A DIRECT DISPLACEMENT-BASED SEISMIC DESIGN METHOD USING A MDOF EQUIVALENT SYSTEM

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Abstract

An improved version of the direct displacement-based (DDBD) method for the seismic design of plane moment resisting frames in the framework of performance-based design approach is presented. The method employs a multi-degree-of-freedom (MDOF) equivalent system instead of the single-degree-of-freedom (SDOF) equivalent system used by the conventional DDBD method. Thus, the proposed method can take into account the higher mode and P-Δ effects more rationally and with higher accuracy than the conventional one. This is accomplished with the aid of the concept of deformation dependent equivalent modal damping ratios and the concept of the design modal displacements. These design modal displacements are determined on the basis of target interstorey displacement ratios for every performance level and the first few modes significantly contributing to the structural response. Thus, one can determine from the displacement spectrum with high amounts of viscous damping the required modal periods for known values of the design modal displacements. From those modal periods, the corresponding required modal stiffness and hence the modal base shear forces can be obtained. The final required design base shear can be obtained by a combination rule, like the SRSS rule. A characteristic numerical example involving the seismic design of a moment resisting reinforced concrete (R/C) plane frame is presented in detail for illustrating the proposed approach and demonstrating its merits over the conventional DDBD method.

Keywords: Seismic design; Direct displacement-based design; Performance-based design; Equivalent multi-degree-of-freedom structure; Equivalent modal damping ratios.

1. Introduction

The force-based design (FBD) method, which uses forces as the main design parameters, has been adopted by all current seismic design codes, such as EC8 [1]. This method performs design in two steps: the first step involves a strength checking, while the second one a displacement checking, usually accomplished iteratively. The displacement-based design (DBD) method, which uses displacements as the main design parameters, has been emerged as a viable alternative of the FBD method. Since displacements are more intimately related to damage than forces, the DBD method can more effectively control damage. Moreover, DBD requires only one step during the design process, i.e., a strength checking, because the displacement checking is automatically satisfied.

The DBD method of Priestley and co-workers [2-4], called the direct displacement-based design (DDBD) method, is the most well-known and highly developed seismic design method. Indeed, a whole book (Priestley et al [2]), two model codes (Calvi and Sullivan [3], Sullivan et al [4]) and a large number of articles [5-14] have been published on this method. Comparisons of the DDBD method against other DBD methods [13] and the FBD method [14] have been also published, revealing its advantages and limitations over the other methods.

The most important problem with the DDBD method is the replacement of the original nonlinear multi-degree-of-freedom (MDOF) structure by an equivalent linear single-degree-of-freedom (SDOF) structure in accordance with the substitute structure concept of Shibata and Sozen [15]. This replacement simplifies considerably the method at the expense of losing modeling accuracy as one goes from the MDOF system to the equivalent SDOF one. Thus, higher mode effects and P-Δ effects are lost because of this simplification, which is based on an assumed first mode displacement profile of the structure. These problems have been detected by the developers of the DDBD method and corrected later on in a rather artificial way by adding correction terms in the proposed expressions for the lateral displacement profile and the design base shear and its distribution to take into account P-Δ and higher mode effects [3,4].

In this work, an improved version of the DDBD method is presented, which takes into account in a rational manner all the aforementioned problems. In this approach, the original nonlinear MDOF structure is replaced by an equivalent linear MDOF structure with the same mass and elastic stiffness as the original structure with the aid of the deformation dependent equivalent modal damping ratios concept developed by the present authors in connection with the seismic design of steel and reinforced concrete (R/C) plane moment resisting frames (MRF) [16,17]. The two MDOF systems are equivalent in the sense that the work of dissipation due to hysteretic forces in the nonlinear system is equal to the work of dissipation due to viscous forces in the linear structure. Thus, this work equivalence concept can be thought of as an extension of that of Jacobsen [18] from a SDOF system under harmonic excitation to a MDOF system under seismic excitation.

Thus, during the employment of the DDBD method, one determines from the displacement spectrum with high amounts of viscous damping the required modal periods for known values of the design modal displacements. From those modal periods the corresponding required modal stiffnesses and hence the modal base shears can be obtained. The final required base shear is obtained by a combination rule, like the SRSS rule. The aforementioned design modal displacements are obtained on the basis of target interstorey drift ratios defined for every performance level and the first few modes significantly contributing to the structural response. This paper concludes with an example involving the seismic design of a sixteen storey R/C MRF for illustrating the proposed approach and demonstrating its merits over the conventional DDBD method.

2. The original DDBD method

For reasons of completeness a brief introduction to the original method following the model code [4] is given. Consider a plane moment resting frame of n stories under a horizontal seismic motion that has to be designed by the DDBD method, as shown in Fig. 1 (a). The basic idea is to determine the required seismic design base shear V_d for this frame that will ensure that its displacements will not exceed the target displacements. This is accomplished by constructing an equivalent linear SDOF system to the MDOF frame under consideration as it is shown in Fig. 1 (b).

