CONSEQUENCES OF DESIGN CHOICES IN DIRECT DISPLACEMENT BASED DESIGN OF R.C. FRAMES

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ABSTRACT:
Direct Displacement-Based Design is a procedure developed for some years on the base of Priestley's works and aims at designing structures in order to achieve displacements corresponding to a specified limit state under earthquake action. The procedure is nowadays rather well documented. However its application to practical cases still let the door open to a number of choices to be made by the designer. The consequences of the different choices and assumptions made along the use of DDBD methodology for designing reinforced concrete structures are evaluated. The main design options considered are dealing with the vertical distribution of column moments, storey shear distribution and horizontal distribution of bending moments in beams, as well the consequences of the design for gravity loads. Conclusions are dealing with an identification of the choices and parameters having the most significant impact on the final design, as well as with practical recommendations for the designer.

Keywords: Displacement-based seismic design, Reinforced Concrete Structures

1. INTRODUCTION

As the importance of displacements, rather than strength, become to be better appreciated, a growing interest appeared during the last years for methods based on displacements, in particular for what regards RC structures. Over the last years various contributions were made towards the development of displacement-based seismic design methodologies, but it was only in the 1990's that formal proposals were made to implement the emerging ideas into formalized design procedures. One of these new design procedures is the Direct Displacement-Based Design (DDBD) initiated by Priestley (Priestley, 1993) and has been developed (Priestley and Calvi (1997), Priestley and Kowalsky (2000) and Priestley, Kowalsky and Calvi (2007)). In this approach, structures are designed in order to achieve displacements corresponding to a specified limit state, rather than to be limited by such displacements. This would essentially result in uniform-risk structures, which is philosophically compatible with the uniform-risk seismic intensity incorporated in most codes. In DDBD, the design process for a multi-degree of freedom structure (MDOF) starts from the determination of the characteristic of an equivalent single-degree-of-freedom (SDOF) structure. This equivalent SDOF is based on the "substitute structure" analysis procedure developed by Shibata and Sozen (1976) and is characterized by the secant stiffness K_e at maximum displacement Δ_d and a level of equivalent viscous damping ξ representing the combined effect of viscous and hysteretic dissipation. Knowing the SDOF characteristics, it is then possible to determine the design base shear V_{base}. The design base shear obtained from the SDOF system is then distributed as equivalent inertia lateral forces in the original structure. The design moment at potential plastic hinges are then determined as well as the design moments and shears for all the others critical structural sections.

2. DDBD METHOD FOR REINFORCED CONCRETE FRAMES

2.1 Overall summary of the procedure

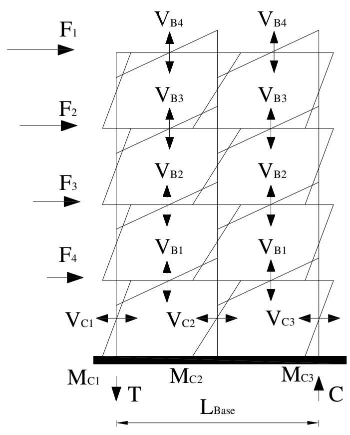
DDBD methodology can be summarized as follows:

Step 1: Define a target displacement shape and amplitude of the MDOF structure on the base of performance level considerations and then derive from there the design displacement Δ_d of the substitute SDOF structure.
Step 2: Calculate the level of equivalent viscous damping ξ. To this purpose, the yield displacement Δ_y is first estimated according to the considered properties of the structural elements, for example through the use of approximated equations proposed by Priestley (2003) based on the yield curvature. Knowing the yield displacement and the target design displacement, the expected ductility level μ can be estimated. The corresponding energy dissipation can then be converted into equivalent viscous damping by one of the equations proposed in the technical literature, such as for example by Priestley (2007) for reinforced concrete frames.
Step 3: Determine the effective period T_e of the structure by using the target displacement defined in step 1 and the design displacement response spectrum corresponding to the damping level estimated in step 2.
Step 4: Derive the effective stiffness of the substitute SDOF structure from its effective mass and effective period. The design base shear is then simply obtained by multiplying the effective stiffness by the design displacement.
Step 5: Distribute the design base shear vertically and horizontally to the structural elements of the lateral load resisting system (frames and/or walls).
Step 6: Evaluate moment capacities at potential hinge locations. For frames, two distinct methods of analysis can be used (Priestley 2007): 1) based on relative stiffness members or 2) based on equilibrium considerations (statically admissible distribution of internal forces). In the following only the latter is considered.

