Assessment of Direct Displacement –Based Seismic Design of Reinforced Concrete Frames

B. Massena
ICIST, IST, Lisbon, Portugal

R. Bento
ICIST, IST, Lisbon, Portugal

H. Degée
University of Liège, Belgium

15 WCEE
LISBOA 2012

SUMMARY:
The purpose of this paper is the assessment of reinforced concrete building frame structures designed according to the Direct Displacement-Based Design (DDBD), recently proposed as Model Code. The latest improvements recommended by the DDBD methodology regarding reinforced concrete structures are specifically investigated. A set of reinforced concrete structures is designed according to DDBD procedure and their assessment is conducted with pushover and non-linear time-history dynamic analyses, performed with Seismostruct. A comparison of frames characterized by a same overall geometry (number of storeys, bay length and storey height) and designed respectively according to DDBD and to the traditional force-based design method (FBD), as proposed in Eurocode 8 (EC8), is carried out and the differences are outlined. The most significant conclusions are drawn for structures irregular in elevation and likely to exhibit a soft-story mechanism.

Keywords: Direct Displacement-Based Design, Reinforced Concrete Structures, irregularity in elevation

1. INTRODUCTION

Nowadays methodologies based on forces rather than displacements, are still the most widespread in various design codes and most used in design offices to estimate the response of structures subjected to seismic action. During the 1990's as a result of the growing interest for methods based on displacements, in particular for what regards RC structures, as they are felt more appropriate and able to overcome inherent deficiencies of traditional force-based methodologies, several displacement-based seismic design methodologies emerged. One of the new seismic design methodologies was the Direct Displacement-Based Design (DDBD) developed on the base of Priestley's works (Priestley *et al*, 2007) and recently proposed as a "Draft Model Code" (Calvi & Sullivan, 2009). In the DDBD procedure, it is necessary to define the required strength at designed plastic hinge locations in order to obtain the targeted structural performance level under design earthquake. Capacity design rules are then applied to guarantee that plastic hinges do not occur in other regions than the desirable locations, avoiding the development of non-ductile modes of inelastic deformation. The objective of this work is to assess a set of reinforced concrete structures characterized by vertical irregularities and designed according to DDBD. The study is carried out considering two sets of reinforced concrete plane frames. The same number of spans and storeys, and a varying level of vertical irregularity characterize each set.

2. DDBD METHOD FOR REINFORCED CONCRETE FRAMES

2.1 Summary of the design procedure

The step-by-step DDBD procedure can be summarized as follows:

Step 1: Definition of the target displacement shape (Eqn. 2.2) and amplitude of the MDOF structure on the base of performance level considerations (material strain or drift limits) and then derive from there the design displacement Δ_d (Eqn. 2.3) of the substitute SDOF structure (see Fig. 2.1).

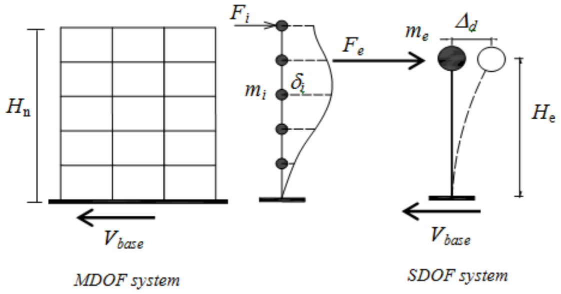


Figure 2.1. Simplified model of a multi-storey building

The design storey displacements Δ_i are found using the shape vector δ_i, defined from Eqn. 2.1, scaled with respect to the critical storey displacement Δ_c and to the corresponding mode shape at the critical storey level δ_c.

$$for\ n \leq 4: \delta_i = \frac{H_i}{H_n};\quad for\ n > 4: \delta_i = \frac{4}{3} \cdot \left(\frac{H_i}{H_n}\right) \cdot \left(1 - \frac{H_i}{4H_n}\right) \tag{2.1}$$

The design storey displacements of the individual masses are obtained from:

$$\Delta_i = \omega_\theta \cdot \delta_i \cdot \left(\frac{\Delta_c}{\delta_c}\right) \tag{2.2}$$

The equivalent design displacement can be evaluated as:

$$\Delta_d = \sum_{i=1}^{n} \left(m_i \Delta_i^2\right) / \sum_{i=1}^{n} \left(m_i \Delta_i\right) \tag{2.3}$$

where m_i is the mass of each storey *i*.
The mass of the substitute structure m_e and the effective height H_e are given by the following equations:

$$m_e = \sum_{i=1}^{n} m_i \left(\frac{\Delta_i}{\Delta_d}\right) = \frac{\sum_{i=1}^{n} m_i \Delta_i}{\Delta_d};\qquad H_e = \sum_{i=1}^{n} \left(m_i \Delta_i H_i\right) / \sum_{i=1}^{n} \left(m_i \Delta_i\right) \tag{2.4}$$