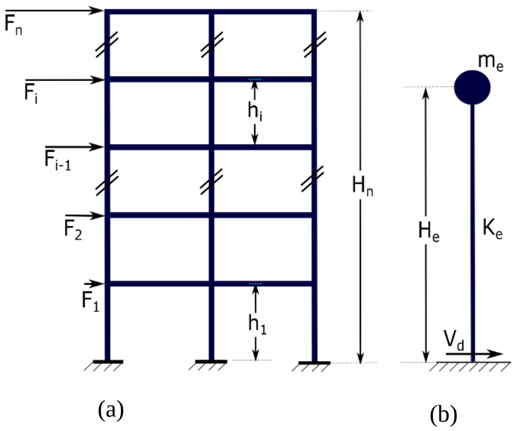


Figure 1 – The MDOF structure (a) and its SDOF representation in the original DDBD [4] method (b)

First the lateral displacement u_i at the storey i $(i = 1,2, \dots, n)$ corresponding to a design or limit IDR_T is obtained as

$$u_i = \omega_\theta \, IDR_T \, h_i \frac{(4H_n - h_i)}{(4H_n - h_1)} \tag{1}$$

where ω_θ is a reduction factor introduced to take into account higher mode effects and H_n and h_i are the total height of the frame and the height at storey i, respectively.

The effective mass m_e of the equivalent SDOF system is evaluated from

$$m_e = \sum_{i=1}^{n} (m_i u_i)/u_d \tag{2}$$

where m_i is the total mass at storey i and u_d is the characteristic or design displacement shown in Fig. 2 (a) and given by

$$u_d = \frac{\sum_{i=1}^{n}(m_i u_i^2)}{\sum_{i=1}^{n}(m_i u_i)} \tag{3}$$

The estimation of the equivalent damping ξ_{eq} for the case of R/C MRFs can be obtained from the relation

$$\xi_{eq} = 0.05 + 0.565\left(\frac{\mu - 1}{\mu\pi}\right) \tag{4}$$

where μ is the displacement ductility, i.e., $\mu = u_d/\mathrm{u}_y$, with u_y being the yield displacement shown in Fig. 2 (a). The yield displacement can be approximated as $u_y = H_e \mathrm{IDR}_y$, where $IDR_y = 0.5\varepsilon_y\,(L_b/h_b)$ with e_y, L_b and h_b being the material yield strain, the length of the beams between column centerlines and the depth of beam sections of the frames considered, respectively, and H_e is the effective height (see Fig. 1 (b)) expressed as

$$H_e = \frac{\sum_{i=1}^{n}(m_i u_i h_i)}{\sum_{i=1}^{n}(m_i u_i)} \tag{5}$$

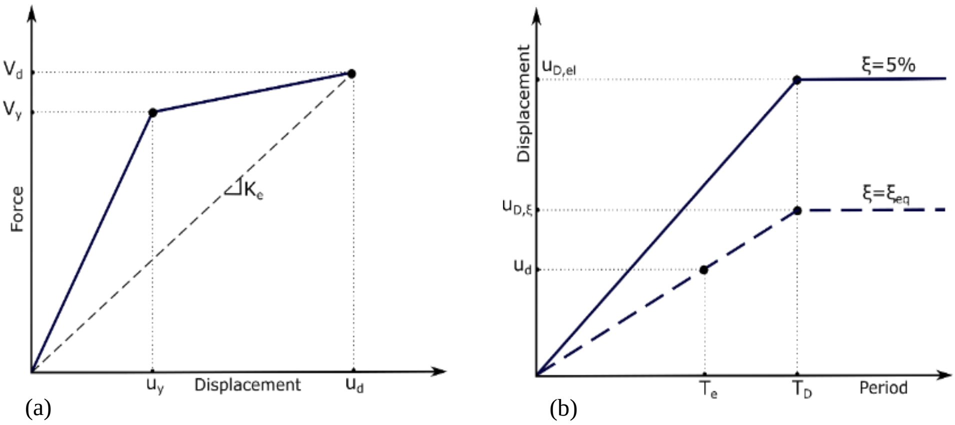
For known values of u_d and ξ_{eq} one can calculate with the aid of a displacement design spectrum like the one shown in Fig. 2 (b), the corresponding damped design displacement $u_{D,\xi}$ and then the effective period $T_e = (u_d/u_{D,\xi})\,T_D$ where, T_D and $u_{D,\xi}$ are the corner period of the displacement design spectrum and the design displacement with damping ξ=ξ_{eq} at T_D, respectively. Then, the effective stiffness is determined by $K_e = 4\pi^2 m_e/T_e^2$ and finally, the deformation dependent design base shear by

$$V_d = K_e u_d + c\sum_{i=1}^{n} P_i u_i/H_e \tag{6}$$

where the second term takes care of P-Δ effects with P_i denoting the total gravity load on storey level i and $c = 0.5$ for R/C structures. One can dimension the frame members after distributing the above design base shear to the floor masses of the frame as:

$$\begin{array}{ll} \text{Floors 1 to n-1:} & F_i = k\,V_d(m_i\Delta_i)/\sum_{i=1}^{n}(m_i u_i) \\ \text{Floor n:} & F_i = (1-k)\,V_d + \mathrm{k}(m_i\Delta_i)/\sum_{i=1}^{n}(m_i u_i) \end{array} \tag{7}$$

where k=0.9, which means that 10% of the base shear is assumed to be additionally applied at roof level in order to take care of higher-modes effects.



