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# DIRECT DISPLACEMENT BASED DESIGN OF REGULAR CONCRETE FRAMES IN COMPLIANCE WITH EUROCODE 8

*Damir Džakić, Ivan Kraus, Dragan Morić*

Subject review
The new concept in designing structures to achieve a specified performance limit state was first introduced in New Zealand, in 1993. Over the following years, USA and Europe have put a great effort focused on research and development of the concept as a viable and logical alternative to the current force-based code approaches. The paper presents the theory and application of this method using a reinforced concrete frame structures as an example. The frame structure is designed with implementing Eurocode 8 regulations. Furthermore, results obtained using direct displacement based design method are compared to the ones obtained using multimodal response spectrum method. Among other things, significant differences are highlighted in regard to current design regulations.

***Keywords***: *displacement-based design, Eurocode 8, example, frame structures, reinforced concrete, seismic*

## Proračun armiranobeonske okvirne konstrukcije sukladno očekivanom pomaku u skladu s propisima Eurocode 8

Pregledni članak
Novi koncept oblikovanja konstrukcija s ciljem postizanja određenog graničnoga stanja prvi puta je predstavljen u Novom Zelandu, 1993. Tijekom narednih godina u SAD-u i Europi se ulaže veliki napor usmjeren na istraživanje i razvoj ovoga koncepta kao održivog i logički alternativnog trenutnom pristupu proračuna preko sile. U ovom je radu prikazana osnovna teorija i primjena metode proračuna sukladno očekivanom pomaku na primjeru armiranobetonske okvirne konstrukcije. Okvirna konstrukcija je proračunata koristeći Eurocode 8 propise. Nadalje, rezultati dobiveni koristeći proračun sukladno očekivanom pomaku, uspoređeni su s rezultatima dobivenim koristeći višemodalnu spektralnu analizu. Između ostaloga naglašene su bitne razlike u odnosu na trenutno propisane postupke proračuna, kao i nedostatci istih.

***Ključne riječi***: *armirani beton, Eurocode 8, okvirne konstrukcije, primjer, proračun sukladno očekivanom pomaku, seizmika*

## 1 Introduction

Over the past few decades earthquake engineering has experienced a kind of revision of methods and philosophies used till now. A new approach in design called Performance Based Design is under continuous development. In addition, numerous nonlinear dynamic analyses have been made and have become a contemporary tool in the field of earthquake engineering research. The mentioned engineering tools are primarily directed onto concrete structures rather than to steel structures, mostly due to the fact that steel, in comparison to concrete, is a material with well-known properties and characteristics. Although the new tools have found their place in the field of research, the professional field of structural engineering is neglecting them mainly because of time consumption and complexity.

Current regulations are mostly defined through probabilistic theory which considers seismic excitation without taking any damage and collapse risks directly into account. Design procedures defined in that way are well accepted and among other things, they have a long tradition. It is important to state that those procedures allow only the check of displacements and drifts of structures at the end of an analysis, without a real insight into the damage and collapse risk level.

Among several different procedures developed in terms of Performance Based Design, the most significant progress was shown in a procedure called Direct Displacement Based Design (DDBD further on). This procedure is deterministically based and shown as very rational and effective in structural analysis and design as it controls structural displacements and thus it controls damage level and collapse risk. It is important to mention that the method is primarily defined as post-elastic.

This paper shows the basics of theory and application of DDBD applied on a characteristic concrete frame in compliance with Eurocode 8. Moreover, main differences, advantages and disadvantages in comparison to other methods are shown.

## 2 Direct displacement based design (DDBD)

DDBD is extensively developing with the aim to correct deficiencies in analysis and design made according to, today mostly used, force based design procedures. The fundamental problem of force based design, particularly when applied to concrete and masonry structures, is the selection of appropriate member stiffness. It is important to make good estimation of member stiffness since the earthquake induced forces are distributed between structural elements proportionally to their stiffness. This is especially important when analysing concrete and masonry structures and when one needs to decide whether to take cracked or un-cracked member stiffness into account. Another problem with force based design methods is when calculating the structural period of vibration which is mostly taken to be low. Lower natural periods lead to greater seismic forces which again lead to oversized structural elements and/or, as in the case of reinforced concrete (RC further on) members, to a greater amount of reinforcing steel. In force based design procedures this approach is mostly considered safe-sided. However, such underestimation of the period of vibration has just the opposite effect since the displacements, calculated on the basis of unrealistically small periods, are also unrealistically small

(Fig. 1). If we consider that the displacement capacity, in comparison to strength, is a key and most important characteristic in defining inelastic behaviour, it is obvious that we are not on the safe side with lower periods of vibration.

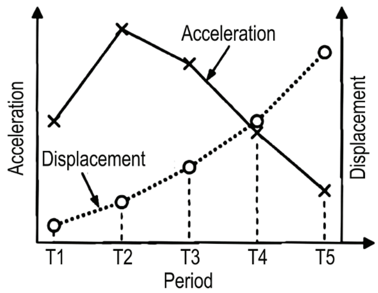


**Figure 1** Elastic response spectrum [1]

The fundamental principle of DDBD is to design structures to achieve a given performance limit state for a specified earthquake intensity rather than being bound by the very limit state as it is the case in current regulations. This kind of approach results in structures of "uniform risk".

The fundamentals of DDBD method are shown in Fig. 2, which considers a SDOF representation of a frame building (Fig. 2a), although it can be applied to any structural type.