Step 2: Estimation of the level of equivalent viscous damping ξ_{eq}. The equivalent viscous damping can be obtained by one of the equations proposed in the technical literature (Priestley *et al*, 2007). To obtain the equivalent viscous damping, the displacement ductility μ must be known. The displacement ductility is the ratio between the design displacement and the yield displacement Δ_y. The yield displacement is estimated according to the considered properties of the structural elements, for example through the use of approximated equations proposed by Priestley (Priestley *et al*, 2007) based on the yield curvature.

Step 3: Determination of the effective period T_e of the SDOF structure at peak displacement response by using the design displacement defined in step 1 and the design displacement response spectrum corresponding to the damping level estimated in step 2, ξ_{eq}, i.e. entering the design displacement of the substitute SDOF structure Δ_d and determining the effective period T_e (Fig. 2.2).

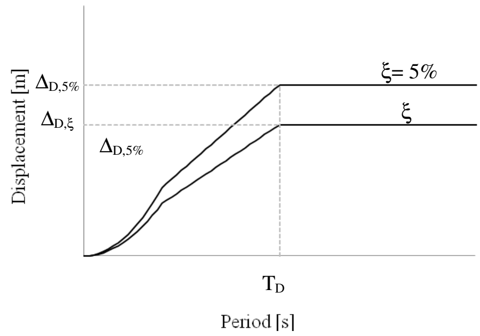


Figure 2.2. Design Displacement Spectrum [Adapted from Sullivan & Calvi (2009)]

According to Model Code (Sullivan & Calvi, 2009), for structures that have a design displacement Δ_d greater than the corner displacement, the effective period, T_e, is obtained by:

$$T_e = \frac{\Delta_d}{\Delta_\xi} T_D \tag{2.5}$$

where, T_D is the spectral displacement corner period (see Fig. 2.2) and $\Delta_{D\xi}$ is the spectral displacement demand at this period for the anticipated level of equivalent viscous damping.

Step 4: The effective stiffness K_e of the substitute SDOF structure, derived from its effective mass m_e and effective period T_e, and the maximum value $K_{e,max}$, are given as:

$$K_e = \frac{4\pi^2 m_e}{T_e^2} \quad ; \qquad K_{e,\max} = \frac{4\pi^2 m_e}{T_e^2} \cdot \frac{\Delta_{D,el}}{\Delta_d} \tag{2.6}$$

where $\Delta_{D,el}$ is the corner spectral displacement demand for the elastic damping level (represented as $\Delta_{D,5\%}$ in Fig. 2.2).

The design base shear V_{base} is the product of the effective stiffness by the design displacement.

$$V_{base} = K_e \Delta_d \tag{2.7}$$

Step 5: Distribution of the design base shear vertically and horizontally to the structural elements of the lateral load resisting system (frames and/or walls).

Step 6: Assessment of moment capacities at potential hinge locations. To this purpose, two different methods of analysis can be used according Priestley (Priestley *et al*, 2007), one is based on relative stiffness members while the other is a simplified method based on equilibrium considerations (statically admissible distribution of internal forces). Herein only the latter is described.

For reinforced concrete structures P-Δ effects should be considered if the stability index θ_Δ is greater than 0.10, with a maximum value of 0.33. The stability index compares the magnitude of the P-Δ effect at expected maximum displacement (Δ_{max}) to the design base moment capacity of the structure (M_D). P is the total gravity load expected at the time of earthquake. The structural stability index is given by:

$$\theta_\Delta = \frac{P\Delta_{\max}}{M_D} \tag{2.8}$$

The base shear accounting the *P-Δ* effects is given by the following equation:

$$V_{base} = K_e \Delta_d + C\frac{P\Delta_d}{H_e} \leq 2.5 R_\xi PGA.m_e + C\frac{P\Delta_d}{H_e} \tag{2.9}$$

where, C is a constant to account for *P-Δ* effects on the displacement response (0.5 for concrete structures), R_ξ is the spectral reduction factor and PGA is the peak ground acceleration at the site for the design intensity level considered.

