First Yield Inter-storey Drift Ratios of Composite (Steel/concrete) and Steel Frames for the Purposes of the Direct Displacement Based Design Method

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FIRST YIELD INTER-STOREY DRIFT RATIOS OF COMPOSITE (STEEL/CONCRETE) AND STEEL FRAMES FOR THE PURPOSES OF THE DIRECT DISPLACEMENT BASED DESIGN METHOD

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ABSTRACT

The present work provides expressions about the Inter-storey Drift Ratios at the damage state of the first yield (IDR_y). The proposed expressions are compatible with i) composite (steel/concrete) moment resisting frames ($MRFs$), ii) steel $MRFs$, iii) steel eccentrically braced frames ($EBFs$) and iv) steel buckling restrained braced frames ($BRBFs$) for soil types B and D. These expressions have been derived by means of statistical and regression analysis incorporating a large databank composed by numerous IDR values that correspond to the first yield of the aforementioned under-study frames. The databank have been produced after extensive dynamic analyses, considering a plethora of seismic recordings compatible with different soil types. The validity of these expressions is proved through an indicative number of numerical examples. Most importantly, the proposed expressions may constitute a very useful tool towards the direct displacement based design ($DDBD$), where the existing equations for the calculation of the IDR_y are crude and oversimplified, leading to unreliable results. The superiority of the proposed expressions over those stipulated in the $DDBD$ code is verifying through an additional numerical example.

Keywords: *Performance-Based Seismic Design (PBSD); Direct Displacement-Based Design (DDBD); composite structures; steel structures; first yield; inter-storey drift ratio (IDR).*

1. INTRODUCTION

During the past decades, one of the most substantial developments in the field of seismic design of structures, occurred via the establishment of the Performance-Based Seismic Design ($PBSD$) concept (Bozorgnia and Bertero 2004). The performance demands are normally derived either deterministically e.g. in terms of damage of structural or non-structural elements (SEAOC 1995, FEMA-356 2000) or probabilistically, considering the interest in different factors mostly related to economic losses or fatalities (O'Reilly and Calvi 2019). Nowadays, to the authors' knowledge, all the existing seismic design codes for building structures (e.g. EC8 2004) incorporate the well-known Force-Based Design (FBD) method which considers forces as the key design parameters. However, despite the diachronic popularity of the FBD , during the last three decades or so, the Displacement-Based Design (DBD) method stands out as the most predominant alternative (Chopra and Goel 2001, Panagiotakos and Fardis 2001). In particular, DBD method uses displacements as the fundamental design parameter. Considering that displacements are trustworthy indicators of structural damage, while the forces are not, the DBD method can control the level of damage more effectively than the FBD method. The most popular and highly advanced DBD method is entitled direct displacement based design ($DDBD$) and has been developed mainly by (Priestley et al. 2007) in the form of a whole book and (Calvi and Sullivan 2008, Sullivan et al. 2012) in the form two model codes. Additionally, a wide number of articles incorporating the $DDBD$ method have been published, be it for the design of reinforced concrete (Priestley and Kowalsky 2000, Pennucci et al. 2009) or steel structures (Sullivan 2013, Roldán et al. 2016, O'Reilly and Sullivan 2016). Nevertheless, the $DDBD$ method has two significant drawbacks. The first is associated with the replacement of the original multi-degree-of-freedom ($MDOF$) structure by a single-degree-of-freedom ($SDOF$) one, which entails significant loss of modeling accuracy. To this end, (Muho et al. 2020) developed an improved version of the $DDBD$ method, regarding the seismic design of moment resisting frames ($MRFs$) made of R/C, which

employs an equivalent *MDOF* system, produced with the aid of deformation dependent equivalent modal damping ratios (Muho et al. 2019). The work of Muho et al. (2020) was expanded in steel plane i) moment resisting frames (*MRFs*), ii) eccentrically braced frames (*EBFs*) and iii) buckling restrained braced frames (*BRBFs*) by Kalapodis et al. (2022). The second drawback of the *DDBD* method regards the simplistic expressions, provided for the calculation of the yield drift, which is the catalyst for an accurate derivation of the displacement profile of the structure. In particular, for the calculation of the yield drift of a steel *MRF*, one is urged by the *DDBD* code to select a trial beam size and then, perform iterations until they find an appropriate design solution.