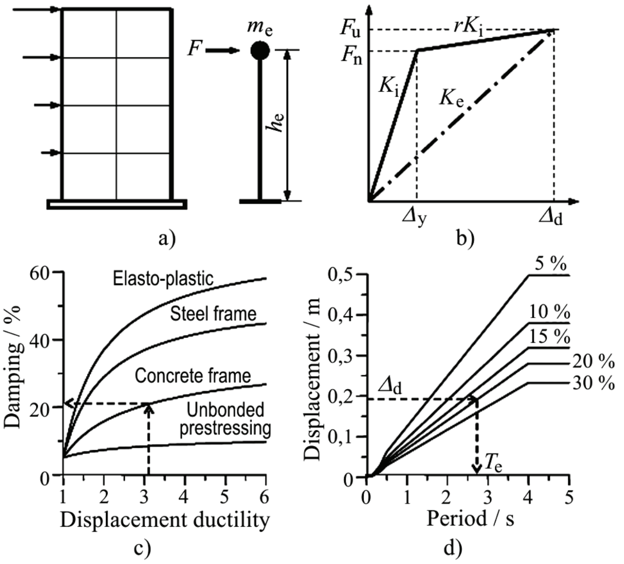


**Figure 2** Fundamentals and design method of DDBD [1]

While force based seismic design characterizes a structure in terms of elastic, pre-yield, properties (initial stiffness $K_i$, elastic damping), DDBD characterizes the structure by secant stiffness $K_e$ at maximum displacement $\Delta_d$ (Fig. 2b), and a level of equivalent viscous damping $\xi_{eq}$, representative of the combined elastic damping and the hysteretic energy absorbed during inelastic response [1, 2]. Thus, as shown in Fig. 2c, for a given level of ductility demand, the corresponding equivalent viscous damping can be determined [1]. With the design displacement at maximum response determined from a desired or given performance state, and the corresponding damping estimated from the expected ductility demand, the effective period $T_e$ at maximum displacement response, measured at the effective height $H_e$ (Fig. 2a), can be read from a set of displacement spectra for different levels of damping, as shown in the example of Fig. 2d, [1].

Therefore and as recommended in [3], modelling of structures discussed here is conducted by using a bilinear, elastoplastic force-displacement diagram for a substitute SDOF model. It approximates the multi-degree of freedom structure at peak response, thus the effective stiffness of the structure is significantly lower than that for an "elastic" structure.

Furthermore, the inelastic behaviour is represented by equivalent structural damping which is taken to correspond to a level of displacement ductility demand, based on energy dissipation capabilities of structure and structural material. Based on this and pre-defined design displacement it is possible to determine the effective period $T_e$ from the displacement response spectrum at the maximum displacement $\Delta_d$. The effective secant stiffness can then be determined by using simple equations defined for SDOF system [1, 4, 5]:

$$K_e = 4 \cdot \pi^2 \cdot \frac{m_e}{T_e^2}, \tag{1}$$

where $m_e$ is effective mass participating in the inelastic first mode of structural vibration. Then, and according to Fig. 2, the design seismic base shear force is equal to [1]:

$$V_B = K_e \cdot \Delta_d, \tag{2}$$

Stiffness of predefined critical regions (plastic hinges) is determined directly through the performance limit criteria (displacement ductility), which is further on combined with capacity design procedures to ensure formation of plastic hinges at predefined locations thus preventing formation of other inelastic deformation modes which could result in brittle failure.

Most of the complexity that exists in DDBD relates to determination of the equivalent SDOF system, determination of the design displacement and development of the displacement response spectrum [3, 6, 7]. Special attention must be paid to proper distribution of seismic base shear force and to the analysis of the structure under the distributed seismic force.

Analysis conducted by the authors on a simple plane frame model shows that the DDBD approach results in a simpler and less rigorous analysis in comparison to analysis procedures defined in the current regulations [1, 7].

## 3 Numerical model

To compare the results, DDBD and multimodal response spectrum (MRS) analyses were carried out on two structures identical by geometry, materials and load applied.

The observed structure is a three-bay, 16-storey façade RC frame, taken from an office building regular in plan and elevation. The office building observed has defined elevator-shaft core walls that serve as a lateral force resisting system in addition to the façade frames. Moreover, the building consists of flat post-tensioned concrete slabs and interior columns that carry only gravitational load. It is assumed that the building is fixed at the ground level. The basement is not included in the analytical model.

The numerical model used to run MRS analysis was fully designed according to Eurocode 8 [7].

## 3.1 Geometry and materials

Height of the first floor is equal to 4,50 m, while the height of all other floors is equal to 3,50 m. The depth of the post-tensioned slabs at each floor is $d$ = 24 cm. Layout of the building is shown by Fig. 3.

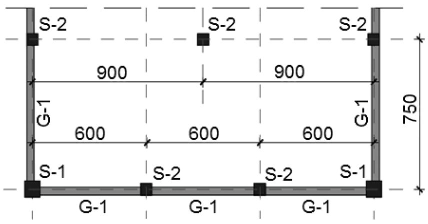


**Figure 3** Layout of the structural system

Beams, slabs and columns are made of normal weight ($\rho_c$ = 2500 kg/m$^3$) concrete of class C 25/30, C 30/37 and C 40/50 respectively (Tab. 1). All structural elements were reinforced with B500C steel bars (Tab. 2).

**Table 1** Structural member geometry and concrete characteristics

| Column | Floor | Location | $b_c/h_c$ (cm) | Concrete | | $\gamma_c$ |
|---|---|---|---|---|---|---|
| | | | | Class | $E_{cm}$ (MPa) | |
| S-1 | 1-8 | outer | 80/80 | C40/50 | 35000 | 1,50 |
| S-2 | 1-8 | inner | 80/60 | C40/50 | 35000 | 1,50 |
| S-3 | 9-16 | out./in. | 80/40 | C40/50 | 35000 | 1,50 |
| Beams | Floor | $L_b$ (cm) | $b_b/h_b$ (cm) | Concrete | | $\gamma_c$ |
| | | | | Class | $E_{cm}$ (MPa) | |
| G-1 | 1-16 | 600 | 40/100 | C25/30 | 30500 | 0,77 |

**Table 2** Reinforcing steel characteristics

| Steel | $f_{yk}$ / MPa | $f_u$ / MPa | $E_s$ / MPa | $\gamma_s$ |
|---|---|---|---|---|
| B500C | 500 | 540 | 200000 | 1,15 |

Symbols used in Tab. 1 and 2 are as follows: $b_c$ width of column cross section, $h_c$ height of columns cross section, $E_{cm}$ secant modulus of elasticity of concrete, $\gamma_c$ the partial factor for concrete, $L_b$ beam span length, $b_b$ the width of beam cross sections, $h_b$ height of beam cross sections, $f_{yk}$ the characteristic yield strength of reinforcement, $f_u$ tensile strength of reinforcement, $E_s$ the modulus of elasticity of reinforcing steel, $\gamma_s$ partial factor for steel.

The partial safety factors for materials for ultimate limit state were adopted according to European codes [8].