2.2 Analysis based on Equilibrium considerations

2.1.1 Beam Moments

Fig.2.1 shows the seismic lateral forces and the corresponding internal forces induced in a regular frame building by the seismic action.



$$T = C = \sum_{i=1}^{n} V_{Bi}$$

T is the total axial force in the outside column in tension

C is the total axial force in the outside column in compression

V_{Bi} is the beam shear in beam i

Figure 2.1 Seismic Moments from DDBD adapted (Priestley 2007).

The total overturning moment *OTM* at the base of the structure is given by:

$$OTM = \sum_{i=1}^{n} F_i H_i \tag{2.1}$$

where n is the number of stories, F_i is the seismic lateral force applied at each floor and H_i is the height of each level. The *OTM* induced by external forces must be equilibrated by the internal forces. Therefore:

$$OTM = \sum_{j=1}^{m} M_{Cj} + T.L_{base} \tag{2.2}$$

where M_{Cj} are the column base moments, T and C are the seismic axial forces (tension or compression) in the exterior columns, and L_{base} is the distance between external columns. The forces T and C are the sum of the beam shear forces all over the building:

$$T = \sum_{k=1}^{n} V_{Bi} \tag{2.3}$$

Any distribution of the total beam shear force that assures Eq. (2.1) will result in a statically admissible equilibrium solution and can be done by engineering judgment. Priestley (2007) and Pettinga (2005) suggest however that the distribution of the total beam shear force should be done in proportion to the storey shears in the level below the beam under consideration. The distribution of the total beam shear force is thus:

$$V_{Bi} = T.V_{S,i} \Big/ \sum_{i=1}^{n} V_{S,i} \tag{2.4}$$

where the storey shear forces at level i are given by:

$$V_{S,i} = \sum_{k=i}^{n} F_k \tag{2.5}$$

When the shear of each beam has been determined, the lateral beam design moments at the column centerlines are defined:

$$M_{Bi,l} + M_{Bi,r} = V_{Bi}.L_{Bi} \tag{2.6}$$

where L_{Bi} is the beam span between column centerlines, and $M_{Bi,l}$ and $M_{Bi,r}$ are the beam moments at the column centerlines at the left and right end of the beam, respectively.

2.1.2 Column Moments

Knowing the beam moments, the columns moments can be obtained directly by equilibrium considerations: the total storey shear force [Eq. (2.5)] is shared between the columns (it is suggested to share the horizontal shear according to the following ratio: 1 for external columns and 2 for internal 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. According to Priestley (2007) the contra-flexure point for the first floor columns could be considered around 60% of the height of the column H_1, therefore:

$$M_{C1,b} = 0.6 V_{C1} H_1 \tag{2.7}$$

$$M_{C1,t} = 0.4 V_{C1} H_1 \tag{2.8}$$

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.

2.3 Objective of the study

The above summary of the procedure shows that although some design assumptions are suggested in the technical literature, it is however clear that some aspects are still under the responsibility of the design engineer (e.g. position of the contra-flexure point in the columns or sharing of the horizontal storey shear between the columns).

