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I regard Luis Esteva with high esteem as a great master of the practice of scientific research in engineering. And am honoured to have him as a friend and teacher. As a peer and former student, I am proud to participate in such a memorable event, the celebration of his 70th birthday. What precisely has Luis done to contribute to my education as a researcher? Since the very first moment I met him he has always been a genuine intellectual, inspiring me to be rational in my actions and to maintain the highest of standards in whatsoever I am involved. Luis was one of my teachers at undergraduate level, however, his apparent shyness and coolness did not allow his merit to shine through. Shortly after starting my graduate work, I became one of his research students and it was then that I really began to know him, finding him a delightful person to work with. There was little doubt that with Luis I had a fine and dedicated teacher who was focused on his students and their learning, an outstanding and active researcher, a wonderful role model and mentor for students, and a genuine and honest person. He is reliable and works hard in whatever assignment or task he is involved. To me Luis is one of the shining examples of the members of our Institute responsible for paving the way towards a very bright future as a research institution in engineering. I sincerely hope that we can continue to celebrate with Luis- the person, the teacher and the scientist- through our collective efforts to reach excellence in education and research.

Thank you Luis, for everything up till now and for the years to come.

A Gustavo Ayala
12 September 2005

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PERFORMANCE BASED SEISMIC EVALUATION AND DESIGN OF STRUCTURES – A UNIFIED APPROACH

ABSTRACT

This paper presents a new seismic evaluation and design procedure for structures in agreement with the performance-based seismic design philosophy. Throughout the paper, the aims and limitations of current seismic evaluation and design practice and the tendencies of the performance-based seismic design are discussed. The procedure considers the non-linear behaviour of the structural elements, performance indices of current design codes, uniform hazard design spectra and in the case of 3D buildings bi-directional earthquake action. To illustrate the application of the seismic evaluation procedure a typical reinforced concrete plane frame irregular in elevation and designed in accordance with the latest version of Eurocode 8 is used as example. For the illustration of the seismic design procedure an asymmetric building located in Mexico City is designed for a basic design objective with a life safety performance level and a design level corresponding to a design spectrum with a given exceedence rate of the chosen performance index. For validation purposes, results of non-linear step by step analyses of the chosen examples are presented and discussed regarding the potential of the procedure to produce expected global but not necessarily local performances.

Introduction

The occurrence of destructive earthquakes in recent years all around the world, e.g., Mexico City (1985), Loma Prieta (1989), Northridge (1994), Kobe (1995), Turkey (1999) and Taiwan (1999), has made evident that seismic evaluation and design methods proposed and used by current codes do not always provide the safety levels and performance expected when the structures are subjected to design demands.

To overcome this problem, during recent years research efforts have been devoted to develop new seismic evaluation and design methods based on the performance of the structures, aimed to provide better predictions of the performance of structures, closer to the real performance that the structures would undergo when subjected to different seismic demand levels throughout their life span. The actual tendency in the seismic evaluation and design of structures is to use simplified non-linear analysis methods which incorporate in a realistic and explicit way the geometric and behaviour characteristics of the structures to approximately predict their seismic performances for evaluation or design purposes.

This paper presents a new procedure for the seismic evaluation and design of structures in agreement with the current tendencies of the performance-based seismic design philosophy. In the case of design, this procedure offers the possibility of satisfying different performance parameters used to define the design performance level, controlling the damage corresponding to a considered seismic demand given by a design spectrum for a given rate of exceedence of a chosen performance parameter. The proposed procedure is applied to the seismic evaluation of a reinforced concrete frame irregular in elevation and to the performance based seismic design of a reinforced concrete mass asymmetric reinforced concrete building. The results obtained for both buildings are discussed when their found or aimed performances are compared to those obtained using non-linear step by step dynamic analyses.

Performance Based Seismic Evaluation

One of the current challenges of seismic evaluation of structures is to predict, as accurately as possible, the performances that they will exhibit when subjected to design seismic demands, all this considering the non-linear behaviour of the materials, realistic dynamic properties and consistent seismic demands. The answer to this challenge is not an easy task, currently there exist two accepted options for the evaluation of seismic performance, one based on non-linear dynamic step by step analyses (NLDA) with a pre-assigned set of seismic demands, and the other, based on simplified non-linear static analyses (NLSA) methods with the same set of seismic demands or in a more direct way using smooth spectra.