All structural elements were modelled with a reduced stiffness to take into account the effect of cracking in concrete. The reduction taken into account is 50 % of gross section properties for bending and shear; according to [7].

## 3.2 Dead and imposed loads

The vertical load includes both dead and imposed load. Self-weight of beams and columns is taken directly through SAP2000 [9], the conventional software for the static/dynamic analysis of structures. Floor slabs are not directly applied to the model, rather they are applied through additional dead load as $G_{FS}$ = 6,0 kN/m$^2$. Similarly, the beams perpendicular to the observed frame were applied to the model as dead load $G_{BP}$ = 142,5 kN along with the live load acting on them $Q_{BP}$ = 36,5 kN. At each floor, additional dead load representing floor layers and partition walls taken as $G_{FLW}$ = 2,5 kN/m$^2$ is applied. Moreover, additional dead load representing roof layers $G_{RL}$ = 4,0 kN/m$^2$ was added to roof.

Imposed load at each floor is taken as $Q_F$ = 3,0 kN/m$^2$, while at roof it is taken as $Q_R$ = 0,75 kN/m$^2$.

All of the loads applied are in accordance with [10].

## 4 Earthquake action

Earthquake demand is defined all in accordance with [7] and for a return period of 475 years. According to [7] in most cases seismic hazard is shown only by one factor called ground acceleration. Due to the fact that the model is planar only one horizontal component of earthquake action is used (horizontal in frame plane). For regular buildings which satisfy provisions according to [7] the vertical component of earthquake action can be neglected and thus the one is not applied to the model. It is assumed that the observed building is located in seismic zone IX on the soil of category D (very soft soil) [7]. Type 1 spectrum is used [7].

DDBD procedure has the advantage to show the influence of ductility on seismic demand to which the structure is exposed because the requested ductility is well known from the very start of analysis, and it can be observed independent of hysteretic characteristics. Due to the fact that ductility is defined as a measure of deformation, the DDBD method requires the use of displacement response spectrum. Thus, the method is direct and allows one to generate the displacement response spectrum for different damping levels and for different earthquake intensities (Fig. 2).

Displacement response spectrum can be generated directly from an existing acceleration response spectrum, assuming steady-state sinusoidal response (increasingly inaccurate at long periods) [7]:

$$S_D = \frac{T^2}{4 \cdot \pi^2} \cdot S_e \cdot g. \tag{3}$$

It should be noted that the corner period $T_D$ is assumed to be 5,0 s in obedience to the more up-to-date information provided in recent work [11]. In fact, using selected sets of high-quality digital strong motion data from different world regions it has been highlighted how the salient features of displacement response spectra in the long-period range (up to 10 s period) are essentially the function of magnitude, source distance, and site conditions [11 ÷ 14].

Now we have the elastic acceleration response spectrum and the corresponding displacement response spectrum defined, which can be seen in Fig. 4.

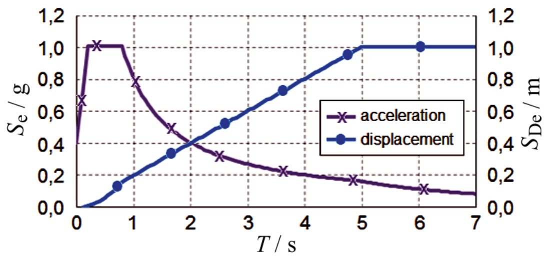


**Figure 4** Elastic acceleration and displacement response spectrum

The inertial effects of the design seismic action are taken into account by calculating the masses associated with all of the dead loads $G_{tot}$ and all of the floor imposed loads (w/o roof imposed loads) $Q_{F,tot}$ as described in [15] and [7]. The following combination of actions is used to calculate effective mass of structure:

$$1{,}00 \cdot G_{tot} + 0{,}24 \cdot Q_{F,tot} . \tag{4}$$

Multimodal spectral analysis was carried out with behavior factor $q$ equal to 3,9, calculated in accordance with [7].

## 5 Equivalent SDOF system and calculation of seismic base shear

Once the performance limit state (ULS or SLS) is chosen to be used while defining the maximum design displacement, which is the key parameter, we can start forming an equivalent SDOF system [1, 3]. Return period of 475 years, defined earlier in this paper, yields the use of ULS.

It is assumed that the here observed building is of importance class III and it is assumed that the building is w/o non-structural elements, thus the design interstorey drift $\Delta_{ri}$ is calculated using expression [7, 16]:

$$\Delta_{ri} \cdot \nu \le 0{,}010 \cdot h_i , \tag{5}$$

where $\nu$ is the reduction factor which takes into account the lower return period of the seismic action associated with the damage limitation requirement and $h_i$ is the height of the $i$-th floor. According to [7] and [16] the recommended value of $\nu$ is 0,40 for importance class of building observed here. Consequently maximum design interstory drift is calculated using expression:

$$\Delta_{ri,\max} = 0{,}025 \cdot h_i . \tag{6}$$

At this point it is important to note that the well-known and conservative approach will not be used fully through the DDBD procedure [15]:

$$E_d \le \frac{R_k}{\gamma_M} . \tag{7}$$

Rather, in DDBD and when designing plastic hinges it is recommended to multiply (NOT divide) the characteristic strengths of concrete and reinforcement steel with partial factors of 1,3 and 1,1 respectively [1]. This is so for the seismic design situation because we expect inelastic response of a structure [1, 13].

**Table 3** SDOF system parameters

| Floor | $H_i$ (m) | $m_i$ (t) | $Ø_i$ | $\Delta_i$ |
|---|---|---|---|---|
| 16 | 57,0 | 127,92 | 1,00 | 1,09 |
| 15 | 53,5 | 133,38 | 0,96 | 1,04 |
| 14 | 50,0 | 133,38 | 0,91 | 1,00 |
| 13 | 46,5 | 133,38 | 0,87 | 0,94 |
| 12 | 43,0 | 133,38 | 0,82 | 0,89 |
| 11 | 39,5 | 133,38 | 0,76 | 0,83 |
| 10 | 36,0 | 133,38 | 0,71 | 0,77 |
| 9 | 32,5 | 133,38 | 0,65 | 0,71 |
| 8 | 29,0 | 137,30 | 0,59 | 0,65 |
| 7 | 25,5 | 141,24 | 0,53 | 0,58 |
| 6 | 22,0 | 141,24 | 0,46 | 0,51 |
| 5 | 18,5 | 141,24 | 0,40 | 0,43 |
| 4 | 15,0 | 141,24 | 0,33 | 0,36 |
| 3 | 11,5 | 141,24 | 0,26 | 0,28 |
| 2 | 8,0 | 141,24 | 0,18 | 0,20 |
| 1 | 4,5 | 155,24 | 0,10 | 0,11 |

