A DISPLACEMENT-BASED MULTI-LEVEL SEISMIC DESIGN METHOD FOR REINFORCED CONCRETE FRAME STRUCTURES

J.E. Barradas & A.G. Ayala
Instituto de Ingeniería, UNAM, México

SUMMARY:
This paper presents a multi-level displacement-based seismic design method for reinforced concrete reticular frames which explicitly considers the performance indices (ductilities) associated to the service and life protection limit states required by most of the current codes, including the control of damaged accepted to occur during design conditions. Furthermore, we comment a procedure to define uniform hazard spectra for the design levels associated to the service and life protection limit states. This design demands can be obtained for several rates of exceedence of the performance indices that define them. In this process the uncertainties in the dynamic and strength properties of the structure are considered. The steps followed in the application of this procedure are illustrated for a soft ground site in the valley of Mexico

Keywords: multilevel seismic design, uniform hazard spectra, displacement-based seismic design, limit states.

1. INTRODUCTION

Most of the current seismic design codes prescribe that a building is to develop a low or null level of structural damage and, hence, preserve its functionality, under low intensity ground motions due to frequent earthquakes, and that it must also be able to avoid local or global collapse for seismic events of greater intensity associated to a lower probability of occurrence, for the sake of providing a margin of safety for such instance. The first damage state is usually known as Serviciability Limit State (SLS) and the second is defined as Collapse Prevention Limit State (CPLS). The coupling of an accepted damage state, i.e., performance level, to be attained under a certain probabilistic seismic intensity is referred to as performance objective PO. According to the current approach of performance earthquake engineering, a structure has to necessarily satisfy the considered PO in order to be deemed to exhibit an adequate seismic performance.

Seismic design codes usually define the design spectrum associated to SLS as a fraction of the CPLS. However, as noted by Reyes (1999), this approach may not be representative of actual low intensity ground motions and is not congruent with the uniform hazard approach, where demands are given according to a probability of exceeding a demand intensity or a performance index. Furthermore, the seismic design procedures suggested by building code, do not guarantee the accomplishment of a PO, since they make use of many simplified assumptions that are not consistent with actual structural behaviour, and, even more important, their framework relies on the force based approach, which is recognized to not be able to represent appropriately the behaviour of structures in their nonlinear range. Due to this reason, various authors have developed seismic design procedures which objective is to truly satisfy a performance objective (Kappos, 2001), (Bertero, 2001), (Terzic, 2006). However, these design procedures do not aboard sufficiently all of the aspects involved in the fulfilment of a performance objective, such as the aforementioned.

This paper presents a multilevel seismic design method for reinforced concrete framed structures, which goal is to allow the assurance of a given performance objective through a single stage design process, seeking to improve the disadvantages of existing design methodologies. The proposed design method is efficient and its application procedure is straightforward, thus, making it an attractive alternative for earthquake engineering practice. Furthermore, a new approach regarding an innovative type of uniform hazard spectra to be used in the design procedure is also show. To prove the effectiveness of the design method two design examples and its validation through nonlinear step by step nonlinear analysis are illustrated.

2. CONCEPTUAL BASIS OF THE PROPOSED METHOD

The proposed seismic design method relies on the use of Inelastic Spectral Analysis, that consists in estimating the maximum inelastic response of a MDOF structure, considering idealized bilinear behaviour, by combining the maximum response of the corresponding uncoupled SDOF oscillators obtained from an inelastic spectrum. In order to do so, both the elastic and postyielding properties of the system must be known.