In the first option, the results are obtained using NLDA may be considered as the "exact" results of the response of the structure to one or several earthquake records consistent with the design seismic demand. This option has the advantage of being analytically robust, but the disadvantages of being based on methods of high technological complexity, not commonly known by design engineers, and of giving responses to a limited set of seismic records, presumably obtained from catalogues, generally incomplete, to assure the statistical validity of the obtained results. Although this last disadvantage may be circumvented using a family of synthetic records compatible with the design spectrum, this option is still open to discussion and widely used among researchers. It is important to mention that the main disadvantage of using NLDA is that it involves an expensive process, due to the significant computational and human resources involved in the determination and evaluation of the massive amounts of results derived from the calculations, nevertheless, availability of more efficient computational tools and better education of practicing engineers show the ever increasing weakness of this limitation.

In the second option, justified by the apparent limitations of the first option, simplified NLSA procedures are used as evaluation methods. Evidently, this is an easier option than the NLDA as the methods used are based and developed using simpler and the Incremental Response Spectrum Analysis (Aydinogliu 2003, 2004). More recently the results of the ATC-55 Project (FEMA-440) were published by FEMA (2005) containing practical recommendations for improved prediction of the inelastic structural response of buildings to earthquakes.

This paper presents and exemplifies a new integrated method for the seismic performance evaluation and design of structures, which considers the contribution to the performance of higher modes of vibration and uses a smooth response spectrum as design seismic demand. The development of this integrated method was aimed to provide structural engineering practitioners with a simplified seismic evaluation and design methodology, based on readily available knowledge and applicable with commercially available analysis tools, e.g., SAP 2000 (CSI, 2000). The evaluation option of the method as presented may be considered as an evolution of the evaluation method proposed by Requena and Ayala (2000) and formalized by Isakovi et al., (2005) for its application to bridges, and for design purposes, an evolution of the method originally proposed by Ayala (2001) for structures with single-mode dominated performances and by Basilio and Ayala (2001) for multi-mode dominated performances.

The most important characteristic of the method is its simplicity as it considers the contribution of higher modes of vibration using existent modal combination rule and constructs the non-linear behaviour curve using evolving modal spectral analyses with modal information corresponding to structures in different damage stages. The method as developed cannot be considered a NLSA method because strictly speaking a pushover analysis is never performed for the construction of the behaviour curve, normally extracted from the capacity curve of the structure which in this case is not required, this characteristic is also present in other methods of evaluation such as that proposed by Aydinogliu (2003) an a number of other authors. The procedure involves four main steps:

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Definition of the seismic demand. The seismic demand is defined by a smooth response spectrum corresponding to a chosen seismic demand level. This definition has some advantages as this is the normal way in which it is presented in most current seismic design codes. This spectrum represents the envelope of the response spectra from many seismic records of various intensities, generated at different sources.

Construction of the behaviour curve. The behaviour curve of a reference system is constructed by a series of modal spectral analyses (MSA) each producing an evolving

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displacement, Sd^*, is calculated using the equal displacement rule, Veletsos and Newmark (1960), with proper consideration of its short periods correction, CEN (2003a). This rule

analyses corresponding to the two performance stages considered with a design elastic spectrum reduced by factors Fe and Fi, defined from the strengths of the elastic system, R_e, as illustrated in Fig. 2.

The overall design process for a performace level defined by a design ductility is illustrated in Fig. 3.