(a) (b)

Figure 2 – The effective stiffness at the design target displacement u_d (a) and the design displacement spectra for equivalent damping ξ_{eq} (b)

3. Equivalent modal damping ratios

For reasons of completeness, this section, mainly taken from a previous work of the present authors [17], presents the theory of calculating the equivalent modal damping ratios for R/C plane frames. Consider first the transfer function R(ω) for a viscously damped linear elastic MDOF plane frame defined in the frequency domain as the ratio of the absolute roof acceleration $\overline{\ddot{\mathrm{U}}}_{\mathrm{r}}(\omega)$ of the frame over the acceleration $\overline{\ddot{\mathrm{u}}}_{\mathrm{g}}(\omega)$ at its base, i.e.,

$$\mathrm{R}(\omega) = \frac{\overline{\ddot{\mathrm{U}}}_{\mathrm{r}}(\omega)}{\overline{\ddot{\mathrm{u}}}_{\mathrm{g}}(\omega)} \tag{8}$$

where $\overline{\ddot{\mathrm{U}}}_{\mathrm{r}}(\omega) = \overline{\ddot{\mathrm{u}}}_{\mathrm{g}}(\omega) + \overline{\ddot{\mathrm{u}}}_{\mathrm{r}}(\omega)$ with $\overline{\ddot{\mathrm{u}}}_{\mathrm{g}}(\omega)$ and $\overline{\ddot{\mathrm{u}}}_{\mathrm{r}}(\omega)$ being the earthquake motion and roof relative motion, respectively, in the frequency domain, ω is the frequency and overbars denote Fourier transformation.

The squared modulus $|\mathrm{R}(\omega)|^2$ of this transfer function $R(\omega)$ can be written in the form

$$\begin{aligned} |\mathrm{R}(\omega=\omega_{\mathrm{k}})|^2 &= 1 + 2\sum_{\mathrm{j}=1}^{\mathrm{N}} \frac{\varphi_{\mathrm{rj}}\Gamma_{\mathrm{j}}\omega^2\left(\omega_{\mathrm{j}}^2-\omega^2\right)}{\left(\omega_{\mathrm{j}}^2-\omega^2\right)^2+\left(2\xi_{\mathrm{j}}\omega_{\mathrm{j}}\omega\right)^2} \\ &+ \sum_{\mathrm{j}=1}^{\mathrm{N}} \frac{\varphi_{\mathrm{rj}}^2\Gamma_{\mathrm{j}}^2\omega^4\left(\omega_{\mathrm{j}}^2-\omega^2\right)^2+4\xi_{\mathrm{j}}^2\omega_{\mathrm{j}}^2\omega^2}{\left[\left(\omega_{\mathrm{j}}^2-\omega^2\right)^2+\left(2\xi_{\mathrm{j}}\omega_{\mathrm{j}}\omega\right)^2\right]^2} \\ &+ 2\sum_{\mathrm{j}\neq\mathrm{m},\mathrm{m}>\mathrm{j}}^{\mathrm{N}} \frac{\varphi_{\mathrm{rj}}\Gamma_{\mathrm{j}}\varphi_{\mathrm{rm}}\Gamma_{\mathrm{m}}\omega^4\left[\left(\omega_{\mathrm{j}}^2-\omega^2\right)\left(\omega_{\mathrm{m}}^2-\omega^2\right)+4\xi_{\mathrm{j}}\xi_{\mathrm{m}}\omega_{\mathrm{j}}\omega_{\mathrm{m}}\omega^2\right]}{\left[\left(\omega_{\mathrm{j}}^2-\omega^2\right)^2+\left(2\xi_{\mathrm{j}}\omega_{\mathrm{j}}\omega\right)^2\right]\left[(\omega_{\mathrm{m}}^2-\omega^2)^2+(2\xi_{\mathrm{m}}\omega_{\mathrm{m}}\omega)^2\right]} \end{aligned} \tag{9}$$

where ξ_j, ξ_m and Γ_j, Γ_m are the damping ratio and the corresponding participating factor at mode j or m, respectively, φ_{rj} or φ_{rm} is the j_{th} or m_{th} modal shape at the top storey (roof) r and ω_j and ω_m are the natural frequencies corresponding to the eigenvalue problem with $m > j$. This form of the transfer function corresponds to a real number and not to a complex number as it is the case with $R(\omega)$.

Consider now a nonlinear MDOF plane frame to be replaced by an equivalent linear MDOF plane frame with high amounts of viscous damping. These two MDOF frames are equivalent in the sense that the work of dissipation due to hysteretic forces in the nonlinear system is equal to the work of dissipation due to viscous forces in the linear structure. Thus, this work equivalence concept can be thought of as an extension of that of Jacobsen [18] from a SDOF system under harmonic excitation to a MDOF system under seismic excitation.

The criterion of equivalence of those two works according to the theory of linear systems, states that the modulus of the transfer function versus frequency curve for a linear system with viscous damping is smooth with local maxima being at the resonant frequencies. When the distorted shape with many and no clearly visible peaks of that curve for the non-linear structure, as shown in Fig. 3 (a), becomes smooth with clearly visible peaks for the first few modes, as shown in Fig. 3 (b), this curve represents the equivalent linear structure. By providing Rayleigh type viscous damping progressively to the nonlinear structure, one succeeds in obtaining smoother and smoother $|R(\omega)|^2$ versus ω curves for that structure until for some value of damping the curve becomes completely smooth with clearly visible peaks (Fig. 3 (b)). Once the $|R(\omega)|^2$ versus ω curve for all modes has become completely smooth, the structure is just below its first yield point (first plastic hinge) and hence, the originally non - linear structure has become an equivalent linear. At that moment, Eq. (9) is applicable and one can evaluate R(ω) for a sequence of values of the resonant frequencies of $\omega = \omega_k$ $(k = 1,2, \dots, K)$, with K being the number of the first significant modes and thus, create a system of K nonlinear algebraic equations to be solved for the modal hysteretic damping ratios ξ_k, provided that φ_{rj}, ω_j, Γ_j and $R(\omega_k)$ are known.