A SDOF bilinear oscillator is defined by its elastic frequency, ω_E, the yield strength per unit mass, Ry/m, postyielding frequency $\alpha\omega_E^2$, where α is the postyield to elastic stiffness ratio. The total inelastic displacement that the system develops, Δt, depends directly from such properties. The maximum response can be represented graphically by strength per unit mass *vs.* displacement plot, which, in the framework of the method proposed is referred as behaviour curve, see Fig. 2.1

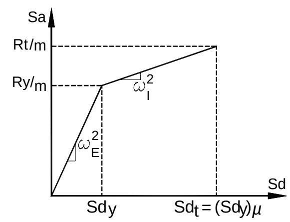
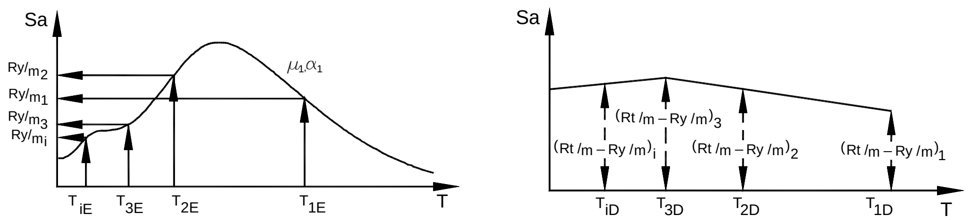


Figure 2.1. Behaviour curve of a SDOF bilinear oscillator

The procedure involves the calculation of the elastic and postyielding properties of a structure through modal analysis of two simplified linear models that represent the stages of behaviour of a MDOF system with bilinear behaviour, accepting that Inelastic Spectral Analysis can approximate in a sufficient manner, i.e., for design purposes, the maximum response of inelastic systems. Hence, the elastic and postyielding frequencies of all modes are estimated. The framework of the proposed method is conceived for structural systems that possess a predominant fundamental mode, thus, the characterization of structural behaviour is carried on through the properties of such mode. The corresponding bilinear oscillator is referred to as Reference System, RS, for which the behaviour curve is drafted.

The design approach consists in the definition of a design behaviour curve for which the considered PO is satisfied. The elastic branch is defined according to the strength and stiffness requirements to satisfy the maximum allowable story drifts stipulated for the SLS and damage onset (incipient yield) for the corresponding level of demand. The second branch is defined in such a way that, for the period and strength required by the incipient yield and a consistent damage distribution, *i.e.*, α of the reference system, the structure is able to develop the maximum allowable storey drifts when subjected to the CPLS for the associated seismic demand.

For incipient structural yielding, the influence of higher modes to the required strength of the structure is obtained from a modal spectral analysis using the inelastic spectra correspondent to the design ductility of the fundamental mode, see Fig. 2.2a. For the ultimate limit state, the influence of higher modes to the required strength of the structure is obtained from a modal spectral analysis using a spectrum built with the complementary strength, i.e., R_u/m-R_y/m, associated to each mode, which is called complementary demand spectrum, CDS. In order to determine the ordinates of such spectra, it is necessary to identify the ductility demands for higher modes, see Fig. 2.2b.



a) Inelastic spectra correspondent to the design ductility of the fundamental mode

b) Complementary demand spectrum

Figure 2.2. Design seismic demands

From preliminary results obtained of a study that is part of this investigation, it has been observed that SDOF oscillators associated to the properties of the higher modes of a MDOF structure develop a ductility that may differ significantly to that of the fundamental mode. Recognizing this fact, the authors propose that the complementary demand of each mode, from which the CDS is built, be calculated as the average value of the complementary demand calculated with a) the postyield to elastic stiffness ratio, α_i, of the ith mode, and b) that corresponding to the first mode, α_1; considering in both of the design ductility of the first mode and the yield strength of the ith mode, R_{yi}/m, obtained from the inelastic spectra correspondent to the design ductility. Preliminary results indicate that the CDS built using this approach provide a good approximation to those obtained from nonlinear analysis of the designed structures.

3. UNIFORM EXCEEDENCE RATE SPECTRA FOR GIVEN PERFORMANCE INDEX

In recent years, it has been recognized that the most rational approach for defining seismic demands is through uniform hazard spectra, that are usually defined in terms of an exceedence rate of a seismic ground motion intensity measure, i.e., acceleration. Some authors, (Ordaz, 2010), demonstrate that the exceedence rates of a structure differ from those of the seismic intensity. This is due to the fact that the properties of a structure, e.g., strength and stiffness, which define its resistance to seismic demands, are not deterministic.

