

# **SEISMIC YIELD DISPLACEMENTS OF COMPOSITE STEEL/CONCRETE PLANE FRAMES**

**Konstantinos Skalomenos**

Dr. Civil Engineer

Department of Civil Engineering, University of Patras,  
Patras, Greece

e-mail: skalomenos@upatras.gr

**George Hatzigeorgiou**

Associate Professor

School of Science and Technology, Hellenic Open University,  
Patras, Greece

e-mail: hatzigeorgiou@eap.gr

**Dimitri Beskos**

Professor

Department of Civil Engineering, University of Patras,  
Patras, Greece

e-mail: beskos@upatras.gr

## **1. ABSTRACT**

Empirical expressions for the estimation of the approximate lateral displacements at first yielding of plane composite steel-concrete moment-resisting frames (MRFs) consisting of I steel beams and concrete filled tube (CFT) columns under seismic excitations, are provided. The approximate expressions are proposed for use in a displacement based seismic design (DDBD) of these CFT-MRFs structures. These expressions, which are functions of the geometrical and design properties of the frames, are derived on the basis of seismic response data obtained with the aid of extensive dynamic inelastic analyses involving 96 moment resisting plane CFT-MRFs with steel grade S275 and S355 and concrete strength C20 and C40 under 100 ordinary seismic ground motions. The DDBD method, using the proposed formulae, is applied to two new CFT-MRFs and comparisons with the base shears and drifts derived from inelastic dynamic analyses with ten acceleration time histories, compatible to EC8 spectrum, are conducted.

## **2. INTRODUCTION**

In recent years, there has been a great tendency toward performance-based seismic design of structures. In this connection, various methods have been developed among which the Direct Displacement-Based Design (DDBD). The method defines the design performance level of the structure in terms of displacement limits. Therefore, displacement is the key parameter of the design method. Since damage is directly related to displacements, seismic design methods based on displacements in a direct or indirect manner, like the DDBD method «[1], [2]» has the advantage over force-based methods «[3]» of an easy and direct

damage control. The DDBD method, which is briefly presented in the next section, during the course of its application requires an estimate of lateral displacement profile at first yielding. The composite moment resisting frames having CFT columns and steel girders (CFT-MRFs) are a new type of structures which offers significant advantages for use as the primary resistance systems in building structures subjected to seismic loading. The CFT-MRFs exhibit desirable features, such as, large energy dissipation and increased strength and stiffness and for these reasons have become increasingly popular in mid-rise and high-rise buildings. This paper provides simple empirical expressions for the estimation of yield displacements for CFT-MRFs. These formulae are expressed in terms of geometrical parameters and design characteristics of these frames and are derived on the basis of dynamic inelastic analyses of 96 moment resisting frames with steel grade S275 and S355 and concrete strength C20 and C40 under 100 acceleration time histories. The DDBD method, using the proposed formulae, is applied to two new CFT-MRFs and comparisons with the base shears and drifts derived from inelastic dynamic analyses with ten acceleration time histories, compatible to EC8 spectrum, are conducted. In addition, comparisons with the existing formulae in the literature for yield displacement of plane all steel MRFs are presented.

### 3. BASIC STEPS OF DIRECT DISPLACEMENT BASED SEISMIC DESIGN

This section briefly describes the basic steps of the DDBD procedure for multi degree of freedom (MDOF) framed building structures in order to create the proper setting for discussing the estimation of the yield displacements needed in that design procedure. The first stage of the design process is the representation of the MDOF frame by an equivalent single degree of freedom (SDOF) frame modelling the first inelastic mode of response. Consider a multi-bay, multi-storey plane frame with diaphragm action at each floor level subjected to lateral seismic load and vertical dead plus live load. This frame can be modelled as a MDOF system with one concentrated mass  $m_i$  per every floor  $i$  and its associated lateral displacement (degree-of-freedom)  $\Delta_i$ . This  $n$  degree-of-freedom system is replaced by an equivalent SDOF with mass  $m_e$ , stiffness  $K_e$ , viscous damping  $\xi_e$  and displacement  $\Delta_e$ , where the subscript “e” stands for equivalent. When these equivalent system properties have been determined, the design base shear  $V_b$  for the substitute structure can be estimated. The base shear is then distributed between the mass elements of the real structure as inertia forces, and the structure analyzed under these forces to determine the design moments at locations of potential plastic hinges. The main relationships from DDBD procedure affected by yield displacements are only presented here due to space limitations. The whole DDBD procedure one can be found in «[1], [2]». Thus, the design floor displacements  $\Delta_i$  of the frame are related to a normalized inelastic mode shape  $\delta_i$ , where  $i = 1$  to  $n$  are the storeys, and to the displacement  $\Delta_c$  of the critical storey by the relationship