Given the above, the present work aspires to contribute towards the tackling of the second drawback of the *DDBD* method, by dint of providing more accurate expressions for the direct calculation of those yield drifts. In this context, parametric equations of the IDR_y , are provided by the authors and involve different types of steel plane frames, like steel *MRFs*, *EBFs*, and *BRBFs*. Since the aforementioned steel frame configurations are also included in the original *DDBD* method, the proposed equations may easily be used in conjunction with *DDBD*, aiming at the derivation of more reliable results along with the shortening of the computational time. Furthermore, for reason of completeness, such expressions are also provided for composite (steel/concrete) moment resisting frames (*CFT – MRFs*), although they do not lie within the scope of the *DDBD* method so far. Apart from the *DDBD* method, the products of this research may assist other, recently advanced, hybrid seismic design methods (Pian et al. 2020, Kalapodis et al. 2020, Serras et al. 2021), which combine the good elements of *FBD* and *DBD*. The parametric equations have been produced after applying regression analysis using the values of a wide data repository, consisted of a mass number of *IDR* values accounting for the first yield of the under-study frames. This repository was created after extensive dynamic analyses, considering a 50 far-field seismic recordings, compatible with soil type B and D (according to the categorization made by EC8 (2004)). The non-linear dynamic analyses were performed by means of Ruaumoko-2D (2006) software. The validity of the expressions is proved through four parametric examples of i) 5-storey and ii) 10-storey *CFT – MRF*, iii) 5-storey *EBF* with long links and iv) 10-storey *EBF* with intermediate links. Finally, the superiority of the proposed expressions against those of the *DDBD* method is proved through an additional numerical example incorporating a 5-storey steel *MRF*.

2. DESIGN AND MODELING OF THE CONSIDERED FRAMES

2.1 Composite (steel/concrete) plane frames

A set of 48 regular composite plane *MRFs* are seismically designed using response spectra of both soil type B and D (according to EC8 (2004)) considering a wide range of structural characteristics, such as the number of stories (n_s) with values of 3, 6, 9, 12, 15, and 20, the yield steel stress (f_y) with values of 275 and 355 *MPa* and compressive concrete strength (f_c) with values of 30 and 50 *MPa*. The number of bays (n_b) equals to 3. Furthermore, the storey height and bay width for each frame are equal to 3 *m* and 6 *m*, respectively. As shown in Fig. 1a, the examined frames consist of circular concrete filled-steel tube (*CFT*) columns (Fig. 1b) and composite beams (steel I-beams connected with concrete floor slabs) of 15*cm* thickness (Fig. 1c). Capacity design considerations have been taken into account by satisfying at every joint the relation $\Sigma M_{RC} \geq 1.3 \Sigma M_{RB}$, where ΣM_{RC} and ΣM_{RB} are the sums of design values of the resistance moments of the columns and beams framing the joint, respectively. Specifically, the frames are designed based on Eurocodes 3 (2009), 4 (2004) and 8 (2004) via SAP2000 (1995) software program. The seismic load combination was taken $G + 0.3Q = 25kN/m$, whereas the gravity load combination $1.35G + 1.5Q = 42kN/m$. Moreover, the design ground acceleration (a_g) and the behavior factor (q) are considered equal to 0.36*g* and 4.0, respectively, accounting for medium class structural ductility (*DCM*) and Spectrum Type 1. For further detail, one may refer to Serras (2019).

Fig. 1 **a** A typical CFT-MRF, **b** circular CFT column section and **c** composite beam section (steel beam connected with concrete slab) (Serras 2019).

2.2 Steel plane frames

Three different types of steel frames, i.e. *MRFs* (Fig. 2a), *EBFs* (Fig. 2b) and *BRBFs* of chevron configuration (Fig. 3a), are considered. Regarding the *EBFs*, use is made of short, intermediate and long seismic links with lengths $x = 0.5, 1.0$ and $1.5m$, respectively (Fig. 2b). In addition, two different types of stiffness is considered for each *EBF* and *BRBF*. For the case of *EBFs*, *BRBFs* and *MRFs*, the selected section types for beams and columns are *IPE* and *HEB* respectively. The brace section of the *EBFs* is considered *CHS*, while the core section of the *BRBs* is of orthogonal shape (Fig. 3b). All types of the examined steel frames follow different structural characteristics, such as the number of stories (n_s) with values of 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16 for *MRFs* and 2, 3, 6, 9, 12, 15 and 17 for *EBFs* and *BRBFs*. The storey height and bay width are taken equal to 3 m and 5 m each, the number of bays (n_b) is equal to 3 and the grade of steel accounts to S275. The number of the examined steel *EBFs* are $7\{\text{different heights}\} \times 3\{\text{different link lengths}\} \times 2\{\text{different types of stiffness}\} = 42$ frames, the number of steel *BRBFs* equals to $7\{\text{different heights}\} \times 2\{\text{different types of stiffness}\} = 14$ frames and the number of *MRFs* is equal to 16. Thus, the total number of steel frames is equal to 72. The frames are designed according to Eurocodes 3 (2009) and 8 (2004) with the aid of SAP2000 (1995) software program. The seismic load combination $G+0.3Q$ is deemed 27.5 kN/m , while the design ground acceleration (a_g) and the behavior factor (q) are equal to $0.24g$ and 4.0 , respectively, for *DCM* and Spectrum Type 1. More information about the design of these plane steel frames can be found in Kalapodis et al. (2018), Kalapodis et al. (2020), Kalapodis et al. (2021), Kalapodis and Papagiannopoulos (2021).