For this reason, Niño (2008) proposes a particular type of uniform hazard spectra, defined in terms of an exceedence rate of ductility of a structural system, since this performance index plays an important role in the measure of structural damage on most of the current seismic design codes. The seismic demand is defined according to the relation between ductility and lateral yield strength of the system, considering the variability of lateral strength and the vibration period of the system. The use of this spectrum provides a clear notion of how frequently may the system exceed a considered limit state during its lifespan, hence, this approach is more consistent with performance based earthquake engineering. The definition of performance objective was not a part of this investigation. The optimal value of the exceedence rate of the limit states involved in a given performance objective can be obtained from a life cycle analysis, i.e., cost benefit during the lifespan of the building. However, the design spectrum for various rates of exceedence were defined, thus, they can be used of any given performance objective. The proposed method presented in this paper is conceived for the use of this spectra, thus, it is able to truly assure multilevel seismic design.

4. DESIGN PROCEDURE

4.1 Design for Serviceability Limit State (SLS)

The procedure begins by setting a preliminary sizing of the elements of the structure that provides the required lateral rigidity to the SLS. To consider the effects of cracking, the assessment of its effective inertia should be done by estimating the amount of reinforcement that would have the sections designed and the stress level for service conditions. The sizing is obtained iteratively through a series of spectral modal analysis, considering as demand an elastic seismic spectrum associated with this design limit state. The iterative process ends when the maximum drift in the structure obtained from an analysis, is approximately equal to the allowable for the SLS, $\gamma_{SLS.}$

4.2 Characterization of the incipient structural yielding.

Recent studies on the seismic behaviour of reinforced concrete structures based on rigid frames have shown that once the layout of the structure and the geometry its elements is defined, lateral deformation corresponding to incipient structural yielding is also defined (Priestley, 1998). Due to this, in a multilevel seismic design approach is necessary to consider the state of incipient yielding, since through this the lateral deformation of the structural system associated with this condition and the corresponding maximum elastic strength are defined. The design procedure for this state is as follows:

1. From the preliminary sizing and the calculated amounts of reinforcement for the elements the effective moments of inertia of the structural elements associated with a stress level corresponding to yielding, characterizing again the analytical model of the structure are estimated.

2. The interstorey where the maximum drift γ_{max}, which produces yielding occurs is located. In this state, the structure is on the limit of its elastic behaviour so that the maximum modal responses are defined for the corresponding ordinates of an elastic spectrum. According to this and considering that for the seismic demand associated with the CPLS the structure will reach the state of incipient yielding, it is considered that the location of the most demanded interstorey may be determined from the modal spectral analysis of the structure with properties corresponding to this incipient yielding state, considering for this purpose, the elastic seismic demand associated with the CPLS.

3. The value of yield drift in this interstorey is obtained using the following empirical equation proposed by Priestley (1998), Eqn.4.1:

$$\gamma_y = \frac{0.30\varepsilon_y l_v}{h_v} \tag{4.1}$$

where: ε_y, is the yielding tensile strain of steel reinforcement of the beams in the interstorey, h_v is its total depth and l_v the axis to axis length of the columns.

4. The storey displacements obtained from the modal analysis spectral are scaled in such an amount, *fe,* that γ_{max} equals γ_y. From the scaled roof displacement, $\Delta^{roof} fe$, the yield displacement, Sd_y, of the RS is obtained, Eqn. 4.2.

$$Sd_y = \frac{\Delta^{roof} fe}{FP_1^{roof}} \tag{4.2}$$

where: FP^{roof} is the product of the participation factor of the fundamental mode of the structure by the amplitude of the modal shape at the roof. It may be observed that *Sdy* was obtained from the total displacements of the structure, however, when a modal combination rule is used to obtain the total response of regular structures up to the median of the fundamental period, the displacement response is little sensitive to the contribution of higher modes, explaining why for this type of structure this approach introduces negligible errors. From this displacement the corresponding spectral acceleration is obtained by Eqn. 4.3.

$$Sa_y = \omega_{1E}^2 Sd_y \tag{4.3}$$

where ω_{1E} is the natural frequency of the fundamental mode of the structure in the incipient yielding state.