When forming an equivalent SDOF model, only the first mode of structural vibrations will be used. As in force methods, higher modes can have important effects on internal forces as well, especially for structural members which need to remain elastic [1, 17]. It is shown that higher modes also have a great influence on floors displacements and thus interstorey drifts. To account for the higher mode effects, a displacement reduction factor is used and calculated as defined in [1, 13]:

$$\omega_\theta = 1{,}15 - 0{,}0034 \cdot H \le 1 , \tag{8}$$

where $H$ is the height of observed building measured in m, here taken as 57 m yields $\omega_\theta$ = 0,9562. Maximum design displacement is calculated as [1]:

$$\Delta_d = \sum_{i=1}^{n} (m_i \cdot \Delta_i^2) / \sum_{i=1}^{n} (m_i \cdot \Delta_i) , \tag{9}$$

where $m_i$ is the $i$-th floor mass and $\Delta_i$ is displacement of the $i$-th floor. The displacement $\Delta_1$ is "critical" floor, i.e. first floor beam displacement dependant. By knowing the first floor displacement all other floor displacements can be determined using the following expression (Tab. 3) [1]:

$$\Delta_i = \phi_i \cdot \frac{\Delta_{r1,max}}{\theta_1} , \tag{10}$$

where $\phi_i$ is the inelastic mode shape of the $i$-th floor, $\Delta_{r1,\max}$ is the first floor maximum design displacement and $\theta_1$ is the inelastic mode shape of the first floor. Inelastic mode shape $\phi_i$ for structures with more than 4 floors can be calculated by using the following expression (Tab. 3) [1]:

$$\phi_i = \frac{3}{4} \cdot \frac{H_i}{H} \cdot \left(1 - \frac{H_i}{4 \cdot H}\right), \tag{11}$$

where $H_i$ is the height of the $i$-th floor measured from the buildings base. It is important to say that the expression (11) is approximate but it gives satisfactory results [1, 13].

Effective SDOF mass now can be calculated using the following expression [1, 3]:

$$m_\mathrm{e} = \sum_{i=1}^{n} (m_i \cdot \Delta_i^2) / \Delta_\mathrm{d}, \tag{12}$$

where $m_i$ is the $i$-th floor mass and the effective height of the SDOF system is [1, 3]:

$$H_\mathrm{e} = \sum_{i=1}^{n} (m_i \cdot \Delta_i \cdot H_i) / \sum_{i=1}^{n} (m_i \cdot \Delta_i) . \tag{13}$$

To be able to calculate the effective displacement at the occurrence of first yield $\Delta_\mathrm{y}$ for the equivalent SDOF system, beside the effective SDOF height, one needs to define the interstorey drift at the occurrence of first yield which is defined as [1]:

$$\theta_\mathrm{y} = 0{,}5 \cdot \varepsilon_\mathrm{y} \cdot \frac{L_\mathrm{b}}{h_\mathrm{b}} , \tag{14}$$

where $\varepsilon_\mathrm{y}$ is the yield strain of reinforcement steel calculated using expression [16, 8]:

$$\varepsilon_\mathrm{y} = \frac{f_\mathrm{yd}}{E_\mathrm{s}} . \tag{15}$$

Finally and to be able to determine the ductility demand of the system, effective displacement at the occurrence of first yield can be determined using the following expression [1]:

$$\Delta_\mathrm{y} = \theta_\mathrm{y} \cdot H_\mathrm{e} , \tag{16}$$

where it is accurate enough to assume a linear distribution of yielding per floor.

The next step is to define the effective damping of the equivalent system, but firstly and to be able to do that, ductility demand coefficient needs to be calculated [1, 17]:

$$\mu = \frac{\Delta_\mathrm{d}}{\Delta_\mathrm{y}} . \tag{17}$$

Since the ductility of the system is known, the effect of inelastic structural response can be taken into account by the equivalent damping of SDOF system for which the design displacement response spectrum will be calculated [1, 2]. The equivalent damping for RC frame structures and a defined ductility is calculated as [1]:

$$\xi_\mathrm{eq} = 0{,}05 + 0{,}565 \cdot \frac{\mu - 1}{\mu \cdot \pi}, \tag{18}$$

and is valid only for a damping ratio of 5 %. For other damping values other expressions need to be determined. Inelastic structural behaviour has a great effect on the response of the structure. This effect is taken into account by design displacement response spectrum (Fig. 6) obtained by modifying the elastic displacement response spectrum by the damping correction factor calculated as [1]:

$$R_\xi = \left(\frac{0{,}07}{0{,}02 + \xi}\right)^\alpha , \tag{19}$$

where $\alpha$ is 0,50 and 0,25 for normal and velocity pulse conditions respectively. The authors assumed normal conditions, thus $\alpha$ = 0,50. It is important to note that there are still doubts whether the expression (19) is appropriate or not [1].

Finally, input parameters of the DDBD procedure calculated using expressions (8), (9) and (12) ÷ (19) are given in Tab. 4.

**Table 4** DDBD card

| Parameter | | Value |
|---|---|---|
| Maximum design displacement, m | $\Delta_\mathrm{d}$ | 0,748 |
| Displacement reduction factor, – | $\omega_\theta$ | 0,9562 |
| Effective SDOF mass, t | $m_\mathrm{e}$ | 1794,37 |
| Effective SDOF height, m | $H_\mathrm{e}$ | 37,80 |
| Interstory drift at the occurrence of first yielding, – | $\theta_\mathrm{y}$ | 0,00825 |
| Relative deformation of reinf. steel at yielding, – | $\varepsilon_\mathrm{y}$ | 0,0275 |
| Displacement at the occurrence of first yielding, m | $\Delta_\mathrm{y}$ | 0,310 |
| Ductility demand coefficient, – | $\mu$ | 2,39 |
| Equivalent damping, % | $\xi_\mathrm{eq}$ | 15,5 |
| Damping correction factor, – | $R_\xi$ | 0,633 |

After all, the effective SDOF period $T_\mathrm{e}$ needs to be determined, which poses a problem because the design displacement response spectrum at 15,5 % of damping and the maximum design displacement do not have a matching value.

