Trends in the Displacement-Based Seismic Design of Structures, Comparison of Two Current Methods

H. Castellanos & A. G. Ayala
Instituto de Ingeniería, Universidad Nacional Autónoma de México, México

SUMMARY:
This paper presents a comparative investigation of the Direct Displacement-Based Design Method and the Displacement-Based Seismic Design Method with Damage Control. Initially the most important characteristics of each method as described by presenting the sequence of steps required for their application. The example used for the comparison is a 12 storey reinforced concrete irregular frame designed considering as seismic demand the SCT record of the 1985 Michoacan earthquake in Mexico The designs obtained with both methods are evaluated by comparing the respective reinforcement requirements, lateral displacements and interstorey drifts. The adequacy of the designs is assessed by comparing the accuracy of the performances obtained from the response results of step by step nonlinear dynamic analyses of the designed structures, subjected to the same seismic as they were designed for, with the performance used as design target. Based on the results of this comparison conclusion are given.

Keywords: displacement-based seismic design, damage control, reinforced concrete structure

1. INTRODUCTION

The analysis of the effects of recent destructive earthquakes has shown that the performances of many structural systems have not been in accordance with their design objectives due to the unexpected and on occasions excessive damage presented in its structural and non-structural elements, attributable to deficiencies and/or inconsistencies in the current design method, based on forces, and to the uncertainty in the definition of more realistic seismic demands.

For this reason, the last two decades have witnessed numerous research efforts to develop new ideas to take into account in a consistent way the seismic demands and to develop more rational seismic design methods. These efforts aim to develop methods easy to implement and to be used in practice and, above all, capable to produce designs which guarantee the performances produced when the structures are subjected to seismic demands, as those considered in their design, are in consistency with the those considered as targets. Accordingly, several design methods based on performance indices such as ductility, energy, damage indices, displacements and deformations, have emerged. Currently, the most widely accepted, due to their explicit consideration of relationship between displacements and structural damage, are the displacement-based methods. Two of these methods, characterized by their practical orientation, ease of implementation and capability of producing reliable designs are the Direct Displacement-Based Design Method, DDBD, (Priestley *et al.*, 2007) and the Displacement-Based Seismic Design Method with Damage Control, DBSDDC, (Ayala *et al.*, 2012). Current literature contains several other displacement-based design methods such as the method developed by Sullivan (2011) involving for the definition of the design strength the use of an empirical ductility-dependent energy dissipation factor and the method of Chopra and Goel (2001) which uses as seismic demand inelastic displacement spectra. Two other methods that use as performance index element deformations, which are somehow related to displacements, are the Deformation-Based Seismic Design Method, developed by Kappos and Stefanidou (2010) and the Displacement-Based Seismic

Design Procedure, developed by Panagiotakos and Fardis (1999), both only applicable to reinforced concrete structures, among other procedures.

Based on these developments, this paper presents a comparison of two of the methods mentioned above, the DDBD by Priestley *et al*. (2007) and the DBSDDC by Ayala *et al.* (2012), chosen for their potential and wide dissemination in the field of seismic design. To illustrate the application of these methods and evaluate the designs they produce, a 12-storey reinforced concrete irregular frame is designed, considering as design demand, the EW component of the SCT record of the 1985 Michoacan earthquake in Mexico. The designs obtained, are evaluated by comparing reinforcement requirements (strength), lateral displacements, interstorey drifts and overall structural performance. The relative advantages and limitations of each design method together with the versatility and the rationality of their formulations are discussed. The accuracy of their results is assessed by comparing the performances extracted from the results of the step by step nonlinear dynamic analysis of each of the structures designed with the target design performance. Finally, some conclusions about the different characteristics evaluated and an opinion on which method offers the most relevant advantages of the displacement based design of structures are given.