2.2 Analysis based on Equilibrium considerations

Global base shear force obtained from Eqn. 2.9 must be then distributed in the structures. In the following only the simplified method based on equilibrium considerations will be described. The base shear force is distributed to the floor levels in proportion to the product of mass and displacement, as:

$$for\ n < 10 \qquad F_i = V_{base}\frac{(m_i\Delta_i)}{\sum_{i=1}^{n}(m_i\Delta_i)} \quad ; \quad for\ n \geq 10 \qquad F_i = F_t + 0.9V_{base}\frac{(m_i\Delta_i)}{\sum_{i=1}^{n}(m_i\Delta_i)} \tag{2.10}$$

where, $F_t = 0.1V_{base}$ at roof level, and $F_t = 0$ at all other storey levels.

The frame building is then analyzed by using statically admissible distribution of internal forces, in order to assess the moment capacities at potential hinge locations.

2.2.1 Beam Moments

The lateral seismic forces F_i (Eqn. 2.10) produce axial forces (compression or tension) and column-base moments (M_c) in each of the columns. The seismic beam shears ($\sum V_{Bi}$) are derived from seismic axial forces induced in each of the columns. Considering the equilibrium at base level, the total overturning moment is given by:

$$OTM = \sum_{i=1}^{n} F_i H_i \tag{2.11}$$

where H_i is the height of floor i.
Knowing that equilibrium should be maintained between internal and external forces, the total overturning moment at the base of the structure is thus:

$$OTM = \sum_{j=1}^{m} M_{cj} + \sum_{i=1}^{n} V_{Bi} \times L_{Bi} \tag{2.12}$$

where M_{cj} are the column-base moments (m columns) and L_{Bi} is the lengths of span of beam i.

Combining Eqn. 2.11 with Eqn. 2.12, the sum of seismic axial forces is defined by:

$$\sum_{i=1}^{n} V_{Bi} = \frac{M_{Bi}}{\sum_{i=1}^{n} M_{Bi}}\left(\sum_{i=1}^{n} F_i H_i - \sum_{j=1}^{m} M_{cj}\right) / L_{Bi} \tag{2.13}$$

Every distribution of the total required beam shear that verifies Eqn. 2.13 will result in a statically admissible equilibrium solution and the final choice has thus to be done on the base of engineering judgment. One suggestion is proposed (Pettinga & Priestley, 2005) for the distribution of the total beam shear V_{Bi} and the storey shear forces at level i $V_{S,i}$ are given by as:

$$V_{Bi}=\sum_{i=1}^{n}V_{Bi}\cdot\frac{V_{S,i}}{\sum_{i=1}^{n}V_{S,i}} \quad \text{and} \quad V_{S,i}=\sum_{k=i}^{n}F_k \tag{2.14}$$

From equilibrium considerations, it is then possible to derive the beam design moments at column centrelines for each beam span by:

$$M_{Bi,l}+M_{Bi,r}=V_{Bi}L_{Bi} \tag{2.15}$$

where, $M_{Bi,l}$ and $M_{Bi,r}$ are the beam moments at the column centrelines at the left and right end of the beam, respectively.

2.2.2 Column Moments

Knowing the beam moments, the columns moments can be obtained directly by equilibrium considerations: the total storey shear force (Eqn. 2.14) is shared between the columns. From the shear forces at the base of each column V_C, it is then possible to obtain the moment at the base and top of the columns, $M_{C1,b}$ and $M_{C1,t}$ respectively. Keeping in mind that structural analysis based on equilibrium considerations is actually an approximation of the real distribution, the designer gets some freedom in choosing the moment capacities at the column-base of first floor, provided the equilibrium is maintained between internal and external forces. In technical literature some suggestions are made to estimate the moment capacities of the column-base hinges (Priestley *et al*, 2007). To obtain the moments in the whole columns, the procedure must then be continued with consideration of equilibrium at the node of level 2 and successively until the top level is reached.

3. CASE-STUDY

3.1 Description and design assumptions

The DDBD, as well as FBD as it is proposed in EC8, were applied to design a set of reinforced concrete plane frames case-studies. Two groups of plane frames with three spans with 5m length each and three and four number of storeys are considered. Each group comprises a vertically regular structure (h_i=3m) and two are characterized by vertical irregularities likely to induce a ground-storey mechanism (first floor with 4m and 5m height respectively - see Fig. 3.1). It is reminded that one of the objectives of the present contribution is to assess in which way DDBD methodology can cope with such irregularities.