**Table 5** Iteration to final maximum design displacement

| | Start | Iteration 1 | Iteration 2 | Final |
|---|---|---|---|---|
| $\Delta_\mathrm{d}$ /m | 0,748 | 0,637 | 0,648 | 0,659 |
| $M$ / – | 2,39 | 2,22 | 2,08 | 2,11 |
| $\xi_\mathrm{eq}$ /% | 0,155 | 0,149 | 0,143 | 0,144 |
| $R_\xi$ / – | 0,633 | 0,644 | 0,655 | 0,652 |

The value of the maximum design displacement needs to be iterated until we have an actual displacement that the structure would achieve under the defined seismic action. Only one condition needs to be satisfied in order for the structure to respond inelastically, and that is that the yield displacement is less than the peak displacement of 5 % damped response spectrum. In this case inelastic response will occur, but not at the level of ductility corresponding to the displacement or drift capacity of the structure (Tab. 4). The final displacement will be somewhere between $\Delta_{D,15,5}$ and $\Delta_d$. First, one should assume the value of maximum displacement and then iterate values until the solution stabilizes (Tab. 5).

The value of inelastic SDOF system effective period $T_e$ can be determined using expression [1]:

$$T_e = T_D \cdot \frac{\Delta_d}{\Delta_{D,5}} \cdot \left( \frac{0{,}02 + \xi_{eq}}{0{,}07} \right)^{\alpha}, \tag{20}$$

where $\Delta_{D,5}$ is peak displacement of the displacement response spectrum at 5 % damping. The value $\Delta_{D,5}$ read from Fig. 5 is equal to 1,006385, thus the $T_e$ is equal to 5,00 s. By using expression (1) effective secant stiffness is calculated equal to 2833,55 kN/m while I. order base shear force is calculated equal to 1869,09 kN by using expression (2).

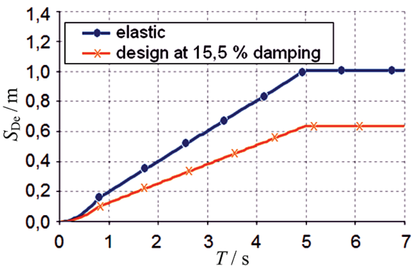


**Figure 5** Elastic and design displacement response spectrums

The next step in the analysis procedure is to check whether second order effects are significant or not, and whether those are needed to be taken into account.

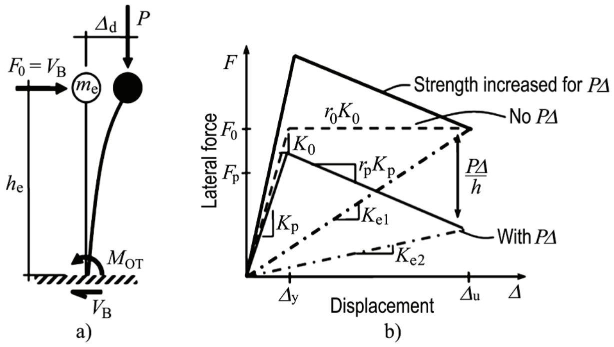


**Figure 6** P-delta effect [13]

The $P$-$\Delta$ effect is significant in aseismic design due to horizontal mass dislocation that gives additional forces to the structure. In DDBD, unlike in the force based methods, maximum design displacements are well known from the very beginning and after all, structures are designed to achieve these displacements. The following step is to calculate the stability index $\theta_\Delta$ and if it exceeds the value of 10 %, then the $P$-$\Delta$ effects are of great significance and the design stiffness at maximum displacement $K_{e2}$ (Fig. 6) needs to be taken into account, [1]:

$$\theta_\Delta = \frac{P_{tot} \cdot \Delta_d}{M_{OT}}, \tag{21}$$

where $P_{tot}$ is the total vertical force (sum of all floor masses multiplied by gravity acceleration) and $M_{OT}$ is the overturning moment.

If P-Δ effects are significant one must calculate the II order seismic base shear force is calculated using expression [1]:

$$V_{B,II} = K_e \cdot \Delta_d + C \cdot \frac{P \cdot \Delta_d}{H_e}, \tag{22}$$

where $C$ is material dependant coefficient equal to 0,5 for reinforced concrete structures.

As can be seen from Tab. 6, stability index is almost twice the limit value at which the $P$-$\Delta$ effect needs to be taken into account, thus $V_{B,II}$ = 2055,31 kN and is relevant for further analysis (Fig. 7).

**Table 6** $P$-$\Delta$ parameters for DDBD

| Parameter | | Value |
|---|---|---|
| Total axial force /kN | $P_{tot}$ | 21 597,30 |
| Overturning moment /kNm | $M_{OT}$ | 74 167,99 |
| Stability index, – | $\theta_\Delta$ | 0,192 |

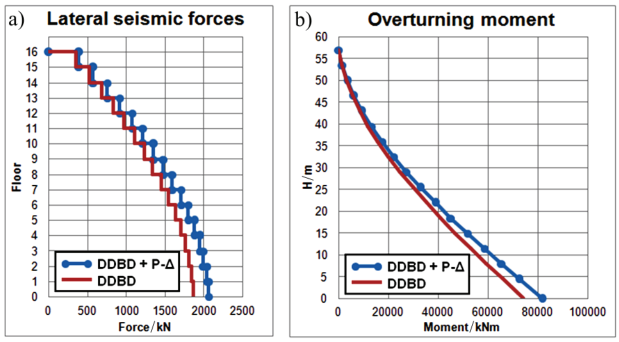


**Figure 7** a) Lateral seismic forces diagram;
b) Overturning moment diagram

## 6 Forming of MDOF model

Providing a good model of the structure is a key step in the analysis. To assure this, element stiffness needs to be estimated and modelled in a way the MDOF model (Fig. 8) behaviour matches the behaviour of the equivalent SDOF model.