$$\Delta_i = \delta_i \cdot \left( \frac{\Delta_c}{\delta_c} \right) \quad (1)$$

where the normalized inelastic mode shape  $\delta_i$ , depends on the height,  $H_i$ , and roof height,  $H_n$ , according to the relationships

$$\delta_i = \left( \frac{H_i}{H_n} \right) \quad \text{for } n \leq 4 \quad (2)$$

$$\delta_i = \frac{4}{3} \cdot \left( \frac{H_i}{H_n} \right) \cdot \left( 1 - \frac{H_i}{4 \cdot H_n} \right) \quad \text{for } n > 4 \quad (3)$$

where  $\Delta_c$  and  $\delta_c$  are obtained in terms of the design interstorey drift ratio (IDR) and relation (2) or (3), respectively, at the critical floor (usually the first one). The equivalent design displacement  $\Delta_{e,d}$  is related to the storey displacements  $\Delta_i$  by the relationship

$$\Delta_{e,d} = \frac{\sum_{i=1}^n m_i \cdot \Delta_i^2}{\sum_{i=1}^n m_i \cdot \Delta_i} \quad (4)$$

where  $m_i$  is the mass at height  $H_i$  associated with displacement  $\Delta_i$ . Furthermore, the SDOF design displacement ductility  $\mu_e$  factor is computed as

$$\mu_e = \frac{\Delta_{\text{eff},d}}{\Delta_{\text{eff},y}} \quad (5)$$

where  $\Delta_{e,y}$  is the equivalent yield displacement and is calculated by replacing in Eq. (4) the displacement  $\Delta_i$  by the yield displacement  $\Delta_{y,i}$ . This  $\Delta_{y,i}$  is obtained by expressions like those developed in this work.

#### 4. CFT-MRFs AND GROUND MOTIONS USED IN THIS STUDY

In order to cover a wide range of structural characteristics of CFT-MRFs, a family of 96 plane regular CFT-MRFs are employed for the parametric studies of this work. These frames have storey heights and bay widths equal to 3 m and 5 m, respectively and columns of square concrete filled steel tube (CFT) sections. Moreover, the frames have the following structural characteristics: number of stories,  $n_s$ , with values 1, 3, 6, 9, 12, 15, 18 and 20, and three bays,  $n_b$ , steel yielding strength ratio  $e_s = 235/f_s$  with the yielding stress  $f_s$  taking the values of 275 and 355 MPa, concrete strength ratio  $e_c = 20/f_c$  with the compressive strength  $f_c$  taking the values 20 and 40 MPa (upper and lower limit for dissipative zones according to EC8 «[3]»), the beam-to-column stiffness ratio,  $\rho$  (calculated for the storey closest to the mid-height of the frame) and column to beam strength ratio,  $\alpha$  (taking various values within practical limits) defined as

$$\rho = \frac{\sum (I/l)_b}{\sum (I/l)_c}, \quad \alpha = \frac{M_{RC,1,av}}{M_{RB,av}} \quad (6)$$

where  $I$  and  $l$  are the second moment of inertia and length of the steel member (column  $c$  or beam  $b$ ), respectively,  $M_{RC,1,av}$  is the average of the plastic moments of resistance of the columns of the first storey and  $M_{RB,av}$  is the average of the plastic moments of resistance of the beams of all the stories of the frame. Every frame was first designed for vertical static load according to EC3 «[4]» and EC4 «[5]» and then checked for seismic load according to EC8 «[3]» for PGA = 0.36g, soil type B and Spectrum Type 1 with behavior factor  $q = 4$ . In addition to the satisfaction of the seismic strength demands in members, other seismic design checks included compliance with stability and drift criteria as well as capacity design considerations «[3], [4], [5]». Then, an ensemble of 100 ordinary (far-field type) ground motions of soil type B and with an average spectrum as close as possible to the EC8 «[3]» elastic spectrum for ground acceleration 0.36 g are selected (without any scaling) and are employed for the nonlinear time history analyses of this study. A full list of all these ground motions and frames with their characteristics can be found in Skalomenos «[6]».