Assumed mechanical properties of materials are: f_{ck} equal to 25 MPa (C25/30) and f_{yk} equal to 500 MPa (B500). In addition to the self-weight of the beams and the slab, a distributed dead load of 1.5 kN/m^2 due to floor finishing and partitions is considered, as well as an imposed live load with nominal value of 2 kN/m^2. The slab thickness is equal to 0.15 m and its contribution to the structural response was taken in account by considering an effective beam width according to Eurocode 8 (EC8, 1998). Adopted dimensions of beams are a width equal to 25 cm and a depth equal to 50 cm. The column cross sections were defined accordingly, in order to limit the normalized axial force (EC8, 1998). In order to simplify the procedure, equal dimensions were considered for external and internal columns, without variation in height. The seismic action is defined according to Eurocode 8 and Portuguese National Annex with the elastic acceleration response spectrum S_a for subsoil class A (rock). The value of the peak ground acceleration a_g used in the definition of the response spectrum is 0.25g. The design elastic 5% damped displacement spectrum S_{De} used for DDBD is characterized by a corner period of 2.0 sec. For the DDBD procedure, an overall drift limit (θ_c) equal to 2.5% is considered, in accordance with DDBD Model Code suggestion. The seismic performance of the structures was evaluated by means of both pushover and non-linear dynamic analyses, performed with Seismostruct (Seismostruct, 2010) and results of both analyses were compared with seismic behavior expected from design. Pushover analyses were developed according to the N2 method proposed in Eurocode 8 (EC8, 1998). Non-linear dynamic analyses were performed using a group of seven accelerograms, generated with the GOSCA software (Denoël, 2001). Reinforcement schemes have been selected and the criteria for ductile behaviour of concrete sections defined in Eurocode 8 fulfilled (Ductility Class Medium-DCM). Adopted longitudinal reinforcement ratios obtained with DDBD procedure is shown in Fig. 3.1 for all elements (represented in red).

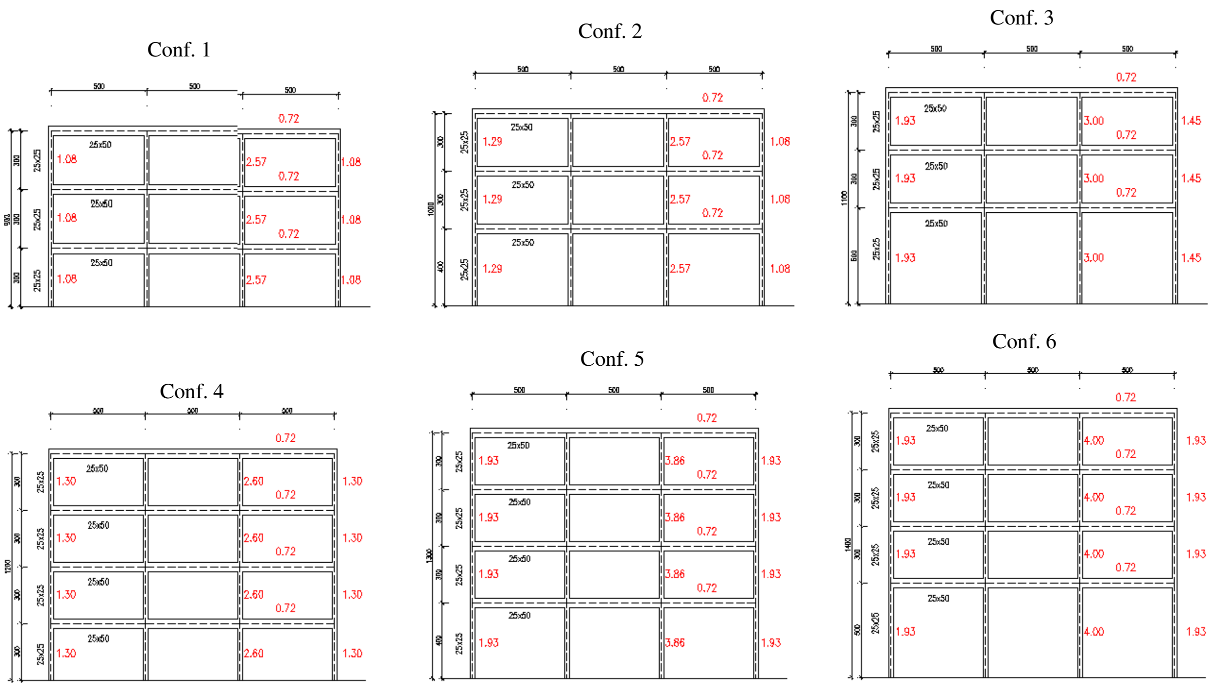


Figure 3.1. Structures under study (dimension in cm, reinforcement ratios in %)

4. RESULTS

Table 4.1 shows the main design parameters of the designed plane frames related with the DDBD procedure for the all configurations up to the definition of the base shear. The required flexural strength of members was obtained by means of equilibrium considerations according to the DDBD procedure.