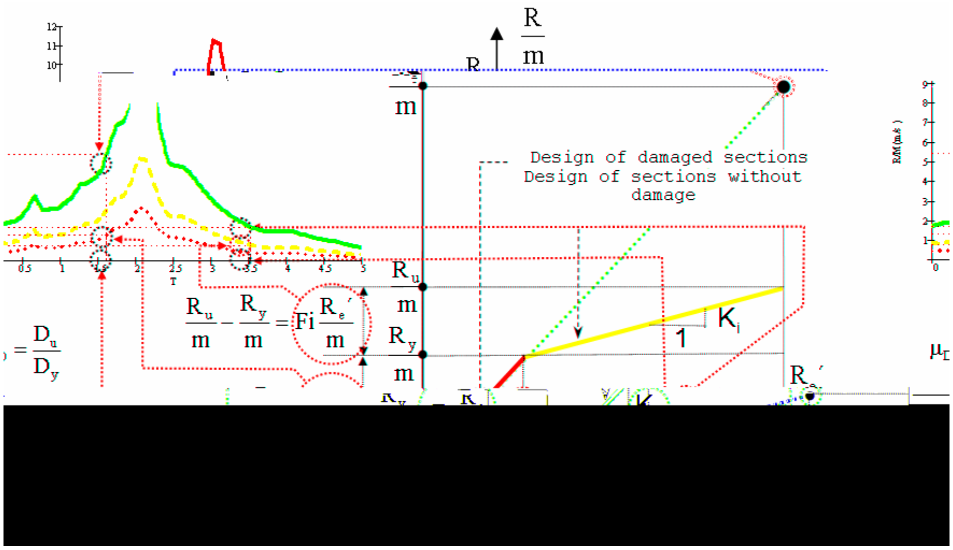

Figure 2. Factors Fe and Fi characterizing the reduction of elastic force

Figure 3. Performance based seismic design process

Superposition of the seismic and gravitational effects.

Moment-curvature diagrams. Throughout this work, the moment curvature diagrams of the sections were idealized with an elasto-plastic model with the following characteristics: the equal energy rule applies, horizontal second branch, and slope of the first branch defined by the secant at 60% of the yield point on the analytical moment curvature diagram, Fig. 6.

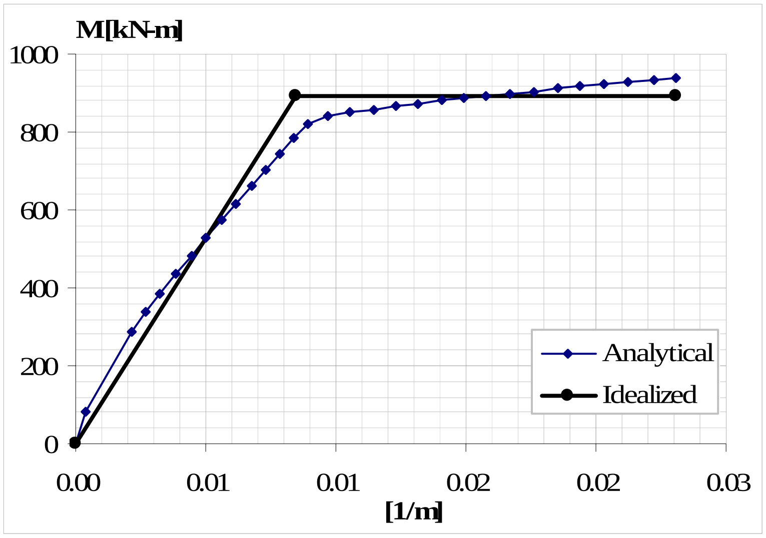

Figure 6. Moment Curvature Diagrams, analytical vs *idealized*

Synthetic accelerograms. To validate the results obtained with this method, a family of 50 synthetic accelerograms, compatible with the target design spectrum, was simulated by Isakovi et al.(2005). A sample synthetic accelerogram is presented in Fig. 7(a), where the shape of the accelerogram and the peak ground acceleration may be observed. To establish the validity of the accelerograms simulated, Fig. 7(b) shows the comparison of the target design spectrum and the mean value of the response spectra generated for the family of 50 accelerograms.

Presentation of results The seismic performance obtained with the method here proposed is compared with the mean value of the results of the 50 synthetic accelerograms; the mean of the time history (TH) analysis +/- one standard deviation; and the results obtained with the N2 method, in order to have a comparison with a simpler method. Displacements, interstorey drifts and storey shears are presented, in Figs. 8, 9 and 10 respectively, for the considered example.

