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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Research Article

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

Posted Date: February 1st, 2022

DOI: https://doi.org/10.21203/rs.3.rs-1304418/v1

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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 ($\text{IDR}_y$). The proposed expressions are compatible with i) composite (steel/concrete) moment
resisting frames ($\text{MRFs}$), ii) steel $\text{MRFs}$, iii) steel eccentrically braced frames ($\text{EBFs}$) and iv) steel
buckling restrained braced frames ($\text{BRBFs}$) 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
$\text{IDR}$ 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 ($\text{DDBD}$), where the existing equations for the
calculation of the $\text{IDR}_y$ 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 ($\text{PBSD}$); Direct Displacement-Based Design ($\text{DDBD}$);
composite structures; steel structures; first yield; inter-storey drift ratio ($\text{IDR}$).

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 ($\text{PBSD}$) 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 ($\text{FBD}$) method
which considers forces as the key design parameters. However, despite the diachronic popularity of the
$\text{FBD}$, during the last three decades or so, the Displacement-Based Designed ($\text{DBD}$) method stands out as
the most predominant alternative (Chopra and Goel 2001, Panagiotakos and Fardis 2001). In particular,
$\text{DBD}$ method uses displacements as the fundamental design parameter. Considering that displacements
are trustworthy indicators of structural damage, while the forces are not, the $\text{DBD}$ method can control the
level of damage more effectively than the $\text{FBD}$ method. The most popular and highly advanced $\text{DBD}$
method is entitled direct displacement based design ($\text{DDBD}$) 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 $\text{DDBD}$ 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
$\text{DDBD}$ method has two significant drawbacks. The first is associated with the replacement of the original
multi-degree-of-freedom ($\text{MDOF}$) structure by a single-degree-of-freedom ($\text{SDOF}$) one, which entails
significant loss of modeling accuracy. To this end, (Muho et al. 2020) developed an improved version of
the $\text{DDBD}$ method, regarding the seismic design of moment resisting frames ($\text{MRFs}$) 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 \( \Sigma M_{RC} \) and \( \Sigma M_{RB} \) 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 0.36g and 4.0, respectively, accounting for medium class structural ductility (DCM) and Spectrum Type 1. For further detail, one may refer to 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 $1.5\, m$, 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 0.24$g$ 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. 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-Δ 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.
\[
IDR_y = \left( T^{k_1} \cdot n_s^{k_2} \cdot \left( \frac{20}{f_c} \right)^{k_3} \cdot \left( \frac{235}{f_y} \right)^{k_4} \cdot S_a(T)^{k_5} \right) \cdot k_6 + k_7
\]  

(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 |

 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.

\[
IDR_y(\%o) = 2.25 + 1.45 \cdot \left( \frac{f_y}{235} - 1 \right)
\]  

(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. 3and 4 along with the values of the corresponding correlation coefficients \(R^2\).

\[
IDR_y = \left( T^{k_1} \cdot n_s^{k_2} \right) \cdot k_3
\]  

(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 ≤ \( n_s \) ≤ 17 | MRFs | 99.41% | -0.5713 | 2.7022 | 0.00010 |
| | BRBFs | 93.37% | 0.0046 | 0.3741 | 0.00186 |
| | B | 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 |
| | MRFs | 99.68% | -0.6256 | 2.7084 | 0.000074 |
| | BRBFs | 88.67% | 0.1751 | 0.2385 | 0.00257 |
| | D | 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 |

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

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 ≤ \( n_s \) ≤ 17 | B | (0.1 ≤ x/b ≤ 0.3) | 80.00% | 0.4239 | -0.0841 | 0.0991 | 0.0063 |
| | D | (0.1 ≤ x/b ≤ 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.](image)

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.

| Soil type | Storey number | CFT columns dimensions – material properties | Floors | Beams (IPE) | $T$(sec) | PSA(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 |
|           | 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 Ruauumoko-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.

| 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 where 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 | 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<sub>y</sub> given by time-history analysis versus IDR<sub>y</sub> given by the proposed equations.**

| Soil | n<sub>s</sub> | EBF type | Mean IDR<sub>y</sub> (%<sub>o</sub>) | Eq. 3 (%<sub>o</sub>) | Error (%) – Eq. 3 | Eq. 4 (%<sub>o</sub>) | 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<sub>y</sub> 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<sub>y</sub> = 0.65<sub>y</sub><sup>L<sub>b</sub></sup>/<sup>H<sub>b</sub></sup><sub>y</sub>, where <sup>L<sub>b</sub></sup> is the length of the beam (5 m), <sup>H<sub>b</sub></sup> is the height of the beam (0.36 m – IPE-360) and <sub>y</sub> is the yield strain of steel (for S275, <sub>y</sub> = 0.00131). Based on this simplistic equation, the IDR<sub>y</sub> = 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<sub>y</sub> 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 <sub>u</sub> and <sub>F</sub> are given in text form. For the original DDBD the <sub>u</sub><sub>(m)</sub> values (starting from the 1<sup>st</sup> to the 5<sup>th</sup> storey) are: 0.075, 0.142, 0.201, 0.253 and 0.296 (same results with the modified DDBD). The <sub>F</sub> (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 | <sub>u</sub><sub>(m)</sub> | <sub>H</sub><sub>(m)</sub> | <sub>μ</sub> | <sub>m</sub><sub>ε</sub><sub>(kN)/s/m</sub> | <sub>ε</sub><sub>eq</sub><sub>ε</sub><sub>(%<sub>o</sub>)</sub> | <sub>u</sub><sub>(m)</sub> | <sub>T</sub><sub>(sec)</sub> | <sub>K</sub><sub>ε</sub><sub>(kN/m)</sub> | <sub>V</sub><sub>(kN)</sub> |
|--------|-----------------|----------------|------|-----------------|-----------------|----------------|----------------|----------------|----------------|
| 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 HEB | Original DDBD IPE | Modified DDBD HEB | Modified DDBD IPE | Eurocode 8 HEB | Eurocode 8 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 Base shear (kN) | IDR≤0.025 | Modified DDBD Base shear (kN) | IDR≤0.025 | Eurocode 8 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 ($IDR_\gamma$), involving a wide range of composite ($CFT$ – $MRF_s$) and steel ($MRF_s$, $BRBF_s$, $EBF_s$) 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 $IDR_\gamma$ 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

The authors declare that no funds, grants, or other support were received during the preparation of this manuscript.

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