4.3. Design for the Collapse Prevention Limit State (CPLS)

The performance criterion for this limit state is to avoid partial or total collapse of the structural system. To achieve this objective at spectral level, the global ductility demand under the design earthquake corresponding to this limit state should be less than or equal to ductile capacity of the structure. At local level, this objective is considered met if the maximum drift is less than or equal to the design drift. The design procedure for this limit state is as follows:

1. A distribution of structural damage, consistent with the damage state representative of the CPLS. Damage considered in the model by introducing plastic hinges at critical sections of the structural elements. A possible and desirable distribution of damage with this characteristic is one that follows the strong column- weak beam strong design philosophy as shown in Fig. 4.1. Once the distribution of damage is introduced in the model an eigen value analysis is carried out to obtain the fundamental period of the structure in its damaged state, T_{1D}, the storey displacements corresponding to the fundamental mode and the resulting maximum drift, γ_{max}. With the values of the periods of the fundamental mode of the structure without damage and with damage T_{1E} y T_{1D}, respectively, the ratio of postyielding to initial stiffness, α, is calculated using Eqn. 4.4:

$$\alpha = \left(\frac{\omega_D^2 m}{\omega_E^2 m} \right) = \left(\frac{4\pi^2 / T_D^2}{4\pi^2 / T_E^2} \right) = \left(\frac{T_E}{T_D} \right)^2 \tag{4.4}$$

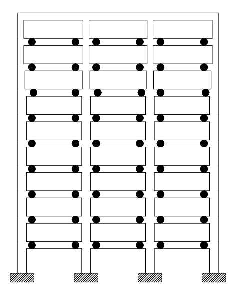


Figure 4.1. Potential distribution of damage consistent with the collapse prevention limit state

2. A value for the drift, γ_p, is proposed within the range prescribed by the code for this limit state, and the storey displacements obtained in the previous step are scaled by an amount, fe, such that the value γ_{max} be equal γ_p. From the scaled roof displacement, $\Delta_1^{roof} fe$, the spectral displacement of the RS corresponding to the CPLS, Sd_{CPLS}, is obtained using Eqn. 4.5. The result of dividing this displacement by that obtained for the incipient yielding state, Sd_y, defines the displacement ductility, μ, of the RS. The value of the ductility must be less than or equal to the maximum established by the code.

$$Sd_{CPLS} = \frac{\Delta^{rooof} fe}{FP_1^{roof}} \tag{4.5}$$

3. An inelastic spectrum is calculated resistance per unit of mass corresponding to the values of μ, α, and a fraction of critical damping, ξ, from elastic spectrum representing the seismic demand associated CPLS. In this spectrum, the ordinate corresponding to the period T_{1E}, R_y/m is read. If

R_y/m is approximately equal to Sa_y, this strength will be adequate to ensure that the RS develops the global ductility, μ, required for the seismic demand associated to the CPLS.

If it results that R_y/m is different from Sa_y a little different distribution of damage in accordance with the CPLS and/or a modification to the value of γ_p must be proposed, in order to adjust the values of μ and α, so that R_y/m be approximately equal to Sa_y.

4. The spectral acceleration for the RS corresponding to the CPLS is obtained using Eqn. 4.6. At this stage of the analysis all the response parameters of the behaviour curve are defined.

$$Sa_{CPLS} = Sa_y \left[1 + \alpha \left(\mu - 1\right)\right] \tag{4.6}$$

5. Next, the design strength of the structural elements damage was expected to occur and part of the design strength of the elements on which damaged under design conditions is not expected are obtain. For this, a modal spectral analysis of the structure is carried out with properties corresponding to incipient yielding and with gravitational loads, considering the inelastic spectrum calculated in step 3 of this design stage.

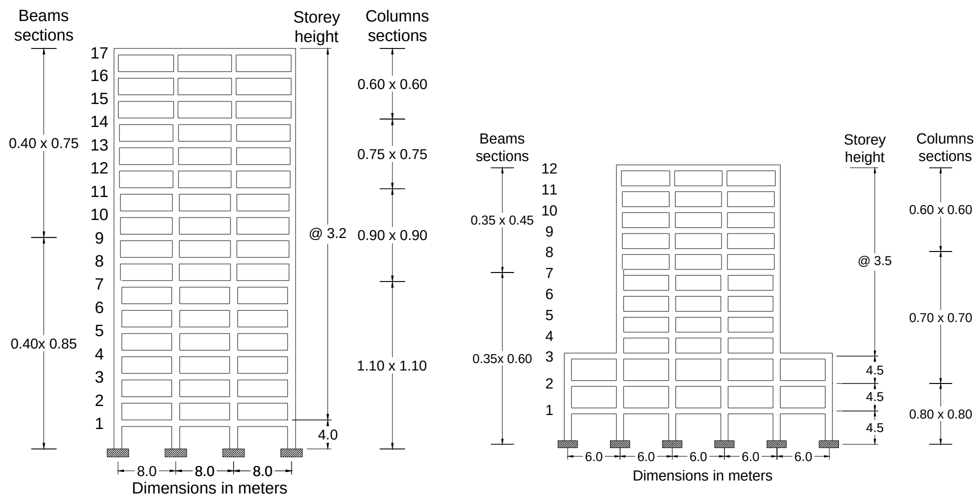
It is important to note that to ensure consistency with the procedure followed to obtain the maximum interstorey drift at the state of incipient yielding, instead of using the inelastic spectrum, elastic design spectrum corresponding to the seismic demand associated with the CPLS, scaled in such a way that for the fundamental period of the structure, T_{1E}, its corresponding spectral acceleration equals R_y/m, should be used. However, as mentioned above, by applying a modal combination rule to obtain the total response of regular structures up to median fundamental period, the displacement response is approximately insensitive to the contribution of the higher modes, so that, for structures with these characteristics virtually identical results with both spectra are obtained.

6. The additional strength of the elements for which damage is not expected, a modal spectral analysis of the structure with damage and without gravity loads is performed, considering the additional seismic demand.

7. The design strengths of the structural elements for which damage is not expected are obtained from the superposition of the elements forces obtained from the two previous modal spectral analysis.

5. APPLICATION EXAMPLES

To illustrate the practical implementation of the proposed design procedure two reinforced concrete frames, one regular and the other irregular in elevation whose configurations are shown in Fig. 5.1, are designed.



a). 17 levels frame b). 12 levels frame

Figure 5.1. Geometric configurations of the two example frames

The design gravity loads considered are those required by the building code of Mexico City for office buildings (GDF. 2004). The mechanical properties of the materials used are those recommended by the code for the seismic design of ductile reinforced concrete structures. The performance indexes stipulated by the regulations for this type of structural systems are: for the SLS maximum interstorey drift of 0.004. For the CPLS of ductile reinforced concrete frames interstorey drifts contained within the range 0.015 to 0.03 and displacement ductilities from 2 to 4 are stipulated.

To validate the proposed procedure and show the approximation of the results obtained through its application, the results obtained are compared with those from the nonlinear step by step dynamic analysis, considering as seismic demands for the CPLS the E-W component of the record earthquake September 19, 1985 obtained at the SCT station in Mexico City and as demand associated with the SLS, the E-W component of the record the April 25, 1989 earthquake obtained in the same station. It is noteworthy that these earthquakes are considered representative of earthquakes associated with their corresponding limit states in the valley of Mexico.

The procedure for the preliminary sizing of the structural elements of both frames, was applied so that the maximum interstorey drift produced by the seismic demand associated with the SLS be equal to the allowable for this state, γ_{SLS}. Fig. 5.1 shows the dimensions of their cross sections. The fundamental periods of the frames corresponding to this limit state, T_{1s}, are shown in Table 5.1. The interstorey drifts corresponding to incipient yielding, γ_y, shown in Table 5.1 were obtained for both frames using the preliminary sizes. The values of yield displacement, Sd_y, and yield strength, Sa_y, corresponding to the RS are obtained from this deformation state. The fundamental periods of the frames corresponding to a state of incipient yielding, T_{1E}, are shown in Table 5.1. When proposing the damage distributions shown in Fig. 6.2 and the values of interstorey drifts, γ_{CPLS}, shown in Table 5.1, both corresponding to the CPLS, the strengths per unit mass R_y/m, read from the inelastic spectra constructed with the associated values of α and μ shown in Table 5.1, and corresponding to their periods T_{1E}, were approximately equal to their respective Sa_y. In their damaged states the frame have fundamental periods, T_{1D}, shown in Table 5.1. The same table shows the response parameters which define the behaviour curves of the RS for the frames designed.

Because the values of displacement ductility required for both designs are within the range prescribed by the regulations, it was not necessary to modify the initial period of the structures. Once the performance curves for the RS for both frames are calculated, the strength spectra for the two stages of analysis are defined and the corresponding modal spectral analysis carried out, giving element forces that add up to the design strengths of beams and columns of the frames.

Table 5.1. Dynamic properties and design parameters of the frames designed

	T_{1s} (s)	γ_y	T_{1E} (s)	Ry/m (m/s^2)	Sdy (m)	γ_{CPLS}	α (%)	μ	T_{1D} (s)	Rt/m (m/s^2)	Sdt (m)
17 levels frame	1.90	0.0067	2.24	1.716	0.219	0.021	7.01	2.42	8.48	1.886	0.531
12 levels frame	1.72	0.0092	2.06	1.997	0.214	0.024	9.81	2.25	6.56	2.243	0.482

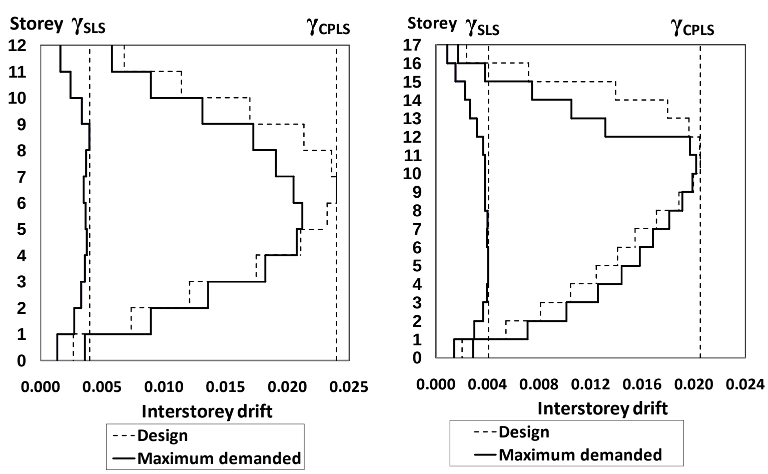
6. RESULT VALIDATION

To evaluate the design procedure, the nonlinear step by step dynamic analyses for the design earthquakes associated with each limit state are carried out. The performance parameters established for each limit state are compared with the demanded response parameters of the nonlinear analysis , considered as exact, i.e.:

- The maximum interstorey drift demanded in the structure the earthquake associated with the SLS should be approximately equal to the permissible for this limit state, γ_{SLS}.
- The distribution of damage and the overall ductility demand for the earthquake associated with the CPLS should be developed similar to those established. Also, the maximum distortion demanded should be approximately equal to the expected for this limit state, γ_{CPLS}.

These analyses were performed using the program DRAIN 2DX (Prakash et al., 1993). The ± 10% relative errors in the response parameters calculated are considered acceptable.

Fig. 6.1 compares the design interstorey drifts with the maximum demanded. For both frames, the maximum drift demanded by the design earthquake associated with SLS, Fig. 6.1a, is approximately equal to the design for this limit state. The drifts demanded in the lower and middle interstoreys by the design earthquake associated with the CPLS, are moderately higher than the design and in the upper interstoreys the design drifts are significantly higher than the demanded, see Fig. 6.1b. These trends are due to the fact that the design drifts were calculated from the displacement configuration of the fundamental mode of the damaged structure, ignoring the contribution of higher modes, which is more significant in the upper levels as the fundamental period of the structure increases due to damage. Despite this, the maximum distortion demanded in the 17 -storey frame is 4% smaller than the design for this limit state, and 13% smaller in the 12 -storey frame. For practical purposes this error may still be considered acceptable.



a) b)

Figure 6.1. Comparison of the design interstorey drifts with the maximum demanded for a.) 12 levels frame and b.) 17 levels frame

Fig. 6.2 compares for the two example frames the damage distributions expected for design with the developed for the design earthquake associated with the CPLS. It is observed that both frames presented damage in several elements that were not expected to be damaged, however, the magnitude of the inelastic deformation of these elements were small.

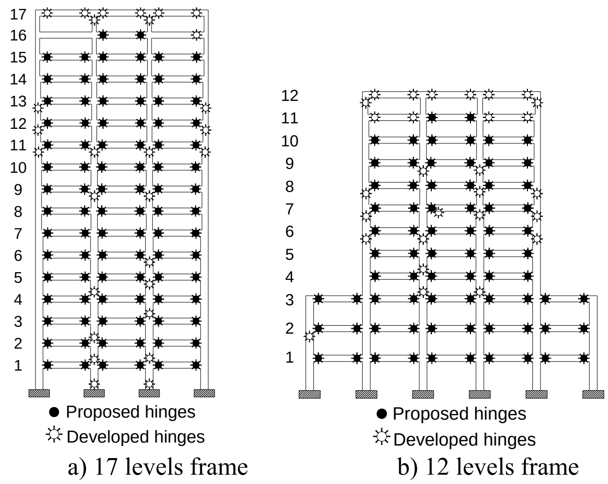


Figure 6.2. Comparison of the damage distribution expected with that produced by the design earthquake associated with the CPLS

7. CONCLUSIONS AND RECOMMENDATIONS

This paper illustrated the application of a procedure for the displacement-based seismic design of structures, which allows the explicit and simultaneous consideration of two design limit states and corresponding seismic design levels. To validate the results obtained nonlinear step by step dynamic analyses were performed considering as seismic demand for each limit state the same earthquakes records considered in design, but it is clear that, for practical design applications the design spectra from the design regulations should be considered. From the analysis of the results obtained the following conclusions may be extracted:

1. For the two frames, the relative differences between the maximum drifts demanded and those recommended for design for the two limit states are acceptable. For the 17 -storey frame, the design interstorey drifts are considerably higher than the demanded during the response to the earthquake used the limit state of collapse prevention, however, considering that the design goal is to limit the value of the maximum drift, the results can be considered acceptable.

As mentioned before, this fact is due to the use of the configuration of displacements of the fundamental mode of the damaged structure to obtained the design drifts. This consideration, however, leads to results more approximate than those obtained by applying most of the current displacement-based design procedures, which assume various lateral displacement configurations such as linear, quadratic or empirical that only apply to particular cases.

3. In both frames the distribution of damage developed was similar to the proposed, and even though some structural elements not expected to be damaged developed plastic hinges, their inelastic deformation demands were small. In fact, this result was expected, as these elements are actually designed to reach a state of incipient yielding when subjected to a seismic demand associated with this limit state. Therefore the design objective of controlling the damage in structural elements was reasonably accomplished.

4. From the comparison of the performance indexes required for the limit states of incipient yielding and collapse prevention of the frames designed, it is observed that due to the different layouts and heights the frames require different elastic stiffnesses and strengths and thus develop different ductility values to simultaneously satisfy the requirements of each limit state. Furthermore, since the values of the design parameters are within the range stipulated by the code, the designs are acceptable. These results contrast with one of the main design criteria stipulated in the code, which indicates that structures of different heights and layouts can be designed for equal values of the design parameters if they meet the same seismic detailing recommendations.

5. This paper does not show the relative errors in the design forces of the structural elements, nevertheless the good approximations obtained suggest that the approximation of method described to determine the complementary resistances of the higher modes, is reasonable, even though the need of developing a more rational analytical or semiempirical procedure is recognized.

6. In general, the structures used as examples met the performance parameters established for each limit state, however to reasonably demonstrate the potential of the proposed design procedure, it would have necessary to obtain similar results by applying the design method to an exhaustive number of structures with different characteristics.

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