Thus, for a large number of plane frames under a large number of far-fault earthquake motions one determines structural seismic response in the time domain by non-linear time history (NLTH) analyses involving large displacement and P-Δ effects. For this purpose, the Ruaumoko 2D software [19] was used. All structural elements were modeled using a component (Giberson) beam element with concentrated hinges described by the modified Takeda hysteresis rule at their both ends. The interaction between axial force and yield moment was taken into account for columns. The stiffness and strength deterioration were considered for all members. More information about the modeling adopted here can be found in [17].

This response is then brought to the frequency domain by a Fourier transform for the construction of the transfer function R(ω) from which the equivalent modal damping ratios ξ_k can be calculated. NLTH analyses are done many times for every seismic motion by progressively scaling that motion in order to capture all the target deformation values. Thus, the resulting ξ_k turn out to be deformation dependent, while due to the way they have been derived, include higher mode and P-Δ effects. Information about the modeling adopted here can be found in [17].

Table 1, taken from [17], provides explicit empirical expressions of $\xi_{eq,j}$ as functions of period T for the first four modes (j=1, 2, 3, 4) and four values of IDR_T (performance levels) only for the case of soil type B (as categorized by EC8[1]). Expressions for soil types A, C and D can be found in [20].

Table 1 – Equivalent modal viscous damping ratios (ξ_k%) for R/C MRFs for different performance levels, soil type B and various values of IDR_T ($\xi_{eq,k} = \xi_k + 5\%$)

	SOIL B							
	Mode 1		Mode 2		Mode 3		Mode 4	
IDR_T=1.0%	0.85<=T<=4.6 ξ=2.9		0.21<=T<=0.46 ξ=-8.4T+5	0.46<=T<1.71 ξ=1.2	0.16<=T<=0.45 ξ=-7.9T+4.5	0.45<=T<1.02 ξ=0.9	0.14<=T<0.34 ξ=-14.5T+5.93	0.34<=T<0.7 ξ=1
IDR_T=1.5%	0.85<=T<=1.76 ξ=-5.1T+18.8	1.76<=T<=4.6 ξ=9.9	0.21<=T<=1.01 ξ=-4.1T+8.7	1.01<=T<1.71 ξ=4.5	0.16<=T<=0.62 ξ=-10.4T+9.5	0.62<=T<1.02 ξ=3	0.14<=T<0.51 ξ=-8.4T+7.3	0.51<=T<0.7 ξ=3
IDR_T=2.5%	0.85<=T<=1.8 ξ=-7.89T+38.2	1.8<=T<4.6 ξ=-2.2T+27.9	0.21<=T<=1.01 ξ=-26.25T+35.51	1.01<=T<1.71 ξ=9	0.16<=T<=0.51 ξ=-55.4T+34.4	0.51<=T<=1.02 ξ=-0.19T+6.2	0.14<=T<0.42 ξ=-106.8T+50.9	0.42<=T<=0.7 ξ=6
IDR_T=4.0%	0.85<=T<=3.1 ξ=-28.9T+94.6	3.1<=T<=4.6 ξ=48	0.21<=T<=0.9 ξ=-69.4T+94.5	0.9<=T<=1.71 ξ=-18.1T+48.3	0.16<=T<=1.02 -		0.14<=T<0.70 -	

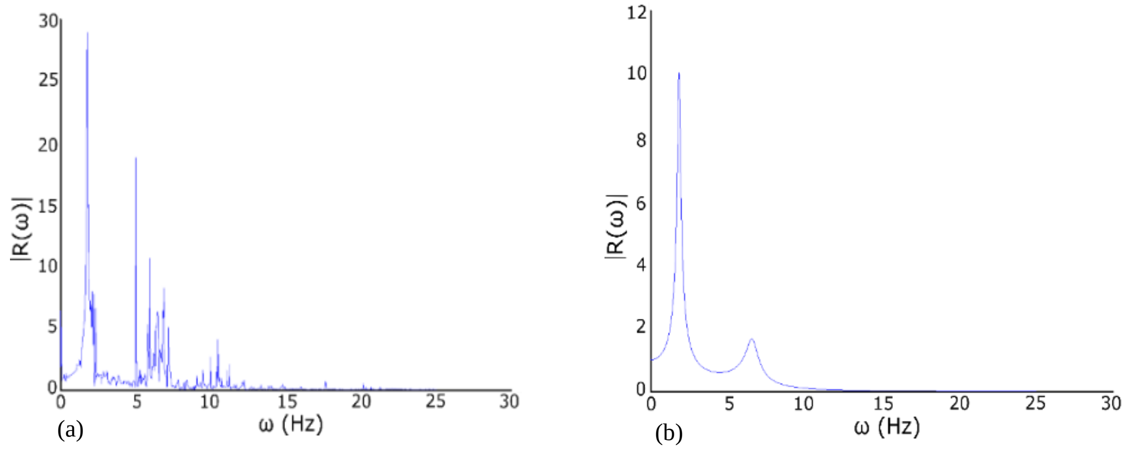


Figure 3 –Transfer function of (a) a non-linear structure and (b) its equivalent linear structure with damping

4. The proposed DDBD method

This section presents in detail the proposed improved DDBD method, which utilizes an equivalent linear MDOF system with viscous damping instead of the corresponding SDOF system employed by the original DDBD [4] method. Thus, in here one has modal lateral displacements $u_{i,j}$ at the storey i for the mode j instead of the lateral displacements u_i of the original DDBD [4] method. Index $i = 1,2,\ldots,n$, where n is the total number of stories of the frame and index $j = 1,2\ldots,K$, where K identifies the first K modes significantly contributing to the seismic structural response. Furthermore, here one has equivalent modal damping ratios $\xi_{eq,j}$ $(j = 1,2,\ldots,K)$ instead of one value of ξ_{eq} for the SDOF system.