Table 4.1. Design parameters for structures under study

Conf.	θ_c [%]	H_n [m]	h_1 [m]	h_i [m]	Δ_{dtop} [m]	H_e [m]	m_e[ton]	Δ_y [m]	Δ_d [m]	μ	ξ_{eq} [%]	T_e [s]	V_{base} [kN]
1		9	3		0.225	7.04	80.12	0.097	0.149	1.539	11.30	2.11	97.95
2		10	4		0.250	7.90	90.63	0.109	0.154	1.413	10.26	2.17	98.01
3	2.5	11	5	3	0.275	8.79	99.87	0.129	0.159	1.316	9.32	2.25	99.60
4		12	3		0.300	9.05	87.56	0.124	0.161	1.291	9.06	2.27	116.60
5		13	4		0.325	9.87	136.10	0.136	0.165	1.214	8.17	2.33	117.30
6		14	5		0.350	10.74	147.35	0.148	0.170	1.152	7.38	2.40	116.40

The results of non-linear time history analysis and Pushover analysis carried out on the six case-studies designed according to DDBD and FBD procedure are presented hereunder. DDBD and FBD are compared in terms of longitudinal reinforcement for all elements. With FBD, much higher values of internal forces, and consequently longitudinal reinforcement ratios were reached for all the elements, especially for the columns. Moreover, according to the FBD the longitudinal reinforcement ratios for all the columns were limited by the maximum values defined in EC8. Nevertheless, the differences between the two design procedures reached for external columns almost 200%. For the nonlinear dynamic analyses the mean values and the mean values minus and plus the standard deviation are depicted for both type of design. The displacement profile inter-storey drift ratio for the three and four storey's building frames configurations are presented in Fig. 4.1 and Fig. 4.2 and compared with the displacement profile from the two design procedures. Fig. 4.3 compares the interstory drift ratios obtained from nonlinear time-history analyses for the frame building of a same subset, in order to emphasize in the effect of vertical irregularities.