Fig. 2 Steel: **a** moment resisting frames and **b** eccentrically braced frames of chevron bracing (Kalapodis et al. 2021).

Fig. 3 a Buckling restrained braced frames and **b** cross section of a buckling restrained brace (Kalapodis et al. 2018, Kalapodis et al. 2021).

2.3 Modeling of frames and ground motions considered

The inelastic behavior of the examined structure models are investigated with the aid of Ruaumoko-2D (2006). In all cases, $P-\Delta$ effects by using the “Large-displacement” analysis option are taken into account. A diaphragm action is also considered at every floor due to the presence of the concrete slab. In addition, a response databank is generated for soil types B and D, taking into consideration various motions compatible with those of soil types. Regarding composite and steel frame members, the circular *CFT* columns are modeled according to (Serras et al. 2016, Serras et al. 2017), while the composite beams in line with Eurocode-4 (2004). On the other hand, steel columns and beams are modeled on the basis of Kalapodis et al. (2018). Both composite and steel frames are simulated by using bilinear hysteretic model (Carr 2006). The effect of gusset plates for the case of *EBFs* and the shear behavior of the short links are taken into consideration (Kalapodis et al. 2018). The under-study frames are subjected to a set of 50 far-field seismic recordings (two groups of 25 recordings compatible either with soil type B or D) (Kalapodis et al. 2018). Their seismic response is determined with the aid of Ruaumoko-2D (2006) for performing non-linear dynamic analyses with Rayleigh damping equal to 3.0% (steel frames) or 4.0% (composite frames) in the first and $N^{st} = n_s$ modes for steel and CFT frames, respectively.

3. PROPOSED EXPRESSIONS FOR THE CALCULATION OF THE IDR AT THE FIRST YIELD

In order to calculate the interstorey drift ratios at the first yield (IDR_y) of the examined frames, parametric equations are proposed in this section, obtained by regression analysis of the aforementioned response databank. The reliability of the proposed equations is verified through the correlation coefficient (R^2). The results of the proposed equations and dynamic inelastic analyses are denoted as “approximate” values and “exact” values, respectively. The accuracy of the proposed equations is found to be satisfactory.

3.1 Expressions compatible with composite (steel/concrete) plane frames.

After performing regression analysis the IDR_y can be expressed in terms of storey number (n_s), first natural period (T) and the design spectral acceleration ($S_a(T)$) for composite frames. Eq. 1 provides expression for *CFT – MRFs* applicable for soil types B and D. Furthermore, the corresponding values of the variables k_i as well as the values of the correlation coefficient R^2 are shown in Table 1. Also, Figure 4 shows indicative scatter diagrams where the results of the Eq.1 are compared with those obtained by dynamic analysis, using a detailed statistical investigation. For this reason, the 16% and 84% confidence levels corresponding to the median plus/minus one standard deviation give also the uncertainties associated with the seismic records.

$$IDR_y = \left\{ T^{k_1} \cdot n_s^{k_2} \cdot \left(\frac{20}{f_c} \right)^{k_3} \cdot \left(\frac{235}{f_y} \right)^{k_4} \cdot S_a(T)^{K_5} \right\} \cdot k_6 + k_7 \quad (1)$$

Table 1 Parameters (k_i) and correlation coefficient (R^2) of Eqs. 1 and 2 for soil types B and D, respectively.

n_s	Soil	Type of CFT frame	R^2	k_1	k_2	k_3	k_4	k_5	k_6	k_7
$3 \leq n_s \leq 20$	B	MRF	74%	1.7145	-9.412	-0.90	-1.656	-7.608	47.869	0.0036
	D		60 %	6.1143	-6.973	-1.013	-1.862	-1.933	51.111	0.0033

(a)

(b)

Fig. 4 CFTFs: IDR_y of Proposed equations versus values obtained by “dynamic analysis” for: **a** soil type B and **b** soil type D.