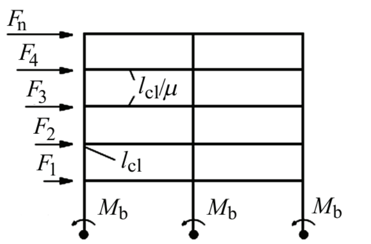


**Figure 8** Frame modelling principle [1] (edited by authors)

Since one frame model is observed, where the weak-beam-strong-column concept is applied through the analysis, the stiffness of beams was determined using expression [1]:

$$I_b = \frac{I_{cr}}{\mu_b}, \tag{23}$$

where $I_{cr}$ is the stiffness of the cracked beam section and $\mu_b$ is the expected ductility demand for beams. To simplify the analysis, expected ductility demand for beams $\mu_b$ can be equal to ductility demand of the whole equivalent SDOF system $\mu$ [1].

By performing capacity design inelastic response of columns was prevented, thus the columns stiffness was taken to be cracked $I_{cr}$ and without the influence of ductility. As the member's real stiffness was not known at the very start of the analysis, according to [7] it is recommended to reduce the stiffness of all members to 50 %. This approximation is satisfactory, because the distribution of internal forces depends on relative member's stiffness, rather than on absolute stiffness.

Another important issue is the proper modelling of 1st floor columns. It is assumed that the plastic hinges in these columns will form only at their bottom. Thus, the most appropriate way of modelling columns is to model plastic hinges at their bottom (pinned supports). To assure full fixity additional moments were applied to simulate the design resistance. It is assumed that the point of contraflexure is at 60 % height of the column, thus it is suggested to have the 1st floor moment to base moment ratio 40 % to 60 % respectively [1, 13]. This way the total moment of fixity is calculated as [1, 13]:

$$\sum M = \sum_{i=1}^{n} F_i \cdot (0{,}6 \cdot h_1) = V_B \cdot (0{,}6 \cdot h_1)\,. \tag{24}$$

It is important to note that the resistance of the critical regions needs to be determined on the basis of seismic action influence, while the influences of seismic and other actions are combined for the elements that need to stay elastic, but all under the assumption that these influences are determined for the same cracked member stiffness so that the compatibility requirements are fulfilled.

## 7 Results and discussion

The results of the DDBD and MRS methods used to analyze the frame structures, both for beams and columns, are presented in the diagrams which are shown in normalized form: the longitudinal reinforcement normalized to the cross section area and the shear reinforcement normalized to stirrup spacing.

Fundamental period of vibration of the frame analyzed and designed using MRS method is equal to 2,12 s, while the base shear calculated for the same frame is equal to 1887,40 kN. When compared to MRS method, parameters used to conduct DDBD method are, respectively, 2,35 and 1,09 times higher.

### 7.1 Beams

By determining the base shear force of the first inelastic mode, the resistance of plastic hinges in bending along with the capacity design shear resistance for beams can easily be determined to prevent brittle failure.

Since the paper discusses a structure with predefined ductility level (DCM), the design of the critical regions is performed by using the following rules [7]:

1) length of critical region:

$$l_{cr} = h_b = 100\,\text{cm}\,,$$

2) minimum area of steel bars needed:

$$\rho_{min} = 0{,}5 \cdot \frac{f_{ctm}}{f_{yk}} = 0{,}0026$$
$$\rightarrow A_{s,min} = 9{,}75\,\text{cm}^2\,. \tag{25}$$

As the bending resistance of the plastic hinges is known, it is possible to perform capacity design with the purpose of preventing brittle failure, i.e. shear resistance is calculated as follows [7]:

$$V_{Ed} = \frac{\sum M_{Rb}}{l_{cl,b}} \pm V_{o,g+\psi 2q}\,, \tag{26}$$

where $V_{Ed}$ is the design shear force, $\Sigma M_{Rb}$ is the sum of the design values of the moments of resistance of the beams framing the joint, $l_{cl,b}$ is clear length of beam and $V_{o,g+\psi 2q}$ is the shear force from quasi-permanent loads. Shear force resistance is conducted in accordance with [8] along with the assumption that concrete is fully cracked and thus neglected in shear resistance calculation. The following expressions are used [8]:

$$A_{sw} = \frac{V_{Ed} \cdot s_w}{z \cdot f_{ywd} \cdot \cot\theta_w}\,, \tag{27}$$

$$V_{Rd,max} = 0{,}3 \cdot \left(1 - \frac{f_{ck}}{250}\right) \cdot b \cdot z \cdot f_{cd} \cdot \sin 2\theta, \tag{28}$$

where $A_{sw}$ is the cross-sectional area of the shear reinforcement, $s_w$ is the spacing of the stirrups, $z$ is lever arm of internal forces, $f_{ywd}$ is design yield of shear reinforcement, $\theta_w$ is the angle between the concrete compression strut and the beam axis perpendicular to the shear force, $f_{ck}$ is characteristic compressive cylinder strength of concrete at 28 days, $f_{cd}$ is design value of concrete compressive strength.

The MRS based design, when compared to DDBD design, generally results in higher amounts of longitudinal reinforcement with the difference ranging from about 19 % for the inner end section of outer bay beams (Fig. 9b), and up to 26,7 % for the outer end section of the same beams (Fig. 9a); and is almost constant with the height of the frame.

Considering the inner bay beams the authors noted 14,7 % less longitudinal reinforcement steel needed for the structure analysed using the DDBD method, when compared to the MRS method. These differences are mainly the consequence of the fact that stiffness of the beams in the DDBD method is additionally reduced to represent the secant stiffness of the whole system at the predefined performance level. Another cause is the fact that gravity load moments in DDBD were neglected in the design of beams.