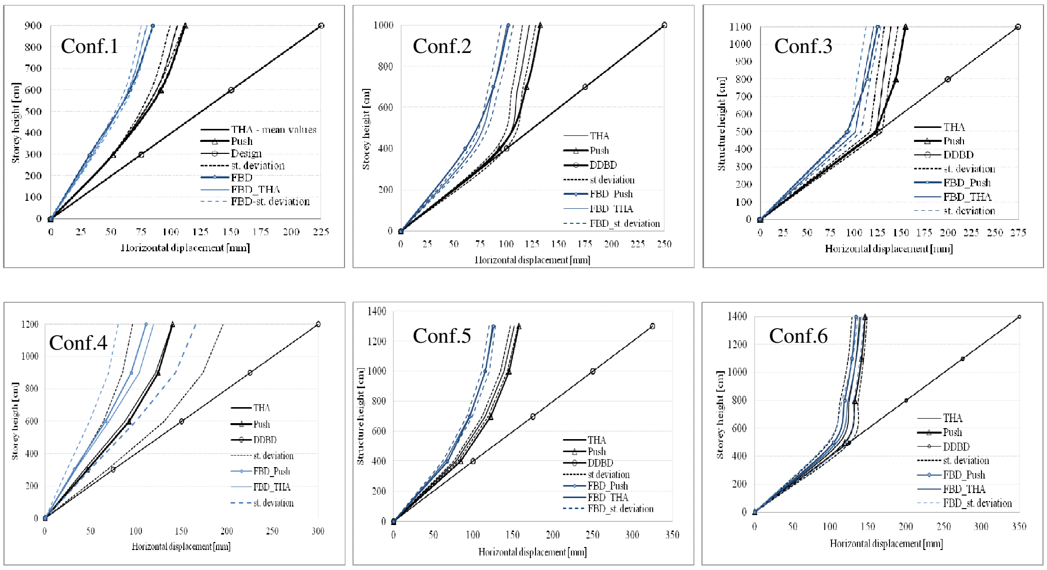


Figure 4.1. Displacement profile

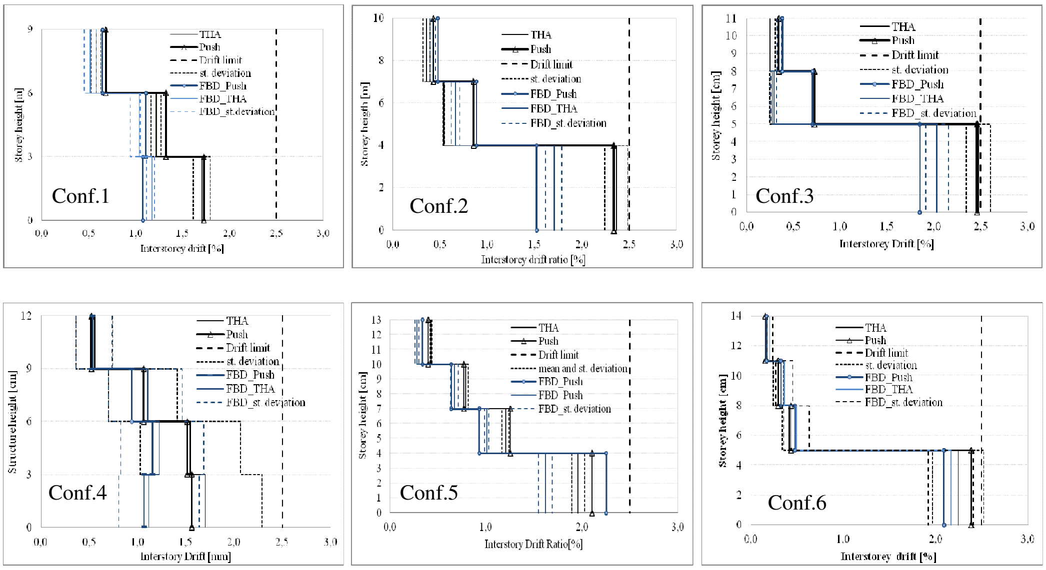


Figure 4.2. Inter-storey Drift ratio

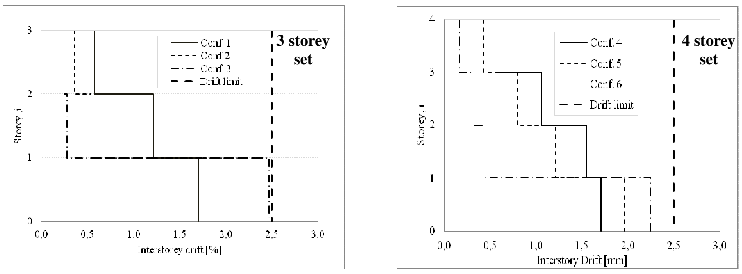


Figure 4.3. Inter-storey drift ratio for the structures design according DDBD (non linear time-history analyses)

From Fig. 4.1 and Fig. 4.2, it can be observed that the design drift limit imposed by the DDBD in terms of displacement profile and inter-storey drift ratio is never reached, whatever configuration and design method is considered. It can be stated that structures designed according to FBD procedure presents, as expected, smaller displacements and inter-storey drift ratio when compared with those obtained by means of the DDBD procedure; i.e. the results are significantly more conservative. To fulfill the requirements of EC8, and especially the capacity design principles, the design according to FBD implies larger sections of the columns. The obtained inter-storey drifts from analyses at the first storeys resulted in values smaller than the design ones, particularly for the vertically regular frames without a soft storey (configuration 1 and 4). The development of a soft storey can be observed in the frame deformation (see Fig. 4.6. - Conf. 3 and 6).

For the sake of comparison, alternative design of the same global configurations was carried out, where, beyond pure DDBD Model Code, the column cross sections are also checked in order to limit the normalized axial force and where the anchorage of beam reinforcements complies with Eurocode 8 rules. It implies that, in order to prevent bond failure, the maximum diameter of longitudinal beam bars crossing a beam-column connection shall be limited by an upper value of the diameter of the longitudinal bars of the beam, d_{bL}, that pass through interior beam-column joints or are anchored at exterior ones as:

$$\frac{d_{bL}}{h_c} \leq \frac{7.5 f_{ctm}}{\gamma_{Rd} f_{yd}} \cdot \frac{1+0.8\nu_d}{1+0.75 k_D \rho' / \rho_{\max}} \quad \text{or} \quad \frac{d_{bL}}{h_c} \leq \frac{7.5 f_{ctm}}{\gamma_{Rd} f_{yd}} \left(1+0.8\nu_d\right) \tag{4.1}$$

where, h_c is the width of the column parallel to the bars; f_{ctm} is the mean value of the tensile strength of concrete; f_{yd} is the design value of the yield strength of steel; ν_d is the normalised design axial force in the column, taken with its minimum value for the seismic design situation; k_D is the factor reflecting the ductility class equal to 1 for DCH and to 2/3 for DCM; ρ' is the compression steel ratio of the beam bars passing through the joint;ρmax is the maximum allowed tension steel ratio; γ_{Rd} is the model uncertainty factor on the design value of resistances, taken as being equal to 1.2 or 1.0 respectively for DCH or DCM (due to overstrenght owing to strain-hardening of the longitudinal steel in the beam).