Furthermore, a simplified expression (compatible for soil types B and D) is also proposed for this type of frames (Serras 2019) where, f_y is the yield steel stress in MPa.

$$IDR_y(\%) = 2.25 + 1.45 \cdot \left(\frac{f_y}{235} - 1 \right) \quad (2)$$

3.2 Expressions for steel plane frames.

In a same manner, equations for the calculation of the IDR_y values of various steel plane frame configurations are presented. The involving parameters are the storey number (n_s) and the first natural period (T) for the *MRFs* and *BRBFs*, while an extra parameter regarding the case of steel *EBFs* is the ratio of the seismic link length (x) over the length of the bay (b) (Fig.2b). Specifically, Eq. 3 provides IDR_y values for steel *MRFs*, *BRBFs* and *EBFs* for soil types B and D. Additionally, for reasons of completeness, a more detailed expression is provided (Eq. 4) for the case of steel *EBFs* per se, incorporating a wide range of seismic link to bay length ratios, i.e. $0.1 \leq x/b \leq 0.3$. Tables 2-3 exhibit the constants k_i involved in Eqs. 3 and 4 along with the values of the corresponding correlation coefficients R^2 .

$$IDR_y = (T^{k_1} \cdot n_s^{k_2}) \cdot k_3 \quad (3)$$

Table 2 Parameters k_i and correlation coefficients (R^2) for Eq. 3.

n_s	Soil type	Type of steel frame	R^2	k_1	k_2	k_3
$2 \leq n_s \leq 17$	B	MRFs	99.41%	-0.5713	2.7022	0.00010
		BRBFs	93.37%	0.0046	0.3741	0.00186
		EBFs (short links)	79.76%	1.3908	-0.8893	0.03127
		EBFs (intermediate links)	86.12%	-0.0754	0.3662	0.00215
		EBFs (long links)	77.66%	0.4908	-0.1930	0.00671
	D	MRFs	99.68%	-0.6256	2.7084	0.000074
		BRBFs	88.67%	0.1751	0.2385	0.00257
		EBFs (short links)	79.27%	-0.4688	-0.8633	0.00060
		EBFs (intermediate links)	89.00%	-0.1334	0.4308	0.00191
		EBFs (long links)	83.08%	0.7402	-0.3607	0.00951

$$IDR_y = \left\{ T^{k_1} \cdot n_s^{k_2} \cdot \left(\frac{x}{b} \right)^{k_3} \right\} \cdot k_4 \quad (4)$$

Table 3 Parameters k_i and correlation coefficients (R^2) of Eq. 4, pertaining to steel *EBFs*.

n_s	Soil types	Type of steel frame	R^2	k_1	k_2	k_3	k_4
$2 \leq n_s \leq 17$	B	$(0.1 \leq x/b \leq 0.3)$	80.00%	0.4239	-0.0841	0.0991	0.0063
	D	$(0.1 \leq x/b \leq 0.3)$	72.47%	-2.5913	3.1896	0.3778	0.0001

Following the same process as with *CFT* frames, indicative scatter diagrams for soil types B and D are shown in Fig. 5 for the case of steel *BRBFs*.

Fig. 5 BRBFs: IDR_y of Proposed equation versus values obtained by “dynamic analysis” for: **a** soil type B and **b** soil type D.

4. VALIDATION OF THE PROPOSED EXPRESSIONS

In this section, four plane frame exemplars (two *CFT – MRFs* and two steel *EBFs*), are investigated in-detail considering soil type B and D, in order to verify the equations proposed herewith.

4.1 Numerical examples involving two CFT-MRFs

A five-storey and a ten-storey composite frames consisting of three-bay founded on soil types D and B, respectively, are designed. Their bay width is equal to 5 m whereas their storey height is taken equal to 3 m. The steel yield stress (f_y) and the concrete compressive strength (f_c) are assumed to be 235 and 40 MPa, respectively. Each joint comprises two circular CFT columns and one composite (steel/concrete) beam consisting of an IPE section connected to a concrete slab with thickness equal to 15 cm. The examined frames are designed based on Eurocodes 3 (2009), 4 (2004) and 8 (2004) by using SAP2000 (1995) software program. The design PGA is assumed 0.36g and 0.30g for soil types B and D respectively, while the behavior factor $q=4$. Table 4 presents further details about the designed frames.