#### 5. RESPONSE DATA AND PROPOSED EXPRESSIONS

The 96 CFT-MRFs mentioned in the previous section, are subjected to a set of 100 accelerograms and their response to those motions at first yielding are determined through

inelastic dynamic analyses with the aid of the computer program RUAUMOKO «[7]». Diaphragm action is assumed at every floor due to the presence of the slab, the effect of large deformations is taken into account and Rayleigh damping corresponding to 3% of the critical damping of the first and the  $i^{\text{th}}$  modes is considered in the analysis, where  $i$  is the number of stories. The inelastic behavior of all the frame members are modeled by means of hysteretic point plastic hinges. The effect of panel zones is taken into account and the connections are assumed to be rigid. The analytical models of frame components utilized here are presented in detail in Skalomenos «[6]».

The response of each frame at first yielding to each accelerogram is obtained. The occurrence of the first plastic hinge in a CFT-MRF, which always happens in beams because of the capacity design, is defined as the state of first yielding. The occurrence of the first plastic hinge can be easily observed with the aid of the ductility ratio  $\phi/\phi_y$ , with  $\phi$  denoting curvature, which is given at the end of each run in the output file of RUAUMOKO «[7]».

By analyzing the response databank for the CFT-MRFs, the effect of the structural characteristics of the frames on their floor yield displacements is identified and the expressions

$$\Delta_{y,i} = a_1 \cdot n_s^{a_2} \cdot \rho^{a_3} \cdot i^{a_4} \cdot \alpha^{a_5} \cdot e_s^{a_6} \cdot e_c^{a_7} \quad (7)$$

$$\Delta_{y,i} = h_i^{b_1} \cdot \left( \frac{h_i}{H} \right)^{b_2} \cdot n_s^{b_3} \cdot e_s^{b_4} \cdot e_c^{b_5} \quad (8)$$

are selected as good candidates for approximating the response databank with  $i$  being the  $i^{\text{th}}$  floor. The grade of steel and concrete strength have been included in the Eq. (7) in the parameters  $e_s = 235/f_s$  and  $e_c = 20/f_c$  together with the frame characteristics  $\rho$  and  $\alpha$ . Furthermore, a simpler expression not depending on  $\rho$  and  $\alpha$  (the frame has not been designed as yet) for the floor yield displacements of CFT-MRFs is also proposed in the form of Eq. (8), where  $h_i$  denotes the height of floor  $i$  and  $H$  is the total height of the frame. A nonlinear regression analysis (Levenberg-Marquardt algorithm) leads to explicit values of the constants  $a_1$  to  $a_7$  of Eq. (7) as given in Table 1 and to explicit values of the constants  $b_1$  to  $b_5$  of Eq. (8) as given in Table 2.

$n_s$	$a_1$	$a_2$	$a_3$	$a_4$	$a_5$	$a_6$	$a_7$
$\leq 4$	0.033	-0.168	0.721	1.130	0.938	-1.117	0.031
$> 4 \ \& \ \leq 12$	0.063	-0.579	0.123	0.967	0.035	-1.088	0.035
$>12 \ \& \ \leq 20$	0.017	-0.138	-0.078	0.925	-0.181	-1.158	0.039

Table 1. Indices of Eq. (7)

With respect to the databank, Eq. (7) offers a ratio  $\Delta_{y,ap}/\Delta_{y,ex}$  (ap=approximate, ex=exact) with a mean value of 0.97, a median value of 0.98 and dispersion equal to 0.23, while Eq. (8) offers a ratio  $\Delta_{y,ap}/\Delta_{y,ex}$  with a mean value of 0.97, a median value of 0.97 and dispersion equal to 0.28. In addition, the approximation of the median values  $\Delta_{y,median,ex}$  of the databank compared to those resulting from the proposed Eqs (7) and (8) give a correlation factor  $R^2$  equal to 0.993 and 0.981, respectively.