The modal displacement profiles $u_{i,j}$ for a target inter-storey drift ratio IDR_T are estimated here based on the first four modal shapes resulting from modal analysis. In order to achieve this, the modal interstorey drift ratio IDR_{Tj} for each mode j is first introduced here and defined as

$$IDR_{Tj} = f_j\, IDR_T \tag{10}$$

where f_j is the mass participation factor for mode j. The modal target displacements $u_{i,j}$ corresponding to the IDR_{Tj} can be expressed in terms of the modal shapes $\phi_{i,j}$ of the frame in the form

$$u_{i,j} = x_j \phi_{i,j} \tag{11}$$

where x_j is a modal multiplication factor with dimensions of length to be determined.

One can easily obtain from Eq. (11) the relation

$$\left(\frac{u_{i+1,j} - u_{i,j}}{h_{i+1} - h_i}\right) = x_j \left(\frac{\phi_{i+1,j} - \phi_{i,j}}{h_{i+1} - h_i}\right) \tag{12}$$

and observing that

$$IDR_{Tj} = \max\left(\frac{u_{i+1,j} - u_{i,j}}{h_{i+1} - h_i}\right)_j \tag{13}$$

he can have from Eq. (12) the expression

$$IDR_{Tj} = \max\left(\frac{\phi_{i+1,j}-\phi_{i,j}}{h_{i+1}-h_i}\right)_j x_j \tag{14}$$

which can be solved for x_j and provide

$$x_j = IDR_{Tj} \, / \, \max\left(\frac{\phi_{i+1,j}-\phi_{i,j}}{h_{i+1}-h_i}\right)_j \tag{15}$$

By substituting Eq. (15) in Eq. (11), one can finally express the modal target displacement $u_{i,j}$ in terms of the modal IDR_{Tj}.

Having found the modal target displacements $u_{i,j}$ one can proceed to the calculation of the characteristic or design modal displacements $u_{d,j}$ from a relation analogous to Eq. (3), which reads

$$u_{d,j} = \frac{\sum_{i=1}^{n}(m_{i,j}u_{i,j}^2)}{\sum_{i=1}^{n}(m_{i,j}u_{i,j})} \tag{16}$$

where $m_{i,j}$ is the modal mass (m_i x f_j) at mode j and storey i.

The equivalent modal damping ratios $\xi_{eq,j} = \xi_j + 5\%$ can be determined by the method briefly described in Section 3, where mode index k is used instead of the present index j. One can find there (Table 1) explicit expressions of ξ_j for the first four significant modes ($j = 1,2,3,4$) of the structure, soil type B and various values of IDR_T.

Using the design modal displacements $u_{d,j}$ of Eq. (16) in conjunction with values of the equivalent modal damping ratios $\xi_{eq,j}$ obtained from Table 1, one can easily determine with the aid of a design displacement spectrum like the one shown in Fig. 4, the corresponding damped design modal displacements $u_{\xi,j}$ and thus, the effective modal periods $T_{e,j}$ of the frame as

$$T_{e,j} = \frac{u_{d,j}}{u_{D,\xi}} T_D \tag{17}$$

where T_D and $u_{D,\xi}$ are the corner period of the displacement design spectrum and the displacement with damping $\xi = \xi_{eq,j}$ at T_D, respectively (Fig. 2(b)).

Hence, the modal effective stiffness $K_{e,j}$ can be obtained by

$$K_{e,j} = \frac{4\pi^2 m_{e,j}}{T_{e,j}^2} \tag{18}$$

where the modal mass $m_{e,j}$, in view of Eq. (2), is equal to

$$m_{e,j} = \sum_{i=1}^{n}(m_{i,j}u_{i,j})/u_{d,j} \tag{19}$$

Thus, the modal design base shear $V_{d,j}$ can be obtained by

$$V_{d,j} = K_{e,j} u_{d,j} \tag{20}$$

The above modal design base shear $V_{d,j}$ is distributed to the floor masses of the frame in the form of lateral modal design forces $F_{i,j}$ reading as

$$F_{i,j} = V_{d,j} \frac{m_{i,j} u_{i,j}}{\sum_{i=1}^{n} m_{i,j} u_{i,j}} \tag{21}$$

Finally, the maximum lateral design force F_i at every storey i (Fig. 1) can be obtained by a modal combination rule, such as the SSRS rule, in the form

$$F_i = \sqrt{\sum_{j=1}^{K} F_{i,j}^2} \tag{22}$$

The above maximum lateral design forces can be used to design an R/C frame through static analysis. It should be noted here that since the proposed method works on the basis of a MDOF representation of the structural system, it takes into account, by default, both P-Δ and higher mode effects without the need of an artificial increase of the lateral forces as is the case of the original DDBD [4] method.

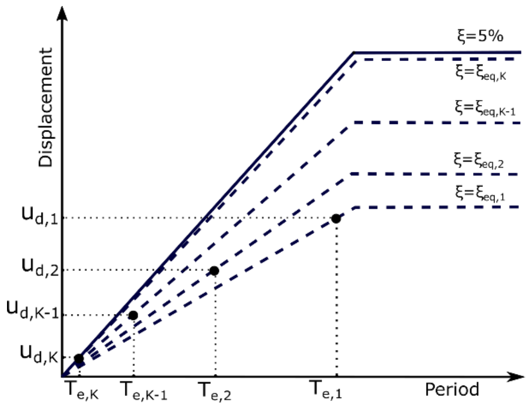


Figure 4 – The design displacement spectrum for equivalent modal damping ratios $\xi_{eq,j}$

5. Design example

The proposed DDBD method is illustrated here through a design example involving a sixteen storey R/C-MRF. For comparison purposes this frame is also designed on the basis of the original DDBD [4] and the FBD method of EC8 [1], while the responses are compared on the basis of NLTH analyses.