2. DISPLACEMENT-BASED SEISMIC DESIGN PROCEDURE

From the evaluation of the current methods for the displacement-based seismic design of building structures it may be concluded that some are essentially variants of the method proposed by Priestley *et al*. (2007), modifying only some parameters, as is done in the method proposed by Chopra and Goel (2001), where inelastic displacement spectra are used instead of elastic spectra reduced by an equivalent damping ratio. Other methods do not, strictly speaking, control displacements but they rather control deformations as is the case of the methods proposed by Kappos and Stefanidou (2010) and Panagiotakos and Fardis (1999). Other displacement-based methods, such as the method proposed by Sullivan (2011), simplify, for a particular type of buildings, the original method of Priestley *et al.*, (2007) using energy concepts.

In what follows, due to their relative importance, potential of practical application, and quality of results obtained, the formulations of two of the methods mentioned above, DDBD (Priestley *et al.* 2007) and DBSDD (Ayala *et al*., 2012), are discussed further.

2.1. Direct Displacement-Based Design Method

In the DDBD method by Priestley *et al*. (2007), the nonlinear multi-degree of freedom system, MDOF, representing the building, is transformed into an equivalent single degree of freedom, SDOF, system using the concepts of the equivalent structure proposed by Shibata and Sozen (1976) This system has an effective stiffness, associated to the secant at maximum displacement of the MDOF system, and an equivalent viscous damping to consider the energy dissipated by the structural elements through hysteresis.

The application of this method involves the following steps:

1. Calculation of the design storey displacement. The design floor displacements of the frame are proportionally related to a normalised inelastic mode, δ_i, by Eqns. 2.1 and 2.2.

$$\Delta_i = \delta_i \left[\frac{\Delta_c}{\delta_c}\right] \tag{2.1}$$

$$\text{for } n \le 4: \quad \delta_i = \frac{H_i}{H_n} \tag{2.2a}$$

$$\text{for } n > 4: \quad \delta_i = \frac{4}{3}\left[\frac{H_i}{H_n}\right]\left[1 - \frac{H_i}{4H_n}\right] \tag{2.2b}$$

where i= 1 to n are the storeys, Δ_c is the displacement of the critical storey and H_i and H_n are the heights of the ith storey and the total height of the structure respectively.

Eqn. 2.1 is valid when the amplification of drift due to the contribution of higher modes is negligible. To consider the contribution of higher modes to response, the displacements Δ_i, must be multiplied by the factor ω_θ, defined by Eqn. 2.3.

$$\omega_\theta = 1.15 - 0.0034H_n \leq 1.0 \quad (H_n \text{ in m}) \tag{2.3}$$

2. Calculation of the equivalent SDOF design displacement, Δd, using the Eqn. 2.4.

$$\Delta_d = \sum_{i=1}^{n} (m_i\Delta_i^2) / \sum_{i=1}^{n} (m_i\Delta_i) \tag{2.4}$$

where m_i is the mass at height H_i associated with displacement Δ_i.

3. Calculation of the mass, M_e, of the equivalent SDOF system using Eqn. 2.5.

$$m_e = \sum_{i=1}^{n} (m_i\Delta_i)/\Delta_d \tag{2.5}$$

4. Calculation of the effective height, H_e, using Eqn. 2.6.

$$H_e = \sum_{i=1}^{n} (m_i\Delta_iH_i) / \sum_{i=1}^{n} (m_i\Delta_i) \tag{2.6}$$

5. Calculation of the yield displacement, Δ_y, using the Eqns. 2.7 and 2.8.

$$\theta_y = 0.5\varepsilon_y \frac{L_b}{h_b} \quad \text{for reinforced concrete frames} \tag{2.7}$$

$$\Delta_y = \theta_y H_e \tag{2.8}$$

where L_b is the beam span between column centrelines, h_b is overall beam depth and ε_y is the yield strength of the flexural reinforcement.

Eqns. 2.7 and 2.8 are valid for regular frames. For irregular frames, where the reason of irregularity is not the length of the spans, to obtain the yield drifts for the critical spans, Eqn. 2.7 must be used, and to obtain the yield displacements of the system, Eqn. 2.9.

$$\Delta_y = \frac{2M_1\theta_{y1} + M_2\theta_{y2}}{2M_1 + M_2} H_e \tag{2.9}$$

where M_1 and M_2 are the resisting moments of the beam and the subindices 1 y 2, the span under study.