Table 4 Characteristics of the designed CFT – MRFs.

Soil type	Storey number	CFT columns dimensions – material properties	Floors	Beams (IPE)	$T(\text{sec})$	$PSA(\text{g})$	
B	1-7	(406.4x6.3)-40-235	1-3	300	1.521	0.374	
			4-6	270			
	8-10	(355.6x6.0)-40-235	7-8	240			
			9-10	220			
D	1-2	(457x8.0)-40-235	1	270	0.760	1.067	
			2	240			
	3-4	(406.4x6.3)-40-235	3	240			
			4	220			
	5	(355.6x6.0)-40-235	5	200			

Each $CTF - MRF$ is subjected to non-linear time history analysis using a set of six seismic recordings, either compatible with soil type B or D, by means of Ruaumoko-2D (2006). Table 5 presents the results produced by Eqs 1 and 2, as well as the values of IDR_y obtained by the non-linear time history analysis for soil types B and D. It can be derived that the results of Eq. 1 provide satisfactory convergence with those of time-history analysis. However, although Eq. 2 is an uncomplicated expression, the level of convergence is not as high as that of Eq. 1.

Table 5 CFT – Frames: IDR_y given by time-history analysis versus IDR_y given by the proposed equations.

Soil type	n_s	Mean IDR_y (‰)	Eq. 1 (‰)	Error (%) – Eq. 1	Eq. 2 (‰)	Error (%) – Eq. 2
B	10	3.03	3.72	18.55	2.25	25.74
D	5	3.18	3.52	9.66	2.25	29.24

4.2 Numerical examples involving two steel EBFs

Additionally, a five-storey and a ten-storey EBF are also designed for soil types B and D, respectively. Their bay width and their storey height are equal to 5 m and 3 m, while the grade of steel is considered S275. The chosen steel sections were IPE (for beams), HEB (for columns) and CHS (for braces). The examined frames are designed according to EC3 (2009) and EC8 (2004) by means of SAP2000 (1995) software program. The design PGA is considered 0.24g for soil types B and D, where the behavior factor $q=4$. Table 6 summarizes the characteristics of the examined frames. Each steel EBF of Chevron bracing is subjected to the same six seismic recordings (as it was the case with the previous example). The results are summarized in Table 7, where the values of the proposed equations are compared with the mean values of IDR_y obtained by the non-linear time history analyses for soil types B and D. On the grounds of

Table 7, it can be seen that the proposed equations provide quite satisfactory convergence with the real results.

Table 6 Geometric and material characteristics of the designed CFT – MRFs.

Soil	Types of EBFs	n_s	Sections: HEB (columns) - IPE (beams) – CHS (braces)	T (sec)
B	EBF (long links)	5	340-360-219.1x5(1-2), 300-300-193.7x4.5(3-4), 260-270-193.7x4.5(5)	0.681
D	EBF (intermediate links)	10	500-450-273.5x5.6(1), 450-450-273.5x5.6(2), 400-400-244.5x5.6(3-4), 360-360-219.1x5(5-6), 320-330-193.7x4.5(7-8), 280-300-193.7x4.5(9-10)	0.932

Table 7 Steel EBFs: IDR_y given by time-history analysis versus IDR_y given by the proposed equations.

Soil	n_s	EBF type	Mean IDR_y (‰)	Eq. 3 (‰)	Error (%) – Eq. 3	Eq. 4 (‰)	Error (%) – Eq. 4
B	5	EBF (long links)	4.02	4.07	1.22	4.24	5.19
D	10	EBF (intermediate links)	5.10	5.17	1.35	5.08	0.40