The frame was first designed based on EC2 [21] and EC8 [1] and all sectional dimensions produced by these codes were used as starting sections for the frames to be designed by both the proposed and the original DDBD [4] methods. Thus, the frame was first designed based on the EC8 [1] design spectrum for peak ground acceleration (PGA) equal to 0.36g, soil type B and a behavior factor q=3.9. The vertical load was assumed to be g_d=21 kN/m and q_d=3 kN/m distributed on beams and g_c=60 kN, q_c=9 kN and g_c=70 kN, q_c=18 kN concentrated on the outer and middle columns, respectively, where g and q denote dead and live loads, respectively. The combinations of 1.35g+1.5q and g+0.3q±seismic forces were used for the design. Concrete and steel material properties were assumed to be those for C25/30 and S500, respectively. The design was accomplished with the aid of SAP2000 [22]. The geometry is summarized in Fig. 5 and the required sectional dimensions with the corresponding reinforcing steel ratios are given in Table 2, where only results for the first three and the last two storeys are presented due to space limitations. Symbols used in Fig. 5 and Table 2 and all subsequent ones, are as follows: h is the cross-section height of columns and beams, ρ is the reinforcement ratio, i.e., the reinforcement area normalized to the cross-sectional area bd, where b is the width, h is the height and d=h-4 cm; the symbol ρ_{tot} indicates the total reinforcement ratio of columns and $\rho_{1A}, \rho_{2A}, \rho_{1B}$ and ρ_{2B} indicate the reinforcement ratios at the left and right ends of the beams. Only the necessary geometric sizes and reinforcement of the members of the frame are shown in Table 2. Frames are symmetric, i.e., with reference to Fig. 2, outer (1,4) and middle (2,3) columns have the same geometry and reinforcement, beams No. 5 and No. 7 also have the same geometry and reinforcement, and so on.

Table 2 – Sectional dimensions and reinforcing steel ratios of the sixteen storey frame designed by the EC2[21] and the EC8 [1]

Column	b (cm)	h (cm)	ρ_{tot}(%)	Beam	b (cm)	h (cm)	$\rho_{1,A}$(%)	$\rho_{2,A}$(%)	$\rho_{1,B}$(%)	$\rho_{2,B}$(%)
1	55	55	1.00	5	30	60	1.34	0.94	1.32	0.91
2	60	60	1.00	6	30	60	1.30	0.91	1.30	0.91
8	55	55	1.00	12	30	60	1.35	0.96	1.30	0.91
9	60	60	1.00	13	30	60	1.34	0.94	1.34	0.94
15	55	55	1.00	19	30	60	1.32	0.93	1.25	0.84
16	60	60	1.00	20	30	60	1.32	0.92	1.32	0.92
99	40	40	1.00	103	30	45	0.87	0.47	0.72	0.37
100	45	45	1.00	104	30	45	0.90	0.51	0.90	0.51
106	40	40	1.00	110	30	45	0.48	0.28	0.48	0.28
107	45	45	1.00	111	30	45	0.60	0.34	0.60	0.34

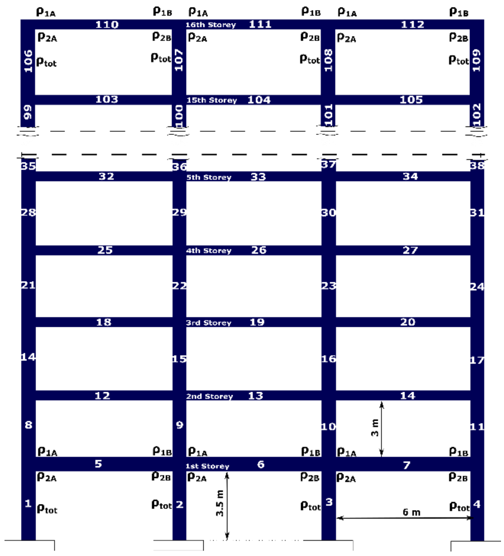


Figure 5 – Frame geometry, section and reinforcing steel numbering for the design example

The comparison of total design base shear and design lateral force profiles as obtained by the proposed DDBD, original DDBD [4] and the FBD method of EC8[1] is given in Fig. 6 (a) and (b), respectively. Figure 6 contains two cases of the lateral force profiles of the original DDBD [4] method, i.e., including P-Δ effects (c=0.5 in Eq. (6)) and higher-modes effects modifications ($\omega_\theta = 0.85$ in Eq. (1) and k=0.9 in Eq. (7)) and without including these effects and modifications. It is evident that in the original DDBD [4] method the inclusion of these effects is done in a rather artificial way. Without the inclusion of these effects, the design lateral force of the original DDBD [4] method results in a much lower value than the one produced by the proposed method as it is shown in Fig. 6 (a) and (b). The proposed method takes into account these effects in a rather rational way, since it works on the basis of the MDOF representation of the structure in contrast to the original DDBD [4] method which works with a SDOF representation of the building. By including P-Δ and higher modes effects, one can observe in Fig. 6 that the response of the original method approaches the one produced by the proposed method. However, even though the total base shear produced by the proposed and the original DDBD [4] method resulted in being almost the same, the lateral force profiles are different.