6. Calculation of the design displacement ductility of the equivalent SDOF, μ, using Eqn. 2.10.

$$\mu = {}^{\Delta_d}\!/_{\Delta_y} \tag{2.10}$$

7. Calculation of the viscous damping of the equivalent SDOF, using Eqn.2.11.

$$\xi_{eq} = 0.05 + 0.565\left(\frac{\mu - 1}{\mu\pi}\right) \quad \text{for reinforced concrete frames} \tag{2.11}$$

8. Calculation of the effective period of the substitute structure, T_e. The value of Δ_d is extracted from displacement spectrum associated to the viscous damping ratio of the equivalent SDOF.

9. Calculation of the effective stiffness of substitute structure, $K_{e,}$ associated to maximum displacement response, using Eqn. 2.12.

$$K_e = \frac{4\pi^2 m_e}{T_e^2} \tag{2.12}$$

10. Calculation of the design base shear force, V_{Base}, using Eqn. 2.13.

$$F = V_{Base} = K_e \Delta_d \tag{2.13}$$

11. Distribution o the design base shear among all floors in proportion to their masses and assumed displacements, using Eqn. 2.14.

$$F_i = V_{Base}\,(m_i\Delta_i) / \sum_{i=1}^{n} (m_i\Delta_i) \tag{2.14}$$

To consider the contribution of higher modes to response, the distribution of base shear among all levels must be done with Eqn. 2.15, where F_t=0.1V_{Base} at roof level and F_t=0 at all other levels.

$$F_i = F_t + 0.9V_{Base}\,(m_i\Delta_i) / \sum_{i=1}^{n} (m_i\Delta_i) \tag{2.15}$$

Once the force vector is calculated, the design forces of the elements are determined from a conventional linear static analysis of the structure subjected to this force vector and assumed stiffnesses for the members consistent with their expected ductility level. The final design of the structural elements is defined from a capacity design aimed to guarantee that the mechanism, required to occur under design demands (*i.e.,* strong column - weak beam behaviour), is attained.

2.2. Displacement based seismic design method with damage control

This method is based on the assumption that an approximation to the performance of a nonlinear MDOF structure may be obtained from the performance of a simplified nonlinear SDOF reference system, generally associated to the fundamental mode of the buildings (Ayala, 2001). The principle of this design method is that the nonlinear capacity curve of a MDOF structure can be approximated by a bilinear curve using the equivalence of deformation energies corresponding to the real and the bilinear capacity curves, and that, in accordance with basic principles of structural dynamics, the bilinear behaviour curve of a reference SDOF system normally associated to the fundamental mode of the structure may be extracted from this capacity curve. The behaviour curve is obtained from the results produced by two conventional modal spectral analyses, one for the elastic phase of behaviour, structure without damage, and the other for the inelastic phase, structure with damage. The slope of the first branch of the behaviour curve represents the stiffness properties of the reference SDOF system in the elastic range, and the slope of the second branch, the stiffness corresponding to the inelastic range. The characteristics of this second branch are defined by the assumed damage distribution associated to the proposed maximum displacement of the given performance level using the philosophy of strong column - weak beam.

The application of the design method involves the following steps:

1. Definition of the structural configuration and the dimensions of the sections of the elements from a preliminary design of the structure using gravity loads and lateral forces in a rough design method based on forces and engineering judgment.

2. Execution of a modal analysis of the undamaged structure designed in the previous step. From this analysis the fundamental period of the structure, T_E, is obtained and from it the slope of the elastic branch of the idealized bilinear behaviour curve of this SDOF system.

3. Definition of a rational damage distribution for a given design performance level in accordance with the characteristics of the structure and the design demands and the location of the element sections of greatest demands. Structural damage is introduced at the ends of the elements where damage is accepted to occur under design conditions adding hinges with rotational stiffness equal to a reduced bending stiffness of the damaged element section. With this modified structural model, referred to as damaged model, a second modal analysis is carried out to obtain the fundamental period of the damaged structure, T_I, and, from it, the slope of the second branch of the idealized bilinear behaviour curve of the reference SDOF system.

4. Calculation of the target roof displacement, d_u, using the deformed configuration of the damaged model and the design interstorey drift prescribed for the considered performance level.