Fig. 8 shows a comparison of the distribution of the maximum displacements obtained from the time history analyses of the twelve-storey frame with that of those obtained with the proposed method. It may be observed that they are similar and on the safe region. Whereas, when these displacements are compared with those obtained with the N2 method, they are considerably different and on the unsafe region. This situation makes it evident that the method works for frames with considerable participation of higher modes and that it also works for significant incursions in the non-linear range, as is the case of the considered irregular frame.

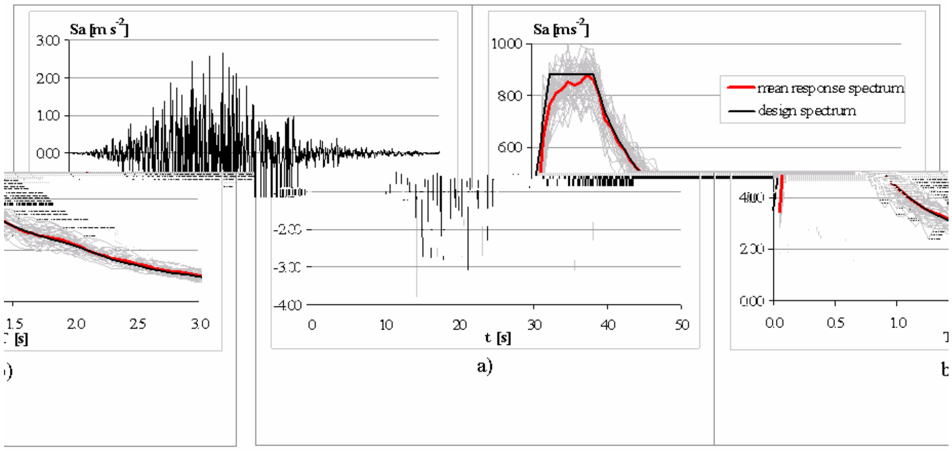

Figure 7. Synthetic accelerogram (a) and comparison of spectra (b)

The interstorey drifts depicted in Fig. 9 show that for the structure evaluated with the proposed method, the results for the lower levels are in good agreement with the statistics of the non-linear TH results and are on the safe side. However, the results for the upper levels are quite different and on the unsafe side. This tendency in the deformed shapes given by the proposed method is due to the use of a modal combination rule to obtain the results. In any case, the results of the proposed method remain within the band generated with the TH response +/- one standard deviation.

Finally, Fig. 10 shows that for the evaluated structure the storey shears calculated with the proposed method and with the N2 method are somewhat smaller than the results of the TH.

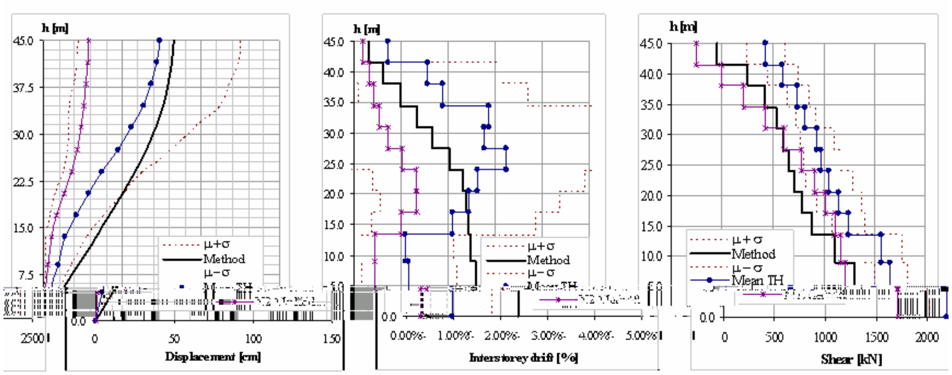

Figure 8. Storey drifts *Figure 9. Interstorey drifts* *Figure 10. Storey shears*

Performance Based Design

The design strategy was applied to a 15-storey RC building with asymmetry given by the distribution of masses at floors. For its design the following assumptions were used: 1) Rigid nodes and cracked inertia moments for the sections, 2) rigid floor diaphragms, 3) masses concentrated at the mass centres of the storeys, 4) P- and soil-structure interaction effects ignored and 5) non-standardized design of the elements.