The comparison of the deformation response of the above two frames as obtained by NTLH analyses is shown in Fig. 6 (c) for the proposed DDBD, the original DDBD [4] and the FBD method of EC8 [1]. One can observe there that all frames did not exceed the target design value of $IDR_T = 2.50\%$. The frame here designed by the proposed DDBD method resulted in having maximum IDR response much closer to the design target IDR_T than the other two methods. Thus, one can conclude here that the proposed method, based on a rather rational approach, results in excellent performance for the cases where higher mode and P-Δ effects are significant. This is done without any rather artificial modification of the lateral forces and the displacement profile, as it is the case with the original DDBD [4].

Table 3 – Storey characteristic structural and design force values for the frame designed by the proposed DDBD method

Storey i	M_i	$m_{i,j}$(kNsec²/m)				$\phi_{i,j}$				$u_{i,j}$(m)				$F_{i,j}$(kN)				F_i(kN)
		1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	
1	68.30	49.51	9.44	3.14	1.74	0.036	-0.097	0.128	-0.190	0.0229	-0.0032	0.0006	-0.0003	1.29	4.61	8.23	9.48	13.44
2	68.30	49.51	9.44	3.14	1.74	0.084	-0.223	0.284	-0.404	0.0536	-0.0074	0.0012	-0.0006	3.03	10.57	18.26	20.14	29.33
3	68.30	49.51	9.44	3.14	1.74	0.136	-0.352	0.423	-0.558	0.0873	-0.0117	0.0018	-0.0008	4.92	16.68	27.24	27.83	42.65
4	68.30	49.51	9.44	3.14	1.74	0.191	-0.471	0.521	-0.606	0.1221	-0.0156	0.0022	-0.0008	6.89	22.34	33.50	30.22	50.81
5	68.36	49.51	9.44	3.14	1.74	0.252	-0.584	0.559	-0.510	0.1614	-0.0193	0.0024	-0.0007	9.11	27.66	35.99	25.43	52.82
6	68.36	49.51	9.44	3.14	1.74	0.318	-0.673	0.518	-0.270	0.2033	-0.0223	0.0022	-0.0004	11.47	31.90	33.33	13.48	49.41
7	68.36	49.51	9.44	3.14	1.74	0.384	-0.728	0.394	0.057	0.2460	-0.0241	0.0017	0.0001	13.88	34.49	25.36	-2.82	45.09
8	68.36	49.51	9.44	3.14	1.74	0.459	-0.738	0.173	0.418	0.2934	-0.0245	0.0007	0.0006	16.55	34.98	11.15	-20.86	45.36
9	68.36	49.51	9.44	3.14	1.74	0.537	-0.690	-0.112	0.671	0.3437	-0.0229	-0.0005	0.0009	19.38	32.71	-7.19	-33.49	51.18
10	68.36	49.51	9.44	3.14	1.74	0.612	-0.584	-0.383	0.682	0.3916	-0.0194	-0.0016	0.0009	22.09	27.68	-24.65	-34.02	54.95
11	68.36	49.51	9.44	3.14	1.74	0.697	-0.384	-0.615	0.352	0.4459	-0.0127	-0.0026	0.0005	25.15	18.21	-39.61	-17.57	53.31
12	68.36	49.51	9.44	3.14	1.74	0.779	-0.119	-0.687	-0.194	0.4983	-0.0039	-0.0030	-0.0003	28.11	5.62	-44.22	9.66	53.58
13	68.36	49.51	9.44	3.14	1.74	0.852	0.182	-0.540	-0.668	0.5453	0.0060	-0.0023	-0.0009	30.76	-8.61	-34.75	33.31	57.77
14	68.36	49.51	9.44	3.14	1.74	0.910	0.461	-0.211	-0.756	0.5821	0.0153	-0.0009	-0.0010	32.84	-21.83	-13.58	37.72	56.24
15	68.36	49.51	9.44	3.14	1.74	0.951	0.687	0.198	-0.402	0.6086	0.0228	0.0009	-0.0005	34.33	-32.57	12.75	20.06	52.96
16	46.64	49.51	9.44	3.14	1.74	1.000	1.000	1.000	1.000	0.6399	0.0331	0.0043	0.0014	36.09	-47.38	64.36	-49.89	100.89

Table 4 – Modal characteristic structural and design force values for frame designed by the proposed DDBD method

Mode j	T_j(sec)	$m_{e,j}(kNsec^2/m)$	f_j	IDR_j(%)	x_j	$u_{d,j}$(m)	$\xi_{eq,j}$(%)	$T_{e,j}$(sec)	$K_{e,j}$(kN/m)	$V_{d,j}$(kN)
1	3.33	573.64	0.72	1.81	0.64	0.4527	25.55	5.897162	651.19	294.79
2	1.27	20.86	0.14	0.35	0.03	0.0497	14.00	0.510526	3160.32	157.06
3	0.76	2.31	0.05	0.11	0.00	0.0096	11.05	0.091065	11010.60	106.18
4	0.56	0.71	0.03	0.06	0.00	0.0046	11.00	0.043385	14923.90	68.68

Table 5 – Reinforcing steel ratios of the frame designed by the proposed and the original DDBD [4] method