5. Calculation of an approximation of the yield roof displacement, d_y, using Eqns. 2.16 to 2.18 (López, 2010), the deformed shape of the undamaged structure and the properties of the elements obtained from the preliminary design through or any other acceptable approximation, *e.g.* the Eqn. 2.8 or 2.9 proposed by Priestley (Priestley *et al.*, 2007) to obtain the yield displacement of the equivalent SDOF system:

$$d_y = \frac{\delta_n}{\psi_n} \tag{2.16}$$

where

$$\delta_n = \frac{0.3\varepsilon_y L_l \left[\frac{I_{v1}}{L_1} + \frac{I_{v2}}{L_2} + \frac{I_{cn}}{H_n} + \frac{I_{cn+1}}{H_{n+1}}\right]}{h_{v1}\left[\frac{I_{cn}}{H_n^2} + \gamma_o \frac{I_{cn+1}}{H_{n+1}^2}\right]} \tag{2.17}$$

$$\gamma_o = \frac{\psi_{n+1}}{\psi_n} \tag{2.18}$$

where: δ_n is the yield interstorey drift at the floor where maximum drift occurs; ψ_n is the drift obtained from a modal spectral analysis of the undamaged structure at the storey where maximum drift occurs, normalized by the maximum roof displacement; ε_y is the yield strain of the reinforcing steel; L_l is the length of the span to the left of the node nearest to the centre of the storey where maximum drift occurs; L_2 is the length of the span to the right of such node; H_n is the height of the storey where maximum drift occurs; H_{n+1} is the height of the storey above the storey where maximum drift occurs; $I_{v1}$and I_{v2} are the moments of inertia of the beams in the spans 1 and 2, respectively; I_{cn} and I_{cn+1} are the moments of inertia of the columns of the storeys n and n+1, respectively; and h_{v1} is the beam depth at span 1.

6. Calculation of the target yield and ultimate spectral displacements of the reference SDOF system, S_{du} and S_{dy} respectively, corresponding to the fundamental mode using the results of modal spectral analysis, its ductility, μ, and its post-yielding to initial stiffness ratio, α, using Eqns. 2.19 and 2.20:

$$\mu = {}^{S_{du}}/_{S_{dy}} \tag{2.19}$$

$$\alpha = {}^{K_I}/_{K_E} \tag{2.20}$$

7. Extraction of the ultimate spectral displacement associated to T_E, from the design displacement spectrum for given μ and α,. Finally, this spectral displacement and the target spectral displacement of the frame, S_{du}, are compared. If the last value obtained is equal or approximately equal to the target, the design is considered satisfactory; otherwise, the initial period of the structure, T_E, and/or the damage distribution needs to be *ad hoc* modified.

8. Once the target displacement of the structure is guaranteed, determination of the yield strength, R_y, for the period that satisfies the target displacement from the inelastic strength spectrum, ISS, corresponding to the values of μ and α previously calculated.

9. Calculation of the ultimate strength, R_u, of the reference system using Eq. 2.21.

$$R_u = R_y[1 + \alpha(\mu - 1)] \tag{2.21}$$

10. Determination of the design forces of the elements carried out three different analyses: a gravity load analysis of the undamaged structure, a modal spectral analysis of the undamaged structure using the elastic design spectrum scaled by the ratio of the strength per unit mass at the yield point of the behaviour curve and the elastic pseudo-acceleration for the initial period, λ_1, and a modal spectral analysis of the damaged structure using the elastic spectrum scaled by the ratio of the difference of ultimate and yield strengths per unit mass and the pseudo-acceleration for the period of the damaged structure, λ_2. The design forces are obtained by adding the forces due to gravity loads and the forces of the modal spectral analyses of the undamaged and damaged structure.

11. Determination of the design of structural elements in accordance with the forces obtained from the analysis of the simplified models using the applicable design rules. The design process must be carried out in such a way that the design criteria of the code do not alter significantly the expected performance.

In this design method the slopes (stiffnesses) of the initial and post-yielding branches of the behaviour curve are defined in terms of the modal mass associated to the undamaged structure and the corresponding periods. The contribution of the higher modes to response is directly included in the modal spectral analyses using an adequate modal combination rule, e.g., CQC. The strengths of the elements that should not be damage under design conditions are directly obtained without having to modify the design of the elements considering concepts of capacity design as it is done in the DDBD method.

3. APPLICATION EXAMPLE

To illustrate the application of the two design methods investigated in this paper, a 12 storey irregular reinforced concrete frames (see Fig. 3.1.), is designed. This frame configuration and its properties were taken from Priestley *et al*. (2007).

The nominal properties of the materials used in the design are: for the concrete, a compressive strength $f'c=3.00*10^4$ kN/m^2, a modulus of elasticity $Ec=27.00*10^6$ kN/m^2, and a weight density $\gamma=23.53$ kN/m^3, and for the steel reinforcement a yield stress $fy=4.50*10^5$ kN/m^2 and a modulus of elasticity $Es=2.00*10^8$ kN/m^2. Based on the results of the preliminary design of the frame the sections of the structural elements were defined, for all columns 0.50x0.50 m, and 0.30x0.60 m and 0.25x0.60 m for beams at floors 1 to 11 and floor 12 respectively. The moments of inertia of the columns were 75% of the calculated for the gross sections, whereas, for the beams half of those calculated using the gross sections. The floor masses were 65.00 kN-s^2/m for the first level, 60.00 kN-s^2/m for the second to the eleventh levels and 70.00 kN-s^2/m for the roof level. A design drift limit of 0.025 was used.

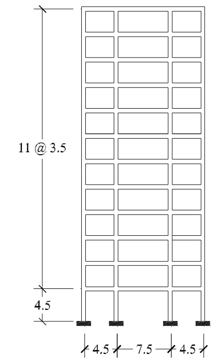


Figure 3.1. Geometry of the studied 2D moment resisting frame (in m)

The seismic demand considered was a response spectra corresponding to the record of the SCT E-W component of the 1985 Michoacán earthquake, in Mexico. To validate the results of the designs, the displacements of the frame were calculated using non linear step by step analyses under the same seismic demand for which they were designed. These non linear analyses were carried out with the program DRAIN 2D-X (Prakash *et al.,* 1993) using the following considerations: a) nearly elasto-plastic bilinear stable hysteretic behaviour for all beams and columns, b) proportional damping matrix with , c) axial load – moment interaction for all columns, d) P-Δ effects no considered and e) yield moments for beam and columns those obtained from the design method, standardizing the design level per level in such a way that all columns had the same strength, equal strength for extreme beams and different strength for the central beam which could be the same than for the extreme beams.

3.1. Consideration and results of the Direct Displacement-Based Design Method

Table 3.1 shows the data required in the determination of the displacement profile of the MDOF structure as well as the target displacement and the effective height of the equivalent SDOF system.

Table 3.1. Calculation for design displacement and effective height

Storey	h_i	Height H_i (m)	Mass, m_i (kN-s^2/m)	δ_i	Δ_i (m)	$m_i\Delta_i$	$m_i\Delta_i^2$	$m_i\Delta_iH_i$	Fi (kN)	Vsi (kN)
12	3.50	43.00	70.00	1.00	0.828	57.95	47.98	2492.01	385.53	385.53
11	3.50	39.50	60.00	0.94	0.781	46.87	36.61	1851.34	182.09	567.62
10	3.50	36.00	60.00	0.88	0.731	43.84	32.04	1578.41	170.34	737.96
9	3.50	32.50	60.00	0.82	0.677	40.60	27.47	1319.52	157.74	895.70
8	3.50	29.00	60.00	0.75	0.619	37.14	22.99	1076.98	144.28	1039.98
7	3.50	25.50	60.00	0.67	0.558	33.45	18.65	853.09	129.97	1169.95
6	3.50	22.00	60.00	0.60	0.493	29.55	14.56	650.15	114.81	1284.77
5	3.50	18.50	60.00	0.51	0.424	25.43	10.78	470.47	98.80	1383.57
4	3.50	15.00	60.00	0.43	0.351	21.09	7.41	316.34	81.93	1465.50
3	3.50	11.50	60.00	0.33	0.275	16.53	4.55	190.08	64.22	1529.72
2	3.50	8.00	60.00	0.24	0.196	11.75	2.30	93.99	45.65	1575.36
1	4.50	4.50	65.00	0.14	0.113	7.31	0.82	32.91	28.41	1603.77
0	0.00	0.00		0.00	0.000				0.00	
Sum			735.00			371.51	226.16	10925.29	1603.77	

The displacement profile is obtained using Eqns. 2.1 and 2.2. This profile shows that the maximum drift occurs between the ground and the first floors. The drift amplification factor due to the contribution of higher modes, ω_θ=1.004, was calculated with Eqn. 2.3 due to its negligible influence, a ω_θ=1 was used for this example.

For the yield strain for the steel reinforcement (ε_s) a yield strength, $f_{ye} = 1.1f_y$ was used. Table 3.2 shows the results obtained in the application of this design method.

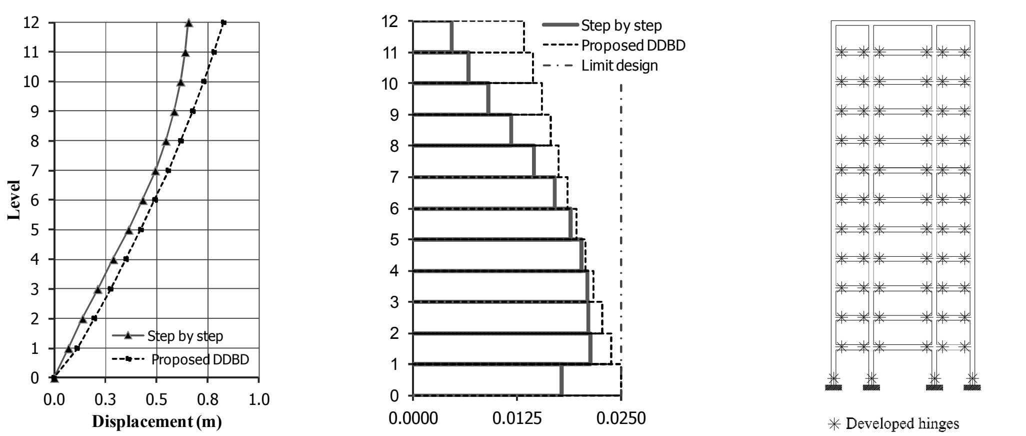
Table 3.2. Design parameters

Δ_d (m) Eqn. 2.4	m_e (kN-s^2/m) Eqn. 2.5	He (m) Eqn. 2.6	Δ_y (m) Eqns. 2.7* and 2.9	μ Eqn. 2.10	ξ_{eq} (%) Eqn. 2.11	T_e** (s)	K_e (kN/m) Eqn. 2.12	V_{Base} (kN) Eqn 2.13
0.609	610.00	29.41	0.334	1.825	13.13	3.00	2633.46	1603.77

* The beam depth and the moment capacity in all bays are constant
** From the displacement spectrum used for design

Using Eqn. 2.15 the base shear is distributed among all floors. The vector of lateral forces and the interstorey shears are respectively shown in the last two columns of Table 3.1. To determine the design forces this force vector is applied to the structural model and a conventional elastic static analysis is carried out. The construction of the model took into account the recommendations made by the authors of this method, such as using cracked stiffnesses in columns and beams, the stiffness of the elements that present inelastic behavior (beams) were multiplied by the factor ($1/\mu$), the distribution of moments in the columns of the ground floor is modified such that the inflection point occurs at 60% of the height of the storey. In order to meet the design strength was necessary to increase the cross section of the columns to 0.55x0.55 m.

Fig. 3.2 shows the comparison of the lateral displacements and the interstorey drifts proposed for design and those obtained using the nonlinear step by step analysis of the structure designed. Considering that the design drift was 0.025 and the corresponding target roof displacement 0.828 m, the maximum roof displacement obtained from the results of the nonlinear analysis, was 0.661 m and the maximum interstorey drift 0.022.



a) Displacement profile b) Interstorey drifts c) Damage distribution

Figure 3.2. Comparison of the displacement and interstorey drifts (design method *vs.* calculated non linear) and the damage distribution

3.2. Results of the DBSDDC

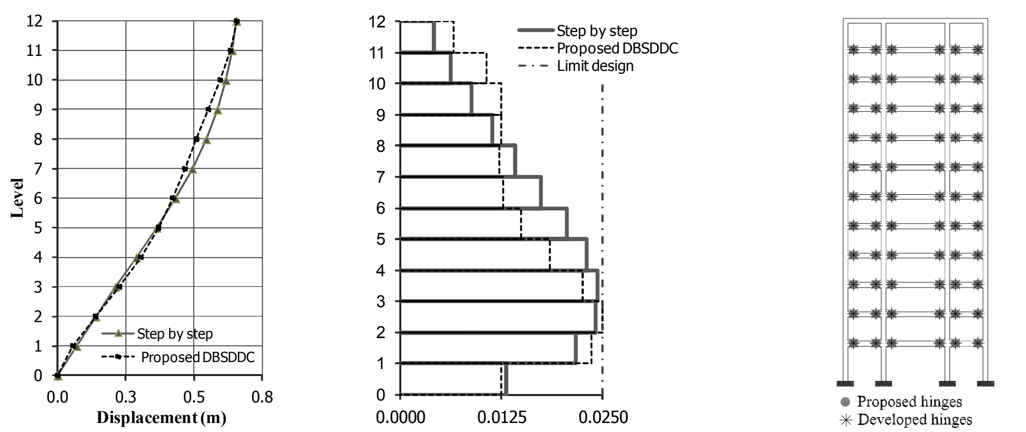
According to the dimensions obtained with the preliminary design, the model of the undamaged structure was built, a modal analysis was carried out and from these results the fundamental period of the structure (T_E=2.40 s), was determined. Subsequently, the model of the damaged structure is constructed by assigning to the model of the undamaged structure the chosen distribution of damage, modal analysis for this new model is carried out to obtain the fundamental period of the damaged structure (T_I=4.80 s). Fig. 3.3c shows the distribution of damage used in this design.

From the results obtained from the damaged model, the target roof displacement (d_u=0.633 m) was determined and, from it and the modal information, the ultimate spectral displacement (S_{du}=0.605 m) is obtained; the yield spectral displacement (S_{dy}=0.334 m) is calculated with Eqns. 2.7 and 2.9.

With the previous information and Eqns. 2.19 and 2.20 the ductility (μ=1.830) and the post-yielding to initial stiffness ratio (α=0.250) are calculated. Using the calculated values of μ and α, a displacement spectrum is constructed and from it the displacement associated to T_E, read. For this example the original geometry of the columns was modified to give a period, for which the spectral displacement equaled that corresponding to the target displacement. The final cross section of the columns was 0.55x0.55 m. From these dimensions, the values of T_E=2.26 s, T_I=4.90 s, d_u= 0.656 m, S_{du}= 0.630 m, S_{dy}=0.334 m, μ=1.888 and α=0.212 were recalculated.

Following the design procedure, the strength spectrum associated to the final values of α and μ is constructed and from it, the yield strength (R_y=2.29 m/s^2), for the period that satisfies the target displacement is read. The ultimate strength (R_u=2.72 m/s^2) is obtained using Eqn. 2.21. Finally, the factors λ_1=0.31 and λ_2=0.98 are determined and the modal spectral analysis are carried out to obtain the design forces.

Fig. 3.3 shows the comparison of the real lateral displacements and the interstorey drifts calculated using nonlinear step by step dynamic analysis of the designed structure and proposed; the target displacement in the roof design was 0.656 m whereas the calculated displacement was 0.647 m reaching a drift of 0.024. Fig. 3.3c shows a comparison between the damage distributions, proposed and obtained from the results of the non linear step by step analysis.



a) Displacement profile b) Interstorey drifts c) Damage distribution

Figure 3.3. Comparison of the displacement and interstorey drifts (design method *vs.* calculated non linear) and comparison of damage distributions

4. CONCLUSIONS

This paper presents a comparison of results of two displacement-based design methods, the DDBD (Priestley *et al.*, 2007) and the DBSDDC, (Ayala *et al.*, 2012). From the analysis of the results obtained the following conclusions regarding the application of both design methods are drawn:

1. Both design methods have a potential of practical application and guarantee structural performance as both show an efficient control of the design objectives. Considering that the maximum lateral displacement and the deformed configurations defined in the design process of both methods were similar to the results obtained from the nonlinear step by step dynamic

analyses considered to give real performances under design conditions. Similarly both designs closely follow the conditions required for drifts by the considered seismic design level as all interstorey drift were slightly larger than the calculated under design conditions In general both methods satisfy the strong column - weak beam design philosophy.

2. The DDBD method considers that the damage produced for design conditions should be consistent with the strong column - weak beam philosophy design, *i. e.,* the calculated damage at the extremes of almost all beams, which may exclude those at roof level, and at the base of the columns at ground level, whereas the DBSDDC method reproduces the distribution of damage explicitly considered during the design process, characteristic that makes this method a more versatile design option.
3. In general, the DDBD method is of easy implementation giving acceptable performance results, against the DBSDDC method requires more effort on its implementation; however it gives performance results closer to those calculated under design conditions.
4. In the DDBD method the enforcement of the strong column-weak beam damage mechanism requires the design of the columns to be modified using capacity design concepts, whereas in the DBSDDC method, the design of all beams and columns, those that under design conditions are accepted to experience damage and those that are required to remain elastic, is obtained from the direct application of the method, making the application of the design process a more straight forward option.

Finally based on the above conclusions it may be generally concluded that in spite that acceptable design results may be obtained with both methods, the DBSDDC has as advantages over the DDBD, to be a more precise and versatile design method however to be able to guarantee this conclusion it is necessary carry out the designs of more examples considering structures of different configurations subjected to different seismic demands.

AKCNOWLEDGEMENT

The project "Development and Experimental Evaluation of a Performance Based Seismic Design Method" and the grants of the student author to carry out graduate were sponsored by the National Council for Science and Technology of Mexico, both authors acknowledge this support.

REFERENCES

Ayala, A.G. (2001). Evaluation of seismic performance of structures, a new approach (in Spanish). *Revista Internacional de Métodos Numéricos para Cálculo y Diseño en Ingeniería*. **17**:**3**. 285-303.

Ayala, A.G. Castellanos, H. and López, S. (2012). A displacement-based seismic design method with damage control for RC buildings. *Earthquakes and Structure*. (accepted for publication).

Chopra, A.K. and Goel, R.K. (2001). Direct Displacement-Based Design: Use of Inelastic vs. Elastic Design Spectra. *Earthquake Spectra*. **17:1**. 47-65.

CSI (2006). SAP2000 Integrated Finite Element Analysis and Design of Structures, *Computers and Structures Inc*.

Kappos, A.J. and Stefanidou, S. (2010). A deformation-based seismic design method for 3D R/C irregular buildings using inelastic dynamic analysis, *Bulletin of Earthquake Engineering*, **8:4**. 875-895.

López, S. (2010). Displacement Based Design of Reinforced Concrete Structures, (in Spanish). Master of Engineering Thesis. Graduate Program in Engineering. UNAM, Mexico.

Prakash, V. Powell, G. and Campell, S. (1993). DRAIN-2DX. Base Program Description and User Guide, version 1.10, *Report No. UCB/SEMM-93/17.* Department of Civil Engineering. University of California, USA.

Priestley, M.J.N. Calvi, G.M. and Kowalsky, M.J. (2007). Displacement-Based Seismic Design of Structures, IUSS Press. Italy.

Panagiatakos, T.B. and Fardis, M.N. (1999). Deformation-controlled earthquake-resistant design of RC buildings. *Journal of Earthquake Engineering*, **3:4**. 498-518.

Shibata, A. and Sozen, M.A. (1976). Substitute structure method for seismic design in reinforced concrete, *Journal of the Structural Division, ASCE*, **102:1**. 1-18.

Sullivan, T. (2011). An Energy-Factor Method for the Displacement-Based Seismic Design of RC Wall Structures. *Journal of Earthquake Engineering*, **15:7**. 1083-1116.