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Code-prescribed seismic actions and performance of buildings

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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.

1 GROUND MOTION RECORDS AND DESIGN SPECTRA

The design forces specified by building codes are 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.

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.

Some examples 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.

The maximum peak of the response spectrum recorded at Corralitos, California, during 1989 Loma Prieta earthquake is more than 20 times greater than the maximum ordinate of the design spectra specified by the UBC Code for California. It must be pointed out that the Loma Prieta earthquake is well below the maximum that can be expected in this area. The SCT-EW record obtained in the lake-bed area of Mexico City, for the 1985 earthquake is about ten times greater than the design ordinate from the 1987 building code. There is clear evidence that the ground motion in same parts of Mexico City significantly exceeded the amplitudes recorded at the SCT site, during this event. 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. Finally, the 1986 San Salvador earthquake was a local event of moderate magnitude (Ms=5.4); nevertheless the response spectrum has a peak of about 2g for a 0.3 sec period. The maximum base shear coefficient for the area is 0.12.

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. 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. 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 for the curvatures at critical sections). 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. 1).

b) Damping: Design spectra are typically derived from linear response spectra for 5% damping. For moderate earthquakes, damping coefficients have been recorded between 2 and 3%. 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, such as load factors and strength reduction factors, differences between actual and nominal strength of materials, conservatism of design formulas, and overstrength generated by minimum requirements for dimensions of members and reinforcement.

ii) Conservatism in the model, specially related to ignoring the so called non-structural members.

iii) Overdesign that can be produced by rounding up of dimensions, by the need to cover load combinations not involving earthquake forces, and by designing for "first yielding" conditions ignoring redistribution of internal forces and the reserve of capacity to reach a mechanism of collapse.

An interesting example of evaluation of the contribution of the different sources of overstrength has been made by Shahrooz and Moehle (Ref. 2), for a six-story concrete frame structure designed according to the UBC and ACI codes. 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, sometimes attaining values of the same order as those in the former example. More often the former ratio has been in the range from three to four. 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 these 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.

2 EVALUATION OF THE RESPONSE OF A HOTEL IN SAN SALVADOR

The El Camino Real Hotel is a nine-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. 1. The seismic design was performed through a static analysis with a

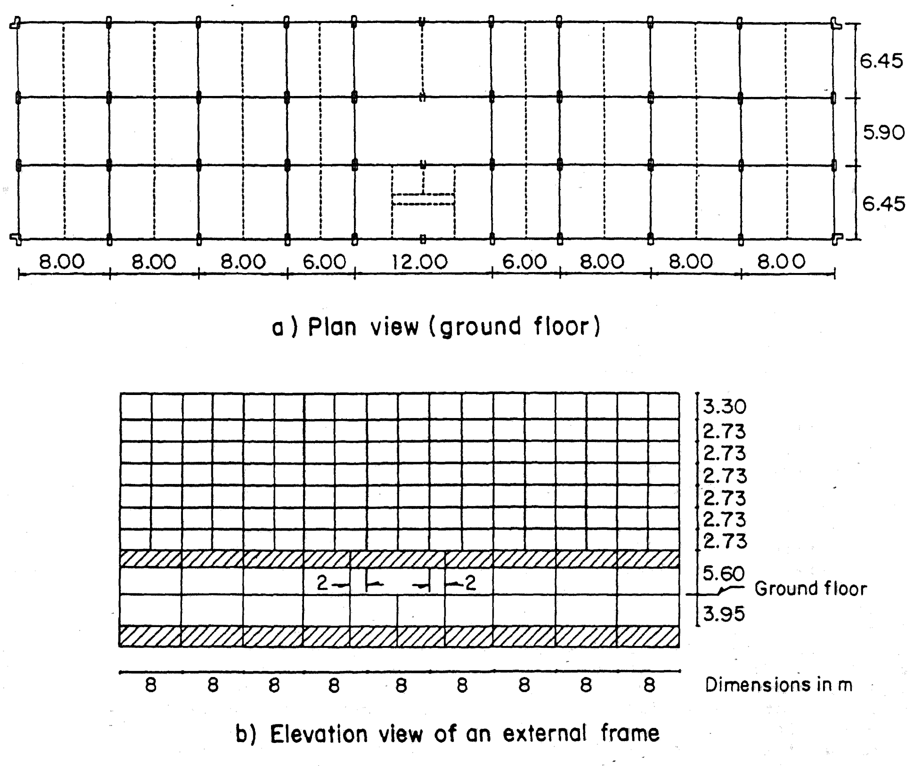

Figure 1. Schematic views of the structure of the El Camino Real Hotel

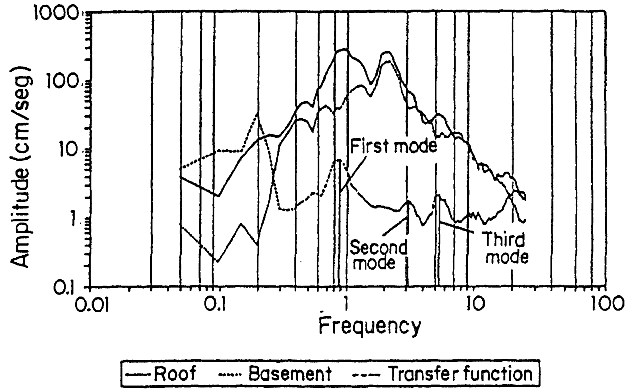

Figure 2. Fourier spectra of the ground motion records at basement and roof of the Camino Real Hotel

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 carefully 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, second 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. 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. 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. A complete report of the analyses performed on the El Camino Real building can be found in Ref. 3.