Proposed DDBD							Original DDBD						
Column	ρ_{tot}(%)	Beam	$\rho_{1,A}$(%)	$\rho_{2,A}$(%)	$\rho_{1,B}$(%)	$\rho_{2,B}$(%)	Column	ρ_{tot}(%)	Beam	$\rho_{1,A}$(%)	$\rho_{2,A}$(%)	$\rho_{1,B}$(%)	$\rho_{2,B}$(%)
1	1.00	5	1.13	0.73	1.12	0.73	1	1.00	5	0.76	0.38	0.76	0.38
2	1.00	6	1.11	0.72	1.11	0.72	2	1.00	6	0.75	0.37	0.75	0.37
8	1.00	12	1.25	0.85	1.22	0.81	8	1.00	12	0.81	0.42	0.80	0.41
9	1.00	13	1.23	0.84	1.23	0.84	9	1.00	13	0.81	0.41	0.81	0.41
15	1.00	19	1.25	0.85	1.21	0.80	15	1.00	19	0.84	0.44	0.81	0.41
16	1.00	20	1.23	0.84	1.23	0.84	16	1.00	20	0.84	0.44	0.84	0.44
99	1.02	103	0.67	0.34	0.60	0.34	99	1.02	103	0.67	0.34	0.60	0.34
100	1.02	104	0.69	0.34	0.69	0.34	100	1.02	104	0.68	0.34	0.68	0.34
106	1.02	110	0.46	0.28	0.47	0.28	106	1.02	110	0.46	0.28	0.48	0.28
107	1.02	111	0.53	0.28	0.53	0.28	107	1.02	111	0.53	0.28	0.53	0.28

Table 6 – Storey characteristic structural and design force values for the frame designed by the original DDBD [4] method

Storey i	u_i(m)	u_d(m)	H_e(m)	μ	m_e(kNsec²/m)	ξ_{eq}(%)	$u_{D\xi}$(m)	T_e(sec)	K_e(kN/m)	V_d(kN)	F_i(kN)
1	0.074										7.18
2	0.136										13.13
3	0.196										18.88
4	0.253										24.44
5	0.309										29.80
6	0.362										34.97
7	0.414										39.94
8	0.463	0.570	32.50	1.19	894.56	7.9	0.226	5.05	549.22	799.73	44.73
9	0.511										49.31
10	0.556										53.71
11	0.600										57.91
12	0.641										61.92
13	0.681										65.73
14	0.718										69.35
15	0.754										72.77
16	0.787										155.98

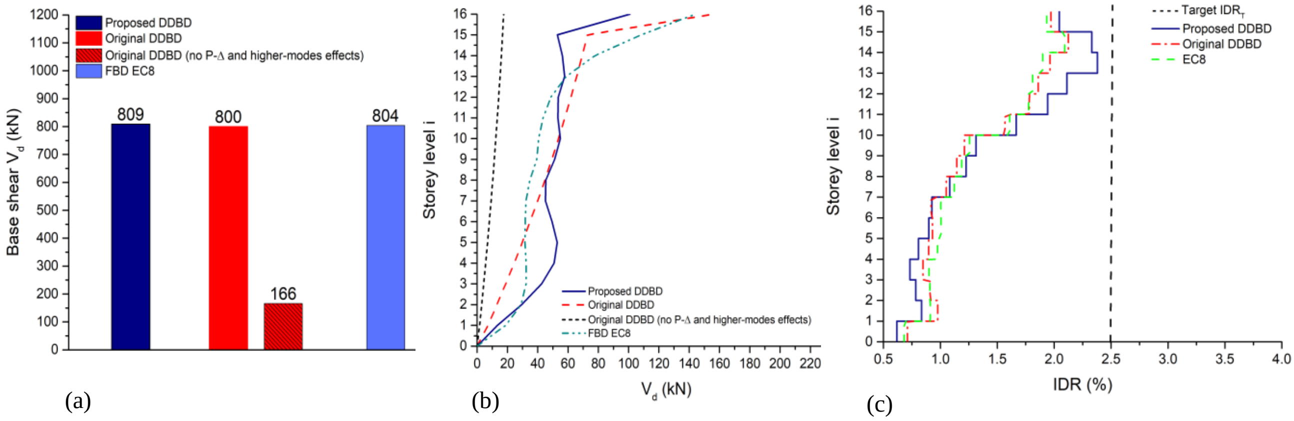


Figure 6 – Comparison of the (a) total design base shear, (b) the design lateral forces and (c) the IDR response profiles of the frame designed by the proposed DDBD, the original DDBD [4] and the FBD of EC8 method

6. Conclusions

From the preceding developments and examples, the following conclusions can be drawn:

1) An improved direct displacement-based design (DDBD) method has been proposed for plane R/C MRFs, which employs a multi-degree-of-freedom equivalent system instead of the single-degree-of-freedom system used by the original method. The method can be used on the basis of either two performance levels (as in current codes) or four performance levels.

2) The method uses the concept of deformation dependent equivalent modal damping ratios previously developed by the present authors for other purposes and the concept of the design modal displacements developed herein. The design modal displacements are determined on the basis of target inter-storey displacement ratios for every performance level and the first few modes significantly contributing to the structural response.

3) The proposed method by its nature can take more rationally and with higher accuracy into account P-Δ and higher mode effects than the original one. However, the proposed method requires an elastic modal analysis for the determination of natural frequencies and modal shapes of the first few modes. Both original and proposed DDBD methods perform the design in one step (strength checking) and not in two steps (strength and deformation checking) as it is the case with the FDB method of EC8.

4) Comparisons with the original DDBD and the FBD method of EC8 on the assumption of members designed to have the same concrete cross-sections showed that all methods result in safely designed frames with the proposed method to produce frames with deformation responses closer to the target response and hence with better use of the required material.

7. Acknowledgements

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