As a consequence, the present paper outlines a preliminary study aiming at investigating the impact of these assumptions on the final design of RC frames and on their seismic performance. To this purpose a simple case-study (described in section 3) is designed according to the following assumptions:

Table 2.1 Design assumptions

Design situation	Assumptions
Case 1	DDBD approach, using all design assumptions suggested by Priestley (2007)
Case 2	DDBD approach considering that horizontal shear is shared between the columns according to their bending stiffness
Case 3	DDBD approach considering that contra-flexure point in the columns is always at mid-height, including for columns of the bottom storey
Case 4	DDBD approach using Blandon-Priestley formula for estimating the equivalent viscous damping (Blandon 2005)
Case 5	DDBD approach using Dwairi-Kowalsky formula for estimating the equivalent viscous damping (Dwairi 2004)
Case 6	Force-based design according to Eurocode 8 (EN1998-1) considering a behavior factor q equal to 3.9.

For DDBD approach, the chosen design drift limit is 2.5 %, i.e. a target value of the top displacement equal to 0.307 m (see section A-A of Fig.3.1).

The results are then expressed in terms of design bending moment at beam and column ends and definition of longitudinal reinforcements. Finally, the six designed structures are studied using non linear time-history analysis to assess their performances.

3. CASE-STUDY

3.1 Description and design assumptions

The DDBD procedure is applied to the interior frame of the four-storey reinforced concrete structure shown in Fig.3.1, with a global geometry (height and spans as well as beams and columns cross-section dimensions) defined in the context of the "Cooperative Research on the Seismic Response of the Reinforced Concrete Structures" (CRSRRCS 1992). The structure is irregular in terms of spans and the lateral resistance is provided by one-way frame action. The exterior columns dimensions are 0.40 m x 0.40 m and 0.45 m x 0.45 m for the interior column. All the beams have a section of 0.30 m x 0.45 m. The slab thickness is equal to 0.15 m. The reinforced concrete frames are made with concrete C25/30 (f_{cd} = 16.7 MPa, E_c = 30.5 GPa). The reinforcement steel is a classical Tempcore steel B500 (f_y =500 MPa, E_s = 200 GPa). In addition to the self-weight of the beams and the slab, a distributed dead load of 2 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. In gravity load combinations, nominal dead and live loads are multiplied by load factors of 1.35 and 1.5 respectively, while in the seismic design combination, dead loads are considered with their nominal value and combined with live loads at 40% of their nominal value. Structures are designed for the envelope of gravity and seismic load combinations

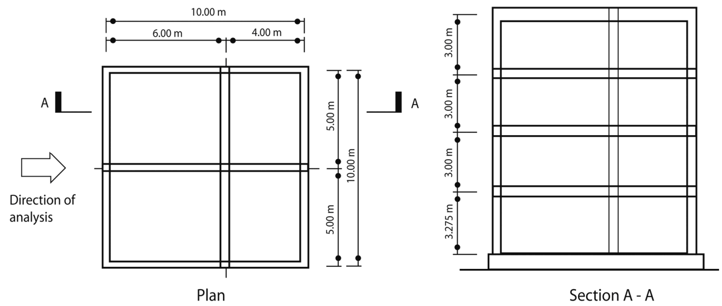


Figure 3.1 General Layout [adapted from (CRSRRCS 1992)]

The frame building is considered being located in Continental Portugal (Algarve) as an ordinary building class of importance II. The seismic action is defined by Eurocode 8 (1998) and Portuguese National Annex with the elastic acceleration response spectrum S_a for subsoil class D as shown in Fig.3.2. The value of the peak ground acceleration a_g used in the definition of the response spectrum is 0.35g. The elastic displacement spectrum S_{De} used for DDBD is the one defined in Eurocode 8 by

