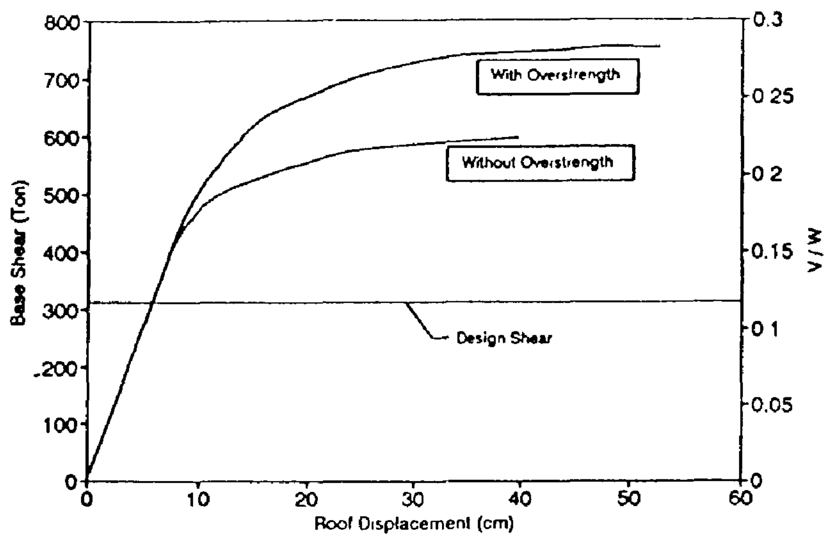


Fig 7 Base shear versus roof displacement of a longitudinal frame subject to a triangular distribution of lateral loads