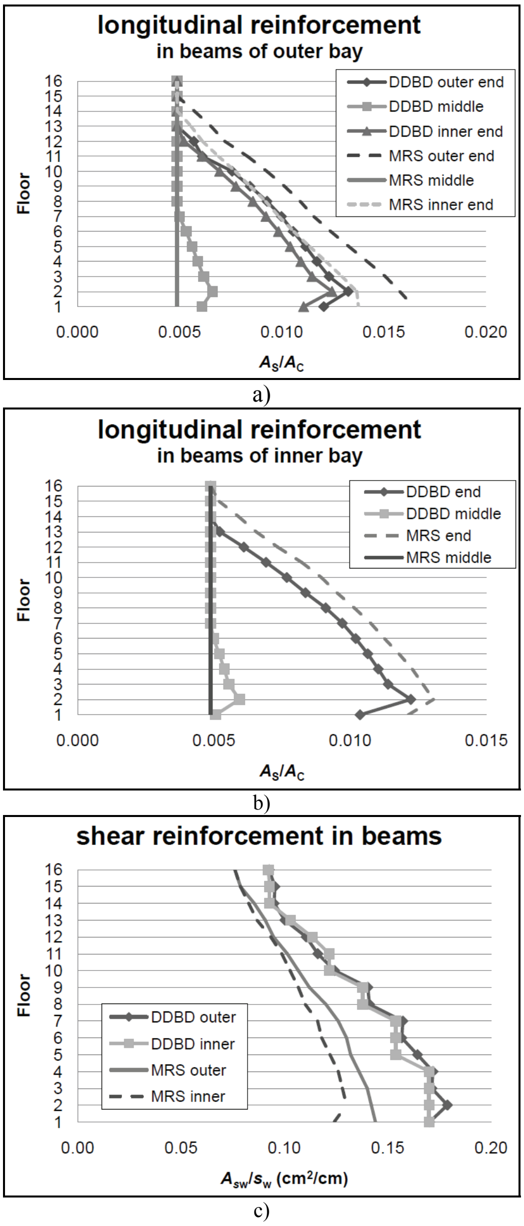


**Figure 9** Normalized amount of required reinforcement by frame floor a) longitudinal reinforcement in beams of outer bay; b) longitudinal reinforcement in beams of inner bay; c) shear reinforcement in beams

Required shear reinforcement based on capacity design calculation is shown in Fig. 9c. The differences in this case are quite significant, showing that the DDBD method requires up to 20 % more shear reinforcement in the outer bay beams and up to 27 % more shear reinforcement in the inner bay beams. This shows that the DDBD method follows capacity design rules. Although the MRS method gives higher amounts of longitudinal reinforcement and capacity design would suggest that shear reinforcement is going to follow that pattern, this is not the case here. The reason behind this is the increased strength of both concrete and steel when calculating design shear forces (capacity design) in the DDBD method.

Cross section of beams with reinforcement bars is shown in Fig. 10.

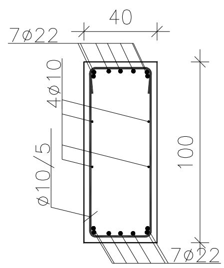


**Figure 10** Cross section of the 2nd floor beam, the beam with the highest amount of required longitudinal reinforcement steel

## 7.2. Columns

Unlike the beams, in the design of columns higher mode effects and possible influence of overstrength in critical regions needs to be taken into account. Higher modes can be combined by using the SRSS rule, with a fundamental difference to the force based design in that the 1st mode is inelastic. There is one more difference to the force based design, higher mode effects are determined by using the elastic response spectrum because higher modes affect only structural members that need to remain elastic. Thus, the expression to calculate the base shear force including higher mode effects reads as follows [1]:

$$V_{\mathrm{B,hme}} = \sqrt{(\gamma_{\mathrm{ov}} \times V_{1,\mathrm{B}})^2 + (V_{2,\mathrm{B}})^2 + \ldots + (V_{n,\mathrm{B}})^2}\,, \qquad (29)$$

where $\gamma_{\mathrm{ov}}$ is overstrength factor and $V_{i,\mathrm{B}}$ to $V_{n,\mathrm{B}}$ are base shear forces of the first inelastic mode, and of the rest of elastic modes denoted by $n$.

Overstrength factor represents the increase in resistance of critical regions and their effects on elastic structural parts, [17]. In accordance with [7] the overstrength factor is taken to be 1,30.

For analysis provided here modes 2 to 5 are to satisfy the code requirement of 90 % participating mass, although lesser modes would be enough as well. SRSS combination of first five modes was made and shown by Fig. 11.

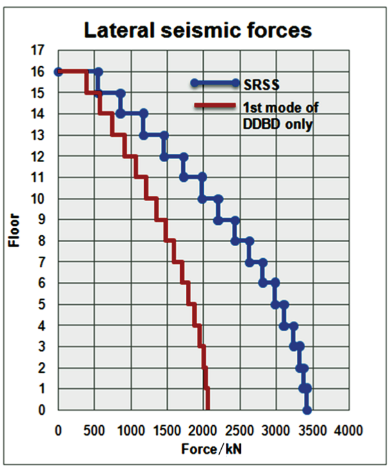


**Figure 11** Comparison of the SRSS combination for first 5 modes and the 1st mode of DDBD lateral seismic forces

For the force distribution shown in Fig. 11, bending resistance of columns can be calculated taking the axial force influence into account. In addition, first floor columns are designed to assure plastic hinge formation at the base. In the same way higher mode effects are taken into consideration when calculating axial force, but with a difference when designing 1st floor columns where only quasi-permanent axial force is taken into account. Namely, during the earthquake, outer columns will resist most of the overturning moment, thus one of the outer columns will be in compression while the other one will be in tension. If one would design both of these columns for tension, the final resistance of those columns would be much higher than the seismic demand, thus the formation of plastic hinges would not be possible.

The following rules defined in [7] were followed when designing critical regions in columns:

1) length of critical region:

$$l_{\mathrm{cr}} = \min\left\{ h_{\mathrm{c}}; b_{\mathrm{c}}; \frac{l_{\mathrm{c}}}{5}; 450\,\mathrm{mm} \right\}, \tag{30}$$

2) minimum area of steel bars needed:

$$A_{\mathrm{s,min}} = 0{,}10 \cdot A_{\mathrm{c}}\ , \tag{31}$$

where $l_{\mathrm{c}}$ is the length of the column and $A_{\mathrm{c}}$ is the area of concrete element cross-section.

Increase in bending resistance due to actual provided reinforcement is an important feature to take into account when designing a structure according to the capacity design procedure. Such an increase is referred to as overstrength. This is important because these moment capacities $M_{\mathrm{Rc}}$ are the demands according to which shear force levels are determined, and if not taken into account could result in brittle failure.