$$S_{De}(T) = S_a(T)\left[\frac{T}{2\pi}\right]^2 \tag{3.1}$$

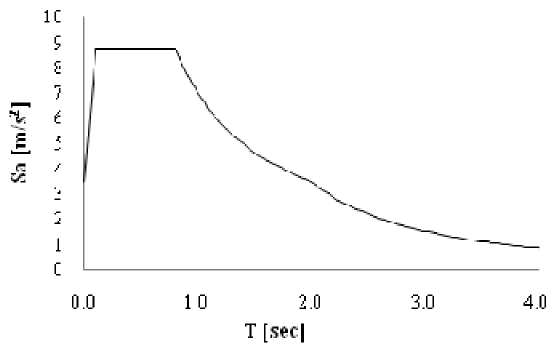


Figure 3.2 Reference acceleration spectrum

4. RESULTS

4.1 Global parameters

From the target drift limit, it is possible to derive the corresponding characteristics of the equivalent SDOF system for which the target displacement is equal to 0.230 m. It is then possible to derive the effective period, effective stiffness and base shear by using the chosen definition of the equivalent viscous damping. Obtained values are given in table 4.1, where it can be seen that the results are rather independent from the chosen damping equation. For sake of comparison, the table gives also the base shear obtained with force-based EC8 approach considering a behaviour factor equal to 3.9, as well as the displacement obtained from force-based analysis, considering that the maximum displacement is equal to q times the displacement obtained from elastic analysis (i.e. $q_d = q$) as suggested by EC8.
It can be observed that design displacement and base shear obtained from EC8 approach are significantly lower than corresponding parameters obtained from DDBD procedure.

Table 4.1 Results of DDBD and EC8 in terms of displacement and base shear

	Target SDOF displacement Δ_d [m]	*Target top displacement [m]*	*T_{eff} [s]*	*K_{eff} [kN/m]*	*V_{base} [kN]*
Case 1	0.230	0.307	1.69	2164	499
Case 2					
Case 3					
Case 4			1.70	2150	495
Case 5			1.75	2038	469
Case 6	-	0.219	0.97	-	300

4.2 Internal forces

Tables 4.2 and 4.3 give the design values of bending moments obtained respectively for beams and columns. Two particular points must be noticed prior to these tables:

- Beam moments are obtained from direct analysis (either using statically admissible distribution for DDBD or spectral analysis for EC8), while column moments are obtained from capacity design considerations. For DDBD, capacity design is performed according to Priestley recommendations, that are different from equivalent rules in EC8;
- Design beam moments for DDBD are obtained as the most challenging situation between gravity loads and seismic loads, while EC8 considers the most challenging situation between gravity combination (similar to DDBD) and seismic combination. The main difference comes from the fact that DDBD considers seismic loads acting alone while EC8 seismic combination include a contribution of gravity loads, although reduced compared to the gravity combination.

Table 4.2 Results of DDBD and EC8 in terms of design beam moments [kNm]

			Case 1	*Case 2*	*Case 3*	*Case 4*	*Case 5*	*Case 6*
Level 1	*Long span*	*Left end*	285.3	285.3	310.3	283.4	268.7	275.6
		Right end	282.8	282.8	307.6	280.9	266.3	293.0
	Short span	*Left end*	258.1	258.1	284.1	256.4	243.0	281.0
		Right end	255.8	255.8	281.6	254.1	240.8	255.2
Level 2	*Long span*	*Left end*	206.3	206.3	205.6	205.0	194.3	270.4
		Right end	207.4	207.4	207.4	207.4	207.4	280.7
	Short span	*Left end*	153.4	153.4	153.4	153.4	153.4	259.3
		Right end	214.1	214.1	214.1	214.1	214.1	246.0
Level 3	*Long span*	*Left end*	271.3	271.3	295.1	269.5	255,47	232.8
		Right end	275.1	275.1	299.3	273.3	259.1	239.0
	Short span	*Left end*	245.4	245.4	270.2	243.8	231.1	195.8
		Right end	248.9	248.9	274.0	247.2	234.4	191.5
Level 4	*Long span*	*Left end*	196.2	196.2	195.5	194.9	184.7	157.0
		Right end	198.9	198.9	198.3	197.6	187.3	195.0
	Short span	*Left end*	128.0	128.0	128.0	128.0	128.0	145.0
		Right end	125.2	125.2	76.6	124.3	117.9	103.7