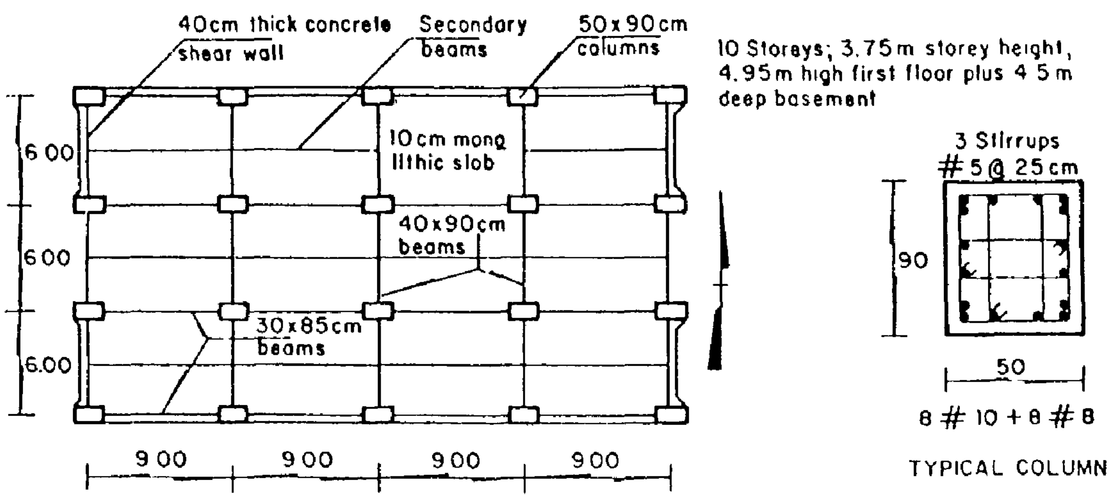


Fig 8 Schematic view of the structure of the office building in Mexico City

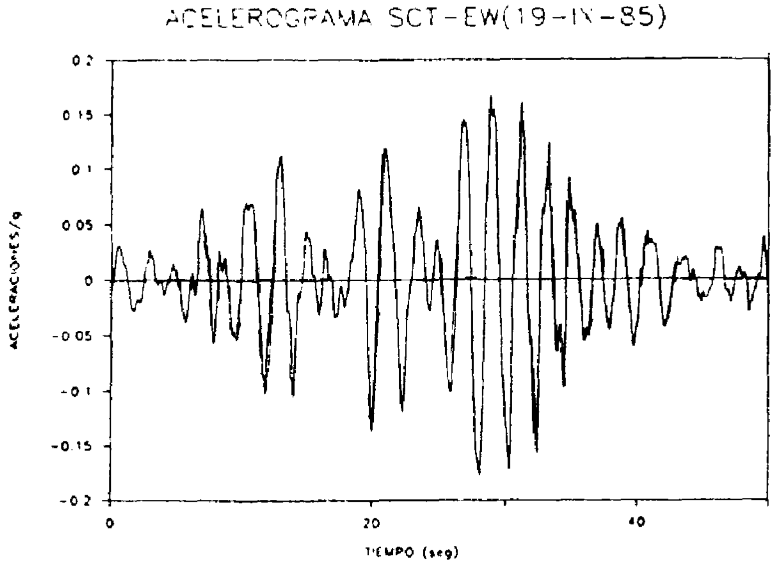


Fig 9 Accelerogram at the SCT station in Mexico City

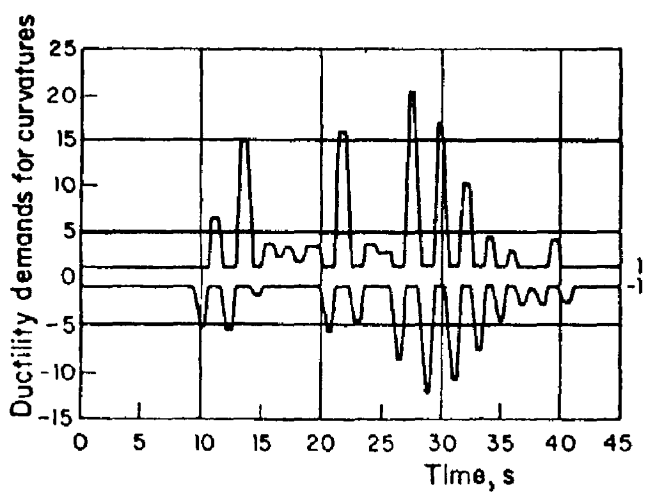


Fig 10 Ductility demand at the end of beam in the first floor

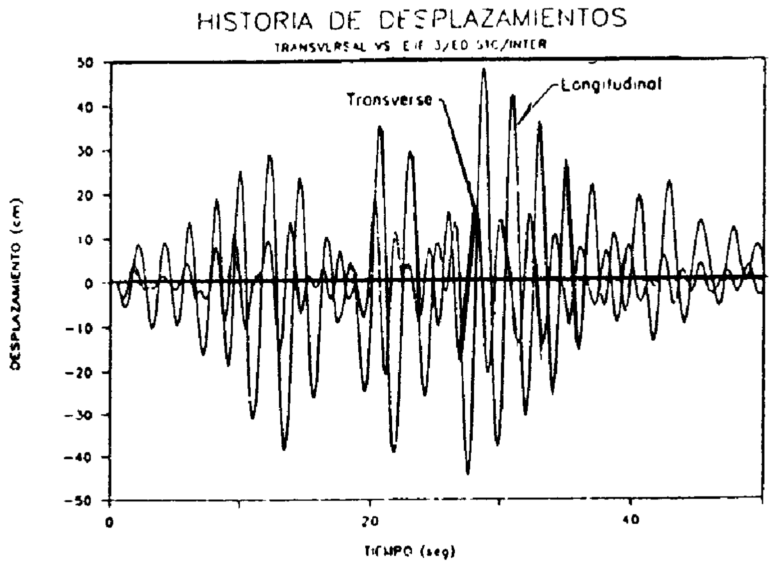
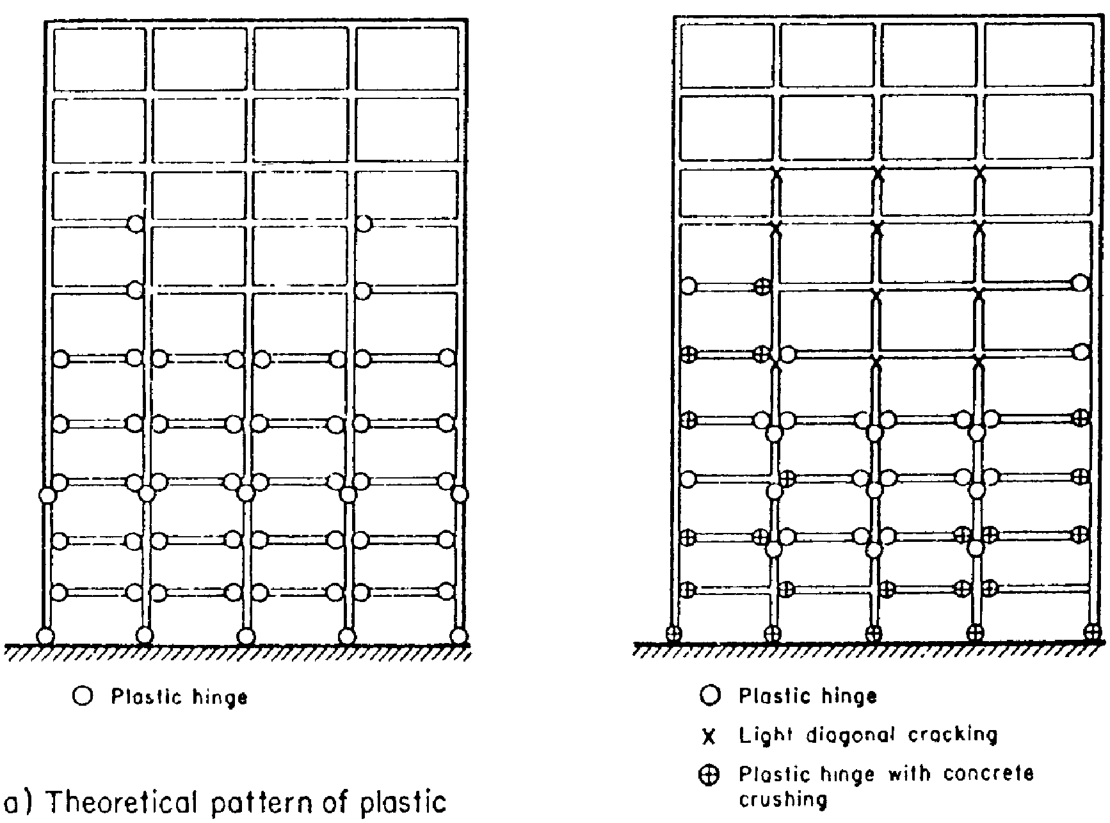