$n_s$	$b_1$	$b_2$	$b_3$	$b_4$	$b_5$
$\leq 4$	-3.474	4.504	4.528	-1.150	0.022
$> 4 \ \& \ \leq 12$	-2.557	3.461	2.912	-1.040	0.023
$>12 \ \& \ \leq 20$	-3.374	4.268	4.007	-1.117	0.025

Table 2. Indices of Eq. (8)

## 6. COMPARISON OF YIELD DISPLACEMENT FORMULAE

In the present section, some comparisons are presented between the proposed expressions for CFT-MRFs and the existing ones in the literature for all steel MRFs. The selected formula from the literature is that proposed by Dimopoulos et al. «[8]», which describes the displacement profile of steel MRFs at first yielding. Figure 1 presents comparisons between the proposed Eq. (8) for the first yielding displacement of CFT-MRFs, the median of 'exact' values as obtained from the databank and the formula proposed by Dimopoulos et al. «[8]» for all steel MRFs. The geometrical characteristics between these different types of structures (composite and steel) were considered to be the same. It is observed by comparing the results that the displacement profile at first yielding of CFT-MRFs gives smaller displacement values than those resulting from the displacement profile of steel MRFs. This mainly happens because the CFT-MRFs are more stiffer than the steel MRFs because of the filled concrete steel tubes. In addition, as shown in Fig. 1, the difference between these two displacement profiles increases, as the number of floors increases.

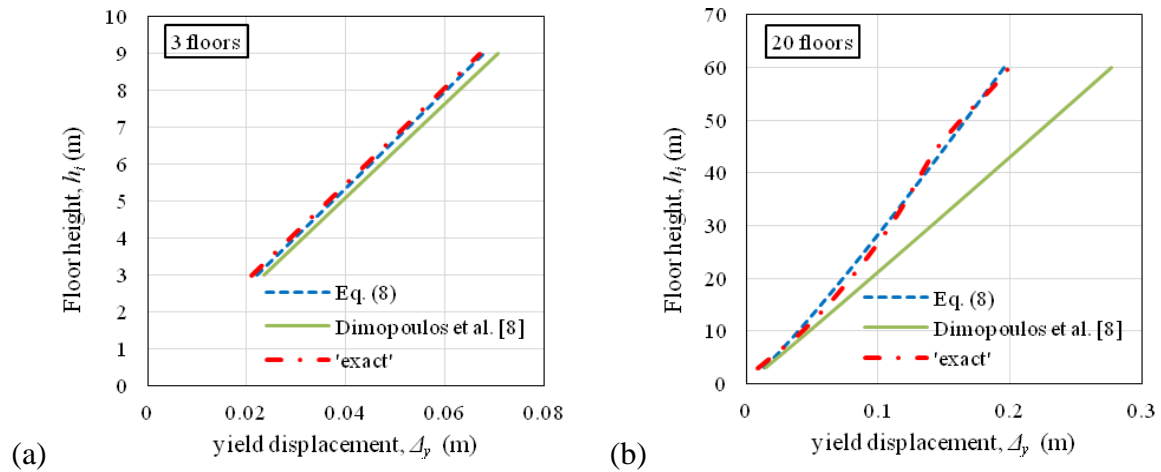


Figure 1. Comparison between CFT-MRFs and all steel MRFs by using Eqs. (8) and equation proposed by Dimopoulos et. al «[8]» for the same frame geometrical characteristics.

Furthermore, Table 3 gives the average values of maximum IDR at first yielding along the height of CFT-MRFs in comparison with those of all steel MRFs presented by Dimopoulos et. al «[8]». It is obvious from this Table that the CFT-MRFs seem to have better seismic behavior until first yielding than the all steel MRFs since they are associated with larger  $IDR_y$ . Furthermore,  $IDR_y$  is not constant as seismic codes consider (e.g., «[9]»), but decreases as the number of stories increases and increases as the grade of steel increases as Table 3 clearly indicates.