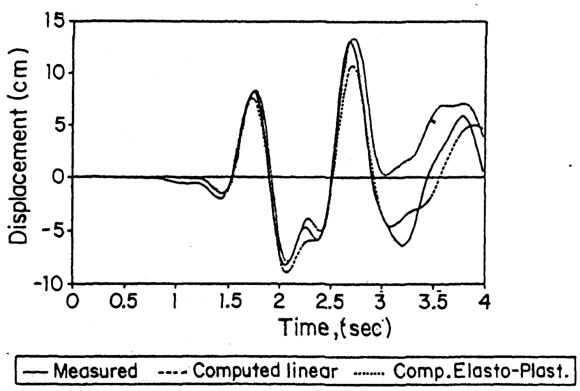

Figure 3. Computed and measured displacement histories at roof level of the El Camino Real Hotel

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. 2. 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. 3.

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. 4. Furthermore, a 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

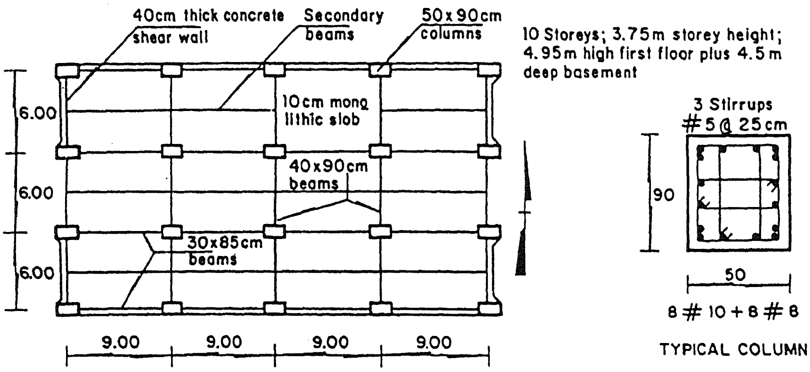

Figure 4. Schematic view of the structure of the office building in Mexico City

0.36, and bending moments at beam-ends which drastically exceeded their capacity, computed with standard design formulas.

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 slab width of 1.5 times the beam depth at each side of the web was assumed to contribute to the flexural strength of the beam. When the applied forces from the linear analysis were compared to the computed capacity, the resistance was 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 negative bending.

A non-linear analysis of an exterior longitudinal frame was performed assuming elasto-plastic behavior.

The displacement history at the roof is shown in Fig. 3. 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.

It can be concluded that the actual capacity of the building was well above that computed taking into account all the foreseeable 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.

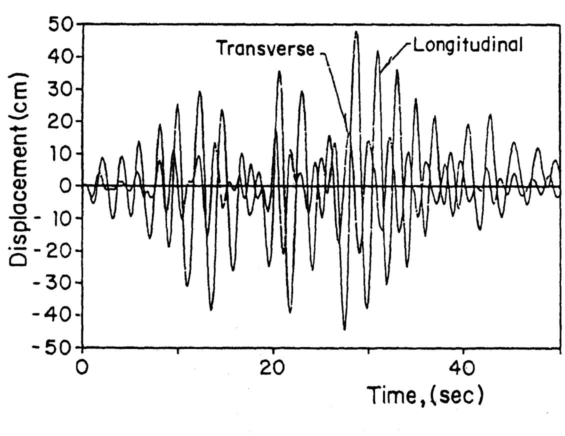

Figure 5. History of displacements at roof level of the office building (considering overstrength and nonlinear behavior for the longitudinal direction).

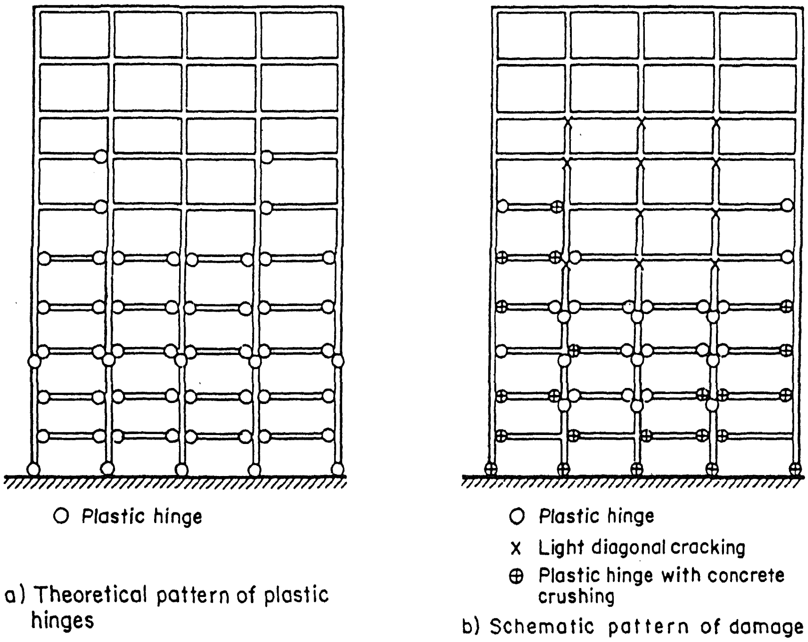

Figure 6. Theoretical and observed patterns of damage for a typical longitudinal frame of the office building

3 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. 4. 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. 3.

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 direction could be found.

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. The 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. 5 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 harmonic vibration must be highlighted. For the longitudinal direction the response was largely non-linear. 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. 6 and compared with the plastic hinges which actually appeared in the frame as derived from the observed damage. A close ressemblance can be perceived.

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. The resistant base-shear coefficient was 0.22 and 0.33 for the longitudinal and transverse direction, respectively.

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.

4 PRACTICAL IMPLICATIONS

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 in some cases by the foreseeable 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.

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