Fig 11- History of displacements at roof level (considering overstrength and non-linear behavior for the longitudinal direction)



a) Theoretical pattern of plastic hinges

b) Schematic pattern of damage

Fig 12 Theoretical and observed patterns of damage for a typical transverse frame

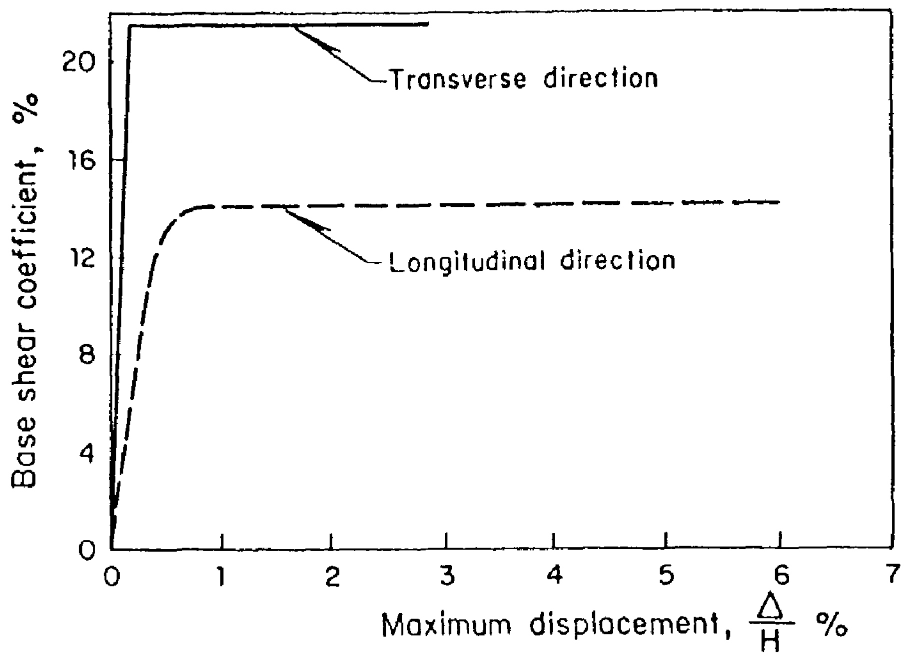


Fig 13 Base shear versus roof displacement for longitudinal and transverse frames subjected to static lateral forces