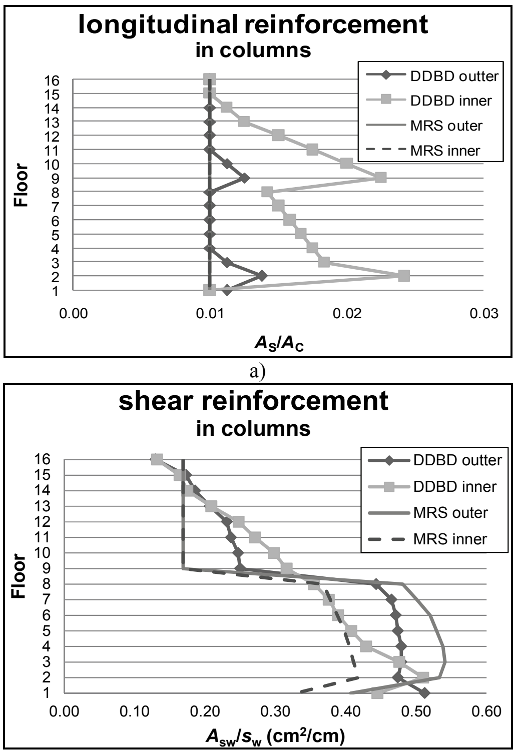


**Figure 12** Normalized amount of required reinforcement in columns by frame floor: a) longitudinal reinforcement; b) shear reinforcement

The results for the columns are as expected because the demand was more or less clear just by looking at the base shear force, which in the case of the DDBD method is almost twice than in the MRS method according to [7].

Reinforcement ratios of the outer columns do not differ as much as in the inner columns (Fig. 12a). The difference in the required longitudinal reinforcement calculated is visible only at few stories with a maximum of 27 % for the DDBD method. The required longitudinal reinforcement of inner columns, however, shows significant differences up to the whole height with a maximum of 59 % (Fig. 12a).

The design shear reinforcement shows much more different variations (Fig. 12b). When compared, the amount of required shear reinforcement calculated by DDBD and MRS method, the outer columns require more reinforcement from the MRS method at the middle part of the structure, and this due to the fact that MRS benefits from larger axial forces when determining the design moment resistance and hence the design shear forces. But what is more important the DDBD ensures about 21 % more shear reinforcing at the base where plastic hinging is expected. The inner columns benefit from the DDBD through the whole height of the structure with differences up to 53 %, which is not strange because the DDBD takes into account the fact that higher modes affect the displacements at the upper part of the structure [1], whereas in the case of the MRS method some minimum shear reinforcing is adopted in the upper stories.

Cross sections of columns with reinforcement bars are shown in Fig. 13.

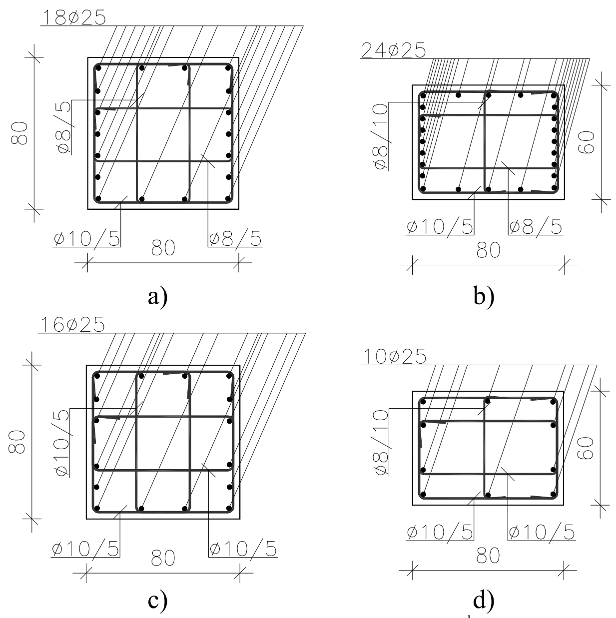


**Figure 13** Cross sections of: a) outer column at 2nd floor; b) inner column at 2 nd floor; c) outer column at 1st floor; d) inner column at 1st floor

## 8 Conclusion

In this paper the theory of DDBD is explained and shown in detail along with the following example, while its advantages over the traditional force-based design procedure are also highlighted. The DDBD is defined in great measure and well accepted in science, but the traditional and well accepted force based method found its place in practical engineering. It offers significant control over the analysis and design. Unlike the force based method, in DDBD the limit states are not checked, rather those are used as an input data.

Furthermore, the ductility, as one of the basic and most important parameters in seismic design, through DDBD comes to its full application in a way that it is directly used in analysis rather than being used indirectly by behaviour factors. The example structure proves this to be right, while also showing that the strength is not the key parameter in seismic design. With this in mind, it is shown that the DDBD is more straightforward and economical than the force based design in regards to highlighting one of the key aspects in seismic design: the weak beam-strong column principle. With higher amounts of longitudinal reinforcement in columns, and shear reinforcement in beams and some columns the DDBD prevents brittle failure and ensures that intended elements remain elastic. In conclusion, the total amount of reinforcement might not vary considerably, but it is "smarter" spent in the right places.

Although the DDBD has been tested for various structural types by many dynamic analyses, the authors suggest that additional analyses be carried out for various structures in the sense of testing the method and sorting out any problems for use in practice.

Definition of seismic excitation is one of the biggest drawbacks of this method, but a great number of high quality digital accelerograms recorded till today partially solved this problem. Although the procedure is very simple and results can be obtained faster in comparison to force methods, development of future computer algorithms is the key to the application of the DDBD method in practical engineering.

It is believed that in the next ten years this method will be accepted in whole and that it will push aside the current regulations by its simplicity and advantages.

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#### Authors' addresses


***Damir Džakić, mag. ing. aedif.***
J. J. Strossmayer University of Osijek
Faculty of Civil Engineering Osijek
Crkvena 21, 31000 Osijek, Croatia
E-mail: damirdzakic@hotmail.com

***Ivan Kraus, mag. ing. aedif.***
J. J. Strossmayer University of Osijek
Faculty of Civil Engineering Osijek
Crkvena 21, 31000 Osijek, Croatia
E-mail: ikraus@gfos.hr

***Prof. dr. sc. Dragan Morić, dipl. ing. građ.***
J. J. Strossmayer University of Osijek
Faculty of Civil Engineering Osijek
Crkvena 21, 31000 Osijek, Croatia
E-mail: dmoric@gfos.hr