The model of the undamaged structure is shown in Fig. 11, and the corresponding damaged model in Fig. 12. The fundamental periods corresponding to these models are Te=1.60 s and Ti=3.53 s. The damaged distribution used in the damaged model is consistent with the expected performance level and it is introduced in the analytical model by moment releases at the ends of elements which accept damage. The damage distribution used led to a post-yielding stiffness ratio, =0.20, proposed as a target parameter and attained through the occurrence of hinges at the beams of the first eight levels.

In this example the design objective of the asymmetric building was for standard occupancy with a life safety performance level and associated seismic design level corresponding to a rare earthquake (SEAOC, 1995). The selected performance level is quantified by a ductility demand of = 4, a value which is thought to be associated to acceptable rotation levels of the sections of the structural elements accepting damage in the assumed damage distribution.s inP5(owET/Cs6 cs171717scn12)]3 232i protection with = 4 and design spectrum R/m (T, =0.20, =4 y Tr=1000 years), the design strength of the reference system is defined by the spectral ordinate corresponding to the fundamental period, T_e, Ry/m=1.28 m/s^2. The yield displacement obtained from the spectrum in the ADRS format is y=(1/ $_e^2$) Ry/m=0.083 m.

When the performance index is given by a ductility demand, as it is the case of this example the characteristic point of the design stage with damage is obtained from the geometric relationship between the two branches of the behaviour curve and the definition of ductility demand in the ADRS format, Fig.14, Ru/m=Ry/m[1 + (-1)]= 2.067 m/s^2. The strengths [Ry/m] and [Ru/m - Ry/m] are distributed to the structural elements by means of modal spectral analyses using the uniform hazard elastic design spectrum defined as R/m(T, = 1, = 1, Tr = 1000 years), Avelar *et al.* (2003).

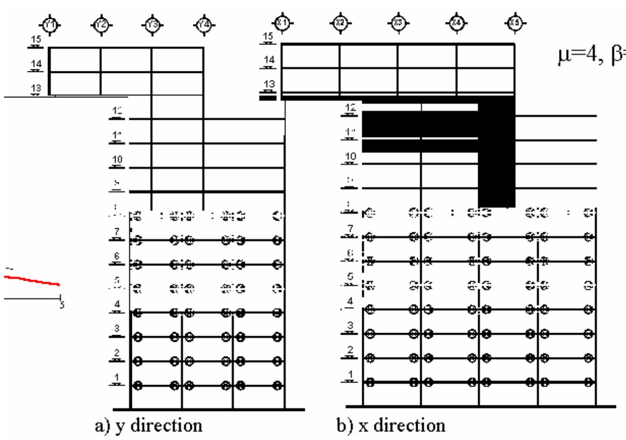

Figure 12. Analytical model with proposed damage distribution

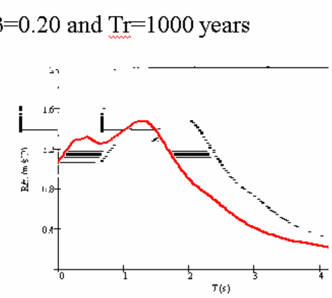

Figure 13. Uniform hazard design spectrum

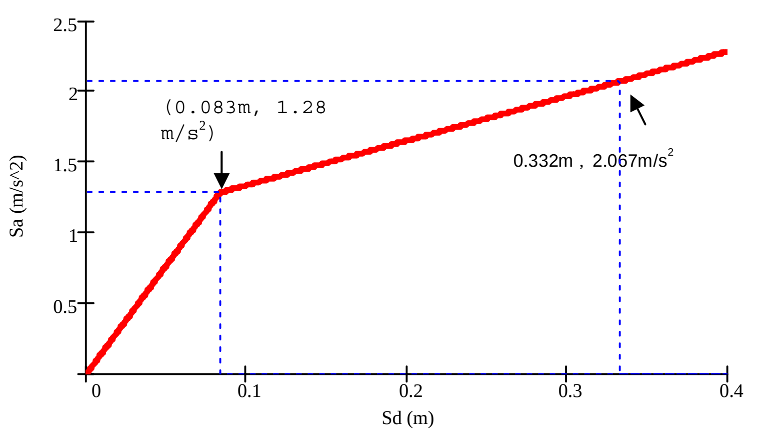

Figure 14. Complete behaviour curve

It is important to mention that it is necessary to check the rotation capacity of the hinged sections to guarantee no local failures, which would stop the structure attaining its design deformation capacity.