Table 4.3 Results of DDBD and EC8 in terms of design column moments (given only for bottom storey) [kNm]

		Case 1	*Case 2*	*Case 3*	*Case 4*	*Case 5*	*Case 6*
Column 1	***Top***	303.7	337.3	303.7	301.7	286.0	162.2
	Bottom	306.2	340.1	306.2	304.2	288.4	194.9
Column 2	***Top***	607.5	540.3	607.5	603.4	572.1	289.7
	Bottom	612.5	544.8	612.5	608.4	576.8	308.7
Column 3	***Top***	303.2	337.3	303.7	301.7	286.0	162.2
	Bottom	306.2	340.1	306.2	304.2	288.4	192.5

From the values given in these two tables, in can be easily seen that the final resulting reinforcement in beams will be practically insensitive to the assumptions made in DDBD approach. The only assumption that leads to some differences corresponds to case 3 (modification of the position of contra-flexure point in columns) that yields slightly higher values of design moments in some beams.
It is even not very different for EC8 approach mainly because the lower value of design base shear is compensated by the fact that seismic combination of EC8 includes a contribution of gravity loads that results in comparable design values of beam moments. It can also be seen that EC8 approach would lead to a less homogeneous distribution of reinforcements in the different levels of the structure.
Concerning columns, EC8 approach yields smaller values because of the lower influence of gravity loads. For DDBD approaches, only case 2 (modification of the distribution of horizontal shear between the columns) yields slightly different values of reinforcements.

4.3 Performance assessment

Appropriate reinforcement schemes have been defined in order to match the requirements of tables 4.2 and 4.3 in terms of design moments, as well as criteria for ductile behaviour of concrete sections. In particular, it is worth noting that Case 1, 4 and 5 are leading to the same reinforcement scheme, showing thus that the final design is insensitive to the choice of equivalent viscous damping formula.
The six resulting structures have then been submitted to NLTHA with the aim of assessing their actual behaviour under seismic conditions. To this purpose, a set of 7 artificial ground motion time histories compatible with the EC8 spectrum given in Fig.3.2 has been generated with the software GOSCA (Denoël 2001). Numerical simulations have then been carried out with Seismostruct using fibre beam elements (Seismosoft 2010).
Results are summarized in table 4.4 in terms of extreme displacement of the top level. These results have to be related to the target top displacement considered from the very beginning of the DDBD procedure (i.e. 0.307 m).

Table 4.4 NLTHA assessment of the designed structures – average and maximum values of the top displacement

	Case 1, 4, 5	*Case 2*	*Case 3*	*Case 6*
Average	0.258 m	0.252 m	0.251 m	0.283 m
Maximum	0.313 m	0.335 m	0.307 m	0.384 m

Results of table 4.4 show that the final behaviour of structures design according to DDBD are quite insensitive to design assumptions, with a variation on the extreme displacement less than 10%. It can also be seen that the target displacement of 0.307 m is reached with a reasonable accuracy. On the other hand, force-based Eurocode 8 design yields a higher value of displacement, mainly because the design is based on a lower design base shear and thus on weaker columns compared to what is obtained from DDBD.

5. CONCLUSIONS

This paper has presented some results of a study aiming at assessing the consequences of design assumption that must be fixed by the designer in the context of DDBD methodology. In particular, the distribution of horizontal shear between columns and the distribution of bending moment in columns, defined by the position of the contra-flexure point, have been considered. Some alternative formulas for defining the equivalent viscous damping have also been adopted.

Although all the developments have been carried out on a single case-study, some preliminary conclusions can be drawn. The main points are that, for the studied situation, the final design and structural response obtained from DDBD is practically independent on the design assumptions. On the other hand, a comparison with Eurocode 8 approach using standard values of the behavior factor q shows that the force-based method yields a less stiff structure with a final displacement state that comes out of the analysis without control and that is higher than the reasonable target level of 2.5% for the design drift limit.

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