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HIGHER MODE EFFECTS IN THE DIRECT DISPLACEMENT-BASED DESIGN OF STEEL MOMENT RESISTING FRAMES WITH SETBACKS

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ABSTRACT

Extensive research has been undertaken by numerous authors in order to develop the Direct Displacement-Based Design method (DDBD) for a range of structural types and materials. However, an effort is still required to investigate the applicability of the methodology to structures that do not comply with the standard definitions of regularity, such as steel moment resisting frames with setbacks, for these make up a generally underexplored area within the Earthquake