$n_s$	Steel MRFs S275	CFT-MRFs S275	Steel MRFs S355	CFT-MRFs S355
3	8.4‰	10.9‰	11.5‰	14.5‰
6	7.5‰	9.4‰	10.2‰	12.3‰
9	6.7‰	8.6‰	9.2‰	11.4‰
12	5.9‰	8.4‰	8.1‰	11.1‰
15	5.7‰	8.0‰	7.8‰	10.8‰
20	5.5‰	7.4‰	7.6‰	10.0‰

Table 3. Average values of maximum IDR along the CFT-MRFs height at first yielding in comparison with those of all steel MRFs presented by Dimopoulos et. al «[8]».

## 7. COMPARISON OF DDBD RESULTS

In this section, the DDBD method as described in section 3, is applied to two regular and plane CFT-MRFs and the obtained drifts and base shears are compared with the average drifts and base shears derived from inelastic dynamic analyses involving ten, compatible to EC8 response spectrum, acceleration time histories. The yield displacement of the equivalent SDOF system needed for the application of the DDBD method is computed by the proposed Eq. (8). The DDBD process is applied to every CFT-MRF for a target design  $IDR_d=1.8\%$ , in the range of life safety performance level. Following the DDBD process «[1], [2]», the ductility index,  $\mu_e$  and the equivalent viscous damping  $\xi_e$  are computed. By applying the computed damping index in the elastic design spectrum of EC8 «[3]» for displacements, the equivalent SDOF period  $T_e$  is obtained for the target SDOF displacement  $\Delta_{e,d}$ . The displacement spectrum is obtained from the corresponding pseudo-acceleration design spectrum of EC8 «[3]» for  $PGA=0.36g$  soil class B and equivalent viscous damping  $\xi_e$ . After the derivation of the equivalent period, the base shear is estimated according to DDBD and is distributed to the floors levels of each CFT-MRF. All the equivalent coefficients and the obtained design base shears are shown in Table 4.

The two CFT-MRFs used here as examples, are designed according to EC3 «[4]» and EC4 «[5]» for the base shear as obtained by the DDBD method, are made of steel S275 and concrete C20 and their geometric characteristics are described in Table 4. In this table, the numeric form, such as 250×12.5 (1-3), means that the first three stories have CFT columns with square steel tubes of width  $b = 250$  mm and thickness  $t = 12.5$  mm, while the numeric forms, such as, 300 (1-3), means that the first three stories have IPE 300 beams.

$n_s$	T (s)	Columns	Beams	$\mu_e$	$\xi_e$ (%)	$T_e$ (s)	$K_e$ (kN/m)	$V_b$ (kN)
3	0.76	250×12.5 (1-3)	300 (1-3)	1.93	13.8	1.29	2563.21	322.96
6	1.27	300×12.5 (1-4), 300×10 (5-6)	330 (1-4), 300 (5-6)	1.80	13.0	1.89	2362.77	455.25

Table 4. Geometrical characteristics of designed CFT-MRFs and their equivalent SDOF coefficients according to DDBD method.

In order to compare the results of the various existing expressions with those of the proposed ones, “exact” results are also obtained on the basis of nonlinear dynamic analyses. Ten ground motions compatible to EC8 «[3]» elastic design spectrum have been produced by a deterministic approach «[10]» and used for nonlinear dynamic analyses of the two frames considered here. The drifts and base shears are recorded and compared with those derived by the DDBD method in Table 5. It can be easily observed from this Table that, the use of the proposed formulae in the DDBD method results in drifts and base shears close to those obtained by nonlinear dynamic analyses.

$n_s$	$V_{b,DDBD}$	Base shears from ten compatible accelerograms											Average (error)
3	322.96	349	368	334	328	376	368	359	346	341	313	348.20 (7%)	
6	455.25	498	477	465	490	460	452	468	488	502	512	481.20 (6%)	
$n_s$	$IDR_{d,DDBD}$	Base shears from ten compatible accelerograms											Average (error)
3	0.018	1.78	1.98	1.65	1.58	1.93	1.70	1.71	1.67	1.77	1.52	1.73 (-4%)	
6	0.018	1.82	1.68	1.52	1.83	1.55	1.68	1.60	1.71	1.80	1.82	1.70 (-5.8%)	

Table 5. Base shears and IDRs from nonlinear dynamic analyses compared with those from DDBD method.