5. NUMERICAL EXAMPLE

A steel plane 5-storey *MRF* is seismically designed according to the provisions of *DDBD* method (Calvi and Sullivan 2008, Sullivan et al. 2012) either obtaining the IDR_y based on the simplistic expression of (Sullivan et al. 2012), or through the use of the proposed equation (Eq. 3). More details about the *DDBD* procedure can be found in Calvi and Sullivan (2008), Sullivan et al. (2012), Muho et al. 2020 and Kalapodis et al. (2022). Then, aiming at achieving a fair comparison, the same frame is also designed according to EC8 (2004). The grade of steel is chosen S275 and the types of steel sections are considered HEB for the columns and IPE for the beams. Finally, the accuracy of the three different design procedures is attested through the use of non-linear time history analysis (Carr 2006), where the three designed frames are subjected to 10 seismic recordings for soil type B, appropriately scaled to be compatible with the elastic design spectrum. The mean values produced by the non-linear time history (NLTH) analysis and the results of the spectral analysis are concentrated in Table 10. At the onset of *DDBD* procedure, the initial steel sections size is derived after a preliminary design via EC8 (2004), assuming $PGA=0.36g$ and soil type B. Furthermore, according to *DDBD*'s provisions, $IDR_y = 0.65\varepsilon_y \frac{L_b}{h_b}$, where L_b is the length of the beam (5 m), h_b is the height of the beam (0.36 m – IPE-360) and ε_y is the yield strain of steel (for S275, $\varepsilon_y = 0.00131$). Based on this simplistic equation, the $IDR_y = 0.0118$. Procedural information of the design according solely to the *DDBD* method is given in Table 8. Moreover, the same procedure is followed with the IDR_y derived by the proposed equation (Eq. 3) and found equal to 0.008. Akin to the original *DDBD* method, the procedural results of the modified *DDBD* method are summarized in Table 8. Due to space limitation, the values of u_i and F_i are given in text form. For the original *DDBD* the u_i (m) values (starting from the 1st to the 5th storey) are: 0.075, 0.142, 0.201, 0.253 and 0.296 (same results with the modified *DDBD*). The F_i (kN) values for the original *DDBD* are: 21.34, 40.44, 57.29, 71.89 and 114.83 while for the modified *DDBD* method the values are: 17.43, 33.03, 46.79, 58.72 and 93.79. The steel sections of the three designed frames is shown in Table 9.

Table 8 Structural and design force values of the designed frame.

Method	u_d (m)	H_e (m)	μ	m_e (kNs ² /m)	ξ_{eq} (%)	$u_{D,\xi}$ (m)	T_e (sec)	K_e (kN/m)	V_d (kN)
DDBD	0.225	10.71	1.78	180.35	13.03	0.183	2.46	956.74	305.8
Modified DDBD	0.225	10.71	2.63	180.35	16.38	0.166	2.72	707.92	249.8

Table 9 Steel sections selected after the seismic design according to various methods.

Storey	Original DDBD		Modified DDBD		Eurocode 8	
	HEB	IPE	HEB	IPE	HEB	IPE
1	450	360	450	330	450	300
2	400	330	400	330	400	300
3	360	330	340	330	340	300
4	320	330	300	330	300	300
5	320	330	300	330	300	300

Table 10 Mean values obtained by NLTH analysis compared with that of the spectral analysis.

Method	Original DDBD		Modified DDBD		Eurocode 8	
	Base shear (kN)	IDR≤0.025	Base shear (kN)	IDR≤0.025	Base shear (kN)	IDR≤0.025
NLTH	370.5	0.0156	341.6	0.0167	318.6	0.0169
Spectral	313.1		249.8		261.3	0.0157

6. CONCLUSIONS

This work focuses on the production of parametric equations for the direct calculation of inter-storey drift ratio at the moment of the first member yield (IDR_y), involving a wide range of composite (*CFT – MRFs*) and steel (*MRFs, BRBFs, EBFs*) plane frames. These equations are produced on the basis of regression analysis, incorporating a large databank of values obtained by non-linear time history (*NLTH*) analysis. The reliability of the proposed equations is verified by the high level of convergence with the corresponding results produced by *NLTH* analysis. Importantly, for the case of steel frames, the proposed equations can be used cooperatively with the provisions of the direct displacement-based design method (*DDBD*), where the expressions, provided by the code, for the calculation of the IDR_y are empirical and oversimplified. In particular, with the aid of a numerical example using a 5-storey steel *MRF*, it is shown that the proposed equations used in conjunction with the *DDBD* (modified *DDBD*) can lead to a more economic design than the original *DDBD* method. Furthermore, the 5-storey steel *MRF* is also seismic designed according to EC8. Interestingly, although the base shears developed after designing with EC8 and the modified *DDBD* are almost equal, the modified *DDBD* method led to less economic design. This can be explained by the high amount of seismic force considering by the modified *DDBD* method at the top of the frame (F_5). Finally, the results of the *NLTH* analyses indicate that all the aforementioned approaches can lead to a safe seismic design since the mean *IDR* values do not exceed the limit of 0.025 which corresponds to the life safety (*LS*) performance level (SEAOC 1995).

STATEMENTS & DECLARATIONS

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