The strengths of the elements which admit damage and of those which remain elastic, are defined using an elastic design spectrum reduced by factors Fe = 0.24 and Fi = 0.47 respectively, Fig. 15.

Fig. 16 illustrates the distribution of shears in the columns of the first interstorey. The shown shear values are presented without standardization and are representative of the characteristic strength distribution of the target design objective.

Fig. 17 shows the global behaviour model characterized by bilinear curves in both orthogonal directions to compare the ultimate design of the complete system strengths which are very close to the design behaviour curve and the design base shear obtained.

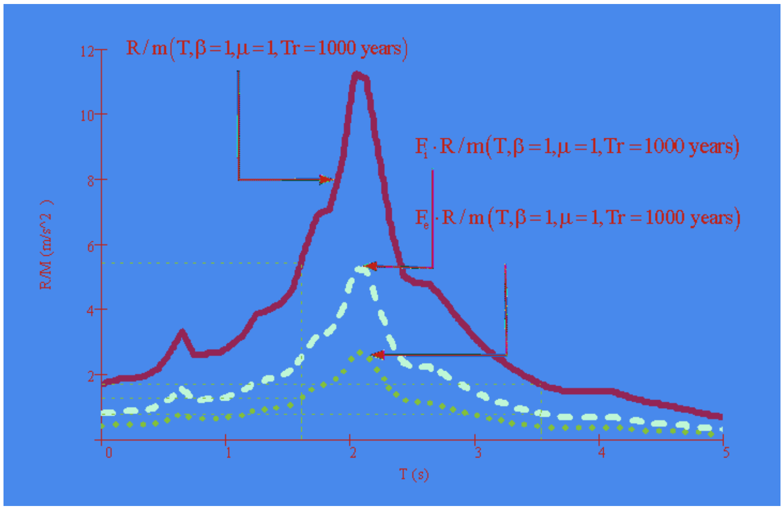

Figure 15. Reduction of the elastic design spectrum

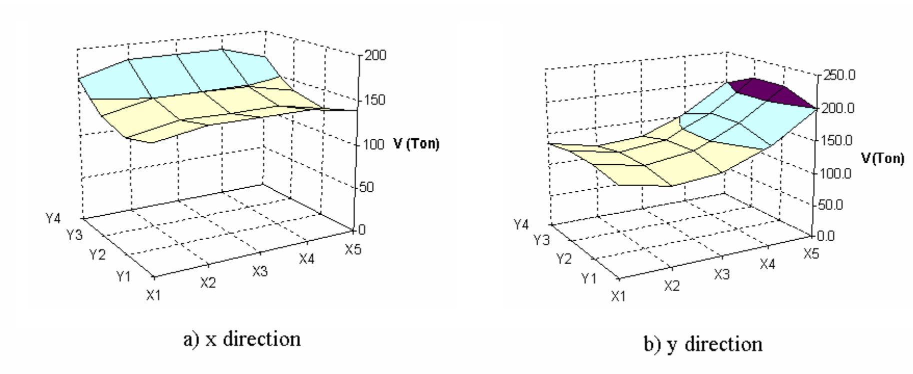

Figure 16. Variation of design shears in the first level columns

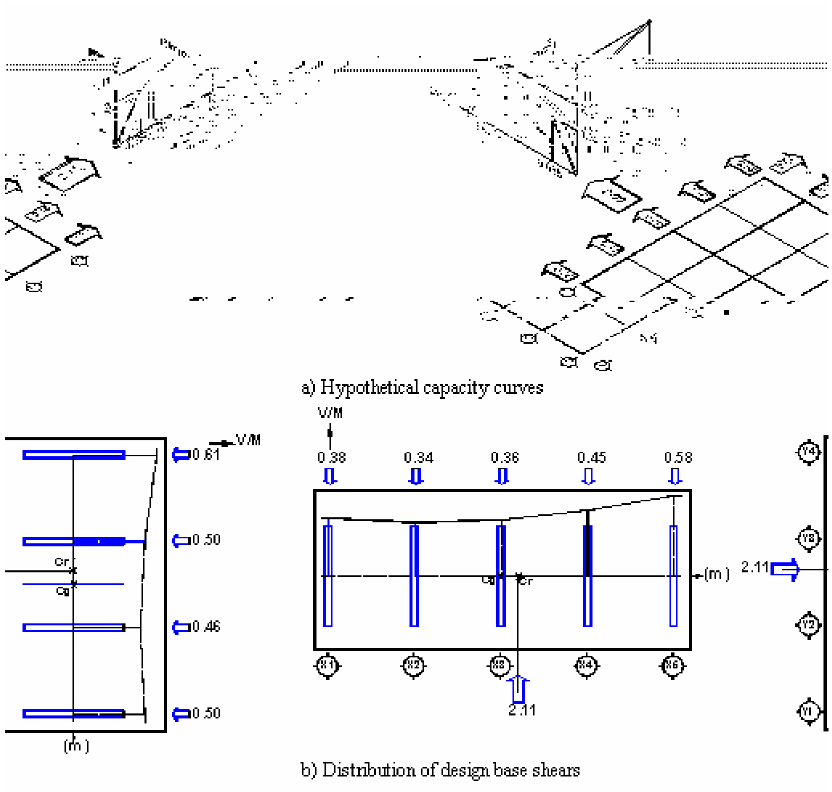

Figure 17. Design base shears

Validation of Results. To validate the seismic performance of the example buildings NLDA were performed for all considered load combinations under the following assumptions:

i). The non-linearities in beams and columns were concentrated at their end sections with a bilinear hysteresis rule without degradation of stiffness or strength and non-standardized designs for the elements.

ii). The damping of the structures was assumed to be of the Rayleigh type.