## 8. CONCLUSIONS

On the basis of the previous developments, the following conclusions can be stated:

1. Approximate design formulae for the estimation of lateral yield displacements of composite/steel plane frames under seismic loads have been derived to be used in the context of the DDBD method or any other method requiring knowledge of this kind of displacements
2. These formulae have been derived on the basis of extensive parametric dynamic nonlinear analyses involving 96 CFT-MRFs under 100 ordinary ground motions. They are simple, easy to use in applications and do not require knowledge of member sections.
3. The displacement profile at first yielding of CFT-MRFs consists of smaller displacement values than those associated with the displacement profile of steel MRFs. In addition, the CFT-MRFs seem to have better seismic behavior until first yielding than the all steel MRFs since they develop larger  $IDR_y$ .
4. Comparisons of the base shears and drifts from nonlinear dynamic analyses with those from the DDBD method using the proposed formulae reveal the accuracy and simplicity of that method to estimate the above response quantities.

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**ΣΕΙΣΜΙΚΕΣ ΜΕΤΑΚΙΝΗΣΕΙΣ ΔΙΑΡΡΟΗΣ ΣΥΜΜΙΚΤΩΝ ΕΠΙΠΕΔΩΝ  
ΚΑΜΠΤΙΚΩΝ ΠΛΑΙΣΙΩΝ ΧΑΛΥΒΑ/ΣΚΥΡΟΔΕΜΑΤΟΣ**

**Κωνσταντίνος Σκαλωμένος**

Δρ. Πολιτικός Μηχανικός  
Τμήμα Πολιτικών Μηχανικών, Πανεπιστήμιο Πατρών,  
Πάτρα, Ελλάδα  
e-mail: skalomenos@upatras.gr

**Γιώργος Χατζηγεωργίου**

Αναπληρωτής Καθηγητής  
Σχολή Θετικών Επιστημών & Τεχνολογίας, Ελληνικό Ανοικτό Πανεπιστήμιο,  
Πάτρα, Ελλάδα  
e-mail: hatzigeorgiou@eap.gr

**Dimitri Beskos**

Καθηγητής  
Τμήμα Πολιτικών Μηχανικών, Πανεπιστήμιο Πατρών,  
Πάτρα, Ελλάδα  
e-mail: beskos@upatras.gr

## **1. ΠΕΡΙΛΗΨΗ**

Στη παρούσα εργασία προτείνονται εμπειρικές εξισώσεις για την εκτίμηση των πλευρικών μετακινήσεων κατά την πρώτη διαρροή των επίπεδων σύμμικτων καμπτικών πλαισίων με υποστυλώματα από χάλυβινες κοιλοδοκούς πληρούμενες με σκυρόδεμα και με μεταλλικές δοκούς τύπου I υπό την επίδραση σεισμικών διεγέρσεων. Οι εμπειρικές εξισώσεις θα είναι ιδιαίτερα χρήσιμες στην εφαρμογή της μεθόδου αντισεισμικού σχεδιασμού με βάση τις μετακινήσεις (ΜΣΜ) σε σύμμικτα πλαίσια. Οι εν λόγω εξισώσεις αποτελούν συναρτήσεις των γεωμετρικών και των μηχανικών ιδιοτήτων των πλαισίων και προέρχονται από τη στατιστική ανάλυση μιας βάσης δεδομένων σεισμικών αναλύσεων των υπό εξέταση πλαισίων. Πιο συγκεκριμένα, η βάση αυτή προέκυψε από εκτεταμένες δυναμικές ανελαστικές αναλύσεις 96 καμπτικών σύμμικτων επίπεδων πλαισίων με χάλυβα ποιότητας S275 και S355 και κατηγορία σκυροδέματος C20 και C40 υπό τη δράση 100 σεισμικών διεγέρσεων μακρινού πεδίου. Η ΜΣΜ, χρησιμοποιώντας τις προτεινόμενες εξισώσεις, εφαρμόζεται στη συνέχεια για τον σχεδιασμό δύο νέων σύμμικτων πλαισίων. Κατόπιν, διεξάγονται συγκρίσεις και προκύπτουν συμπεράσματα για την τέμνουσα βάση και τις μέγιστες σχετικές μετακινήσεις των ορόφων των πλαισίων με βάση την προτεινόμενη μέθοδο και με την εκτέλεση δυναμικών ανελαστικών αναλύσεων για 10 επιταχυνσιογραφήματα συμβατά με τις διατάξεις του EC8.