The resulting cross sections of columns are larger than the previous ones. The overall geometrical characteristics and material properties described previous in section 3 were maintained. Fig. 4.4 presents the results of the assessment by nonlinear analysis, in terms of displacement profile, for the alternative configurations designed by means DDBD.

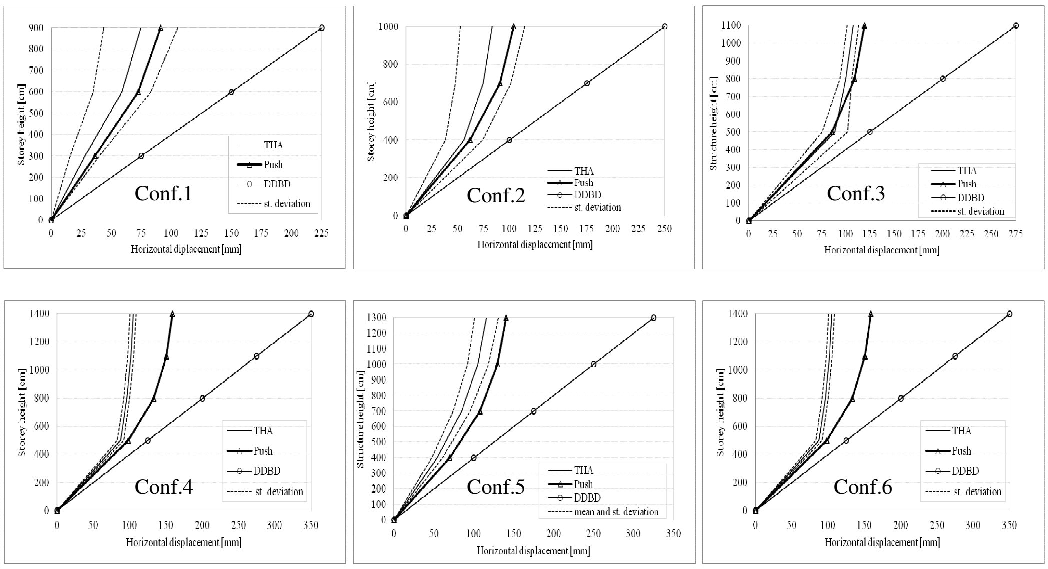


Figure 4.4. Displacement profile

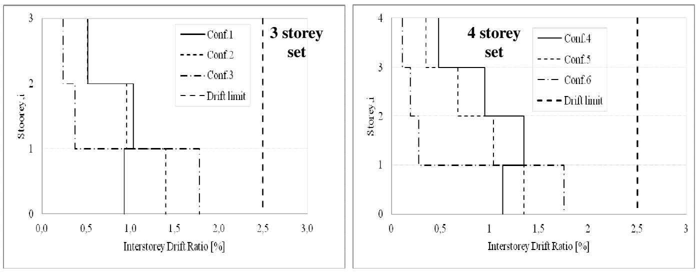


Figure 4.5. Inter-storey drift ratio (non linear time-history analyses)

By comparing Figs. 4.1 and 4.4, it can be seen that the displacement profile values resulting from NLTHA and pushover analyses are smaller if considering the additional requirements on columns and anchoring, when compared with the pure DDBD procedure. Fig. 4.5 shows that the values of interstory drift limit are also much away from the drift limit imposed in the beginning of the design, essentially due to highly increased stiffness because of the two additional requirements.

Shear Strength verification

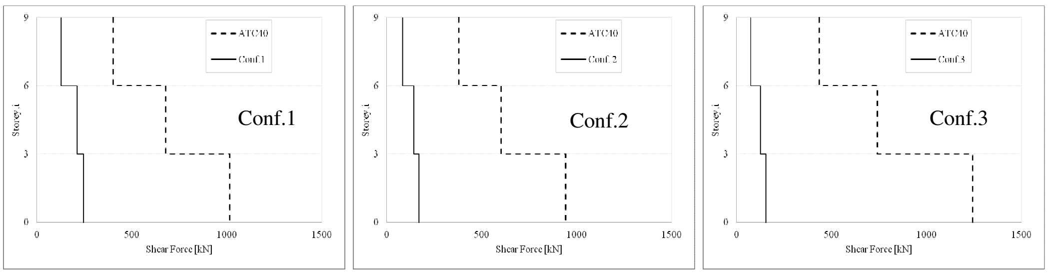
Up to this point of the developments, it has been implicitly considered that the shear resistance of the frame structural components is not a critical issue. Fig. 4.6 compares the total shear obtained from NLTHA for each storey with the column shear capacity estimated according to ATC40 recommendations (ATC40, 2005). According to ATC40 recommendations, the column shear capacity may be calculated by the following equations:

$$V_n = V_C + V_S \tag{4.3}$$

where,

$$V_C = 35\lambda\left(k + \frac{N}{2000A_g}\right)\sqrt{f_c^{'}}\,b_{wd} \quad \text{and} \quad V_S = \frac{A_V f_{yd}}{0.6s} \tag{4.4}$$

k is equal to 1 in regions of low ductility and 0 for regions of moderate and high ductility; λ= 1 for normal-weight aggregate concrete; N is the axial compression force (N=0 for tension force); V_n is the total shear strength; V_c is the shear strength due to the concrete; V_S is the shear strength due to the transverse reinforcement; $f_c^{'}$ is the design strength of the concrete; b_w is the section width; d is the section useful height; A_g is the gross section; f_y is the design strength of the transverse reinforcement steel; A_g is the transverse reinforcement area and s is the spacing of the transverse reinforcement. All units are in pounds and inches.



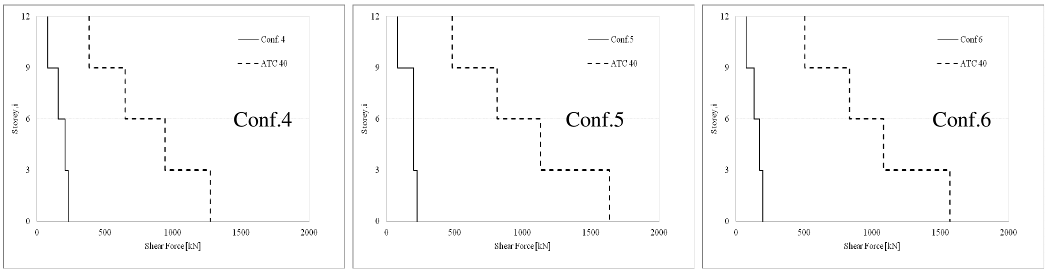


Figure 4.6. Shear capacity NLTHA and according to ATC 40 recommendations

Fig. 4.6 shows that in the structures under study the shear demand is always smaller than the shear strength, confirming the general assumption considered in the previous developments.

5. CONCLUSIONS

Two sets of reinforced concrete plane frames characterized by irregularity in elevations were designed according to FBD and DDBD proposed as a "Draft Model Code" recently. The obtained interstory drifts results smaller than the design drift limit imposed by the design procedure in terms of displacement profile for all configurations and for both design procedures. Therefore and based on the results obtained for the set of frames analyzed it seems that DDBD methodology can cope with the vertical irregularities studied and the results are significantly less conservative than the ones obtained by FBD according to EC8 rules. It is seen that requirement for anchorage of reinforcement for beams lead to an over dimensioned structures, influencing the final performance of the designed structures that comply this rule. It was observed that designed structures according to FBD imply larger sections to fulfill both, the capacity design rules and the limit values of reinforcement ratio.

AKCNOWLEDGEMENT

The authors would like to acknowledge the financial support of the Portuguese Foundation for Science and Technology (Ministry of Science and Technology of the Republic of Portugal) through the research project PTDC/ECM/100299/2008 and through the PhD scholarship SFRH/BD/36648/2007 granted to B. Massena. H. Degée thanks for the support received from F.R.S.-FNRS (Belgian Fund for Research).

REFERENCES

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

Sullivan, T. J., Calvi, G.M. (2009) A Model Code for the Displacement-Based Seismic Design of Structures. *IUSS Press,* Pavia, Italy.

Pettinga, J.D. and Priestley, M.J.N. (2005) Dynamic behavior of reinforced concrete frames designed with direct displacement-based design. *Journal of Earthquake Engineering*. **Vol 9** **(2)**, 309-330.

EN 1998-1:2004. Eurocode 8: Design of Structures for Earthquake Resistance - Part 1: General Rules. Seismic Actions and Rules for Buildings. CEN. Brussels. Belgium.

NP EN 1998-1:2007. Portuguese National Annex.

SeismoSoft (2010). SeismoStruct - A Computer Program for Static and Dynamic Nonlinear Analysis of Frame Structures. Available from URL: www.seismosoft.com.

Denoel. V. (2001) Generation of Spectrum Compatible Accelerograms. Research Report. Université de Liège.

ATC, Applied Technology Council.(2005) Improvement of Nonlinear Static Seismic Analysis Procedures, *FEMA440 Report*. Redwood City, CA.