iii). As seismic demands the NS and EW components recorded at the SCT site during the 1985 México earthquake, records with characteristics compatible with those used to define the design seismic demand for the soil type in which the structure is assumed to be located.

iv). The performance of the building was evaluated for the following combinations of gravitational loading (GL), North-South seismic action (NS) and East–West seismic action (EW): 1) GL+EW+NS, 2) GL+EW-NS, 3) GL+NS+EW and 4) GL+NS-EW.

The design evaluation is carried out revising the parameters that characterize the seismic performance of the structure at global and local levels as calculated wit the non-linear analysis program Canny-E (Li, 1996).

Global ductility of the building. The global ductility of the building, defined as the ratio of the maximum roof displacement, u, to the yielding displacement, y, was =5.6.

Maximum roof displacements and interstorey drift. The global displacement of the centre of mass of the roof of the building has a maximum value of u=0.45 m and a corresponding interstorey drift of u=0.0091.

Interstorey drifts. Fig. 18, shows the maximum interstorey drifts obtained from the step by step analyses. The largest values were min=0.017 and máx=0.021, and from these values the damage in the structural elements may be inferred, since the structural system is formed with seismically detailed RC frames. Existing literature, e.g. Penelis and Kappos (1997), shows that the values of interstorey drifts for the onset of damage are of the order of undamaged=0.005 and for the total damage, of the order of damaged=0.030.

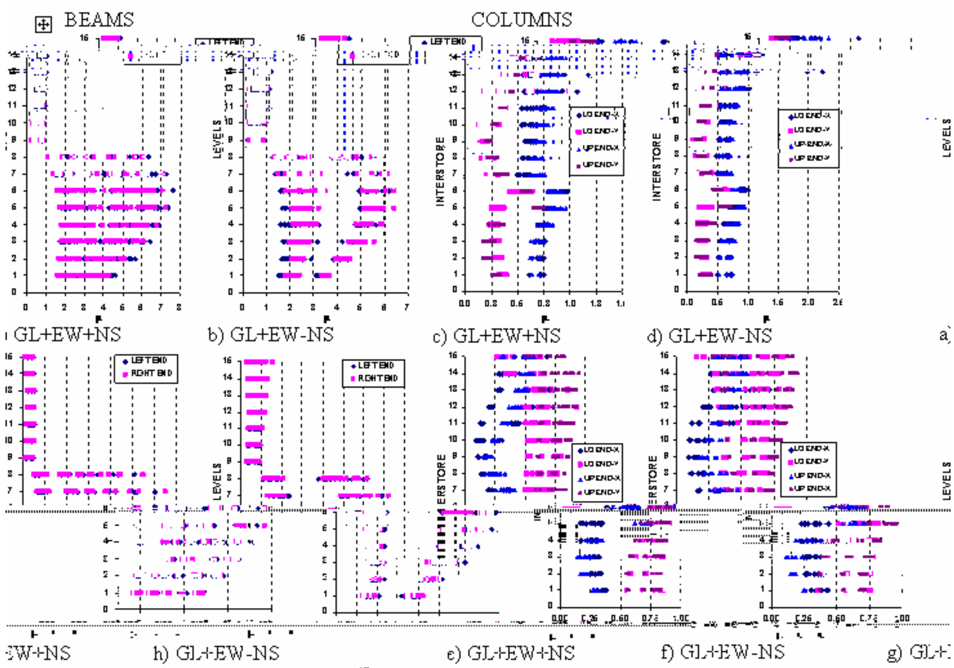

Figure 19. Ductility demands in beam and column elements

Local ductilities in beams and columns. Fig. 19 shows the rotation ductility demands at the end sections of the beams and columns in both directions. It is observed that the largest ductility demands in beams occur at levels 1 to 8 in accordance with the originally proposed damage distribution, with a maximum of beam with damage = 13.4, and that for the elements in which damage was not accepted, they remained practically elastic with only a few elements with maximum rotation ductilities of beam without damage= 1.5. For the case of columns it is observed that they remained practically undamaged with limited inelastic incursions i.e., maximum values of rotation ductility demands of column=2.2.

Damage distribution in frames. As illustration, Fig. 20 shows the final distribution of damage in the frames oriented longitudinally and transversally, correspondingly, indicating the maximum ductilities at the end sections obtained from the performed evaluations.

It is important to mention that the damage distribution proposed for design purposes is practically the same as that obtained when the structure is subjected to the design demands, with light ductility demands in other beam and column elements where damage was not supposed to have occurred. It is observed that the ductility demands at all levels maintain certain uniformity. This is not the case in the frames furthest away from the stiffness centre. These demand larger rotation ductilities at end sections, a result of the torsional effect.

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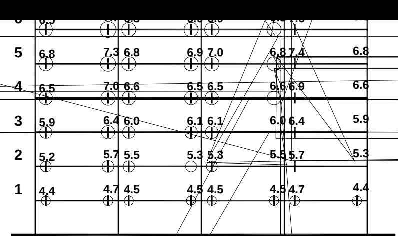
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more precise than other simplified procedures that do not consider this contribution.

The method takes into account the redistribution of forces in the structure due to non-linear behaviour of the materials.

The seismic demand is defined by means of design spectra given by smoothed spectra.

The behaviour curve is constructed using modal spectral analyses and does not depend on results of pushover analyses with particular lateral force distributions.

The application of the method is simple, as it only requires of the results of a limited number of MSA, an easy and well known procedure which may be applied using commercial analysis software.

The design strategy is consistent with the philosophy of designing a building for a given performance level involving a performance index related to an accepted distribution of damage, proportioning the structure with an adequate lateral strength function of these performance indices.

The deformation capacity of the structure is obtained by means of a proposal of damage distribution, explicitly defined in the design process.

The results obtained in the examples show that the established design objectives are satisfied in an acceptable way and that damage was controlled with the formation of hinges at end element sections without excessive plastic rotations.

The application of modal spectral analysis with accepted mode combination rules gives evaluations and designs corresponding to maximum expected performances, which, although they statistically produced design on the safe side, the validation of performances obtained though TH analyses may have important errors when the performance of the structure is validated only for a limited number of design earthquakes.

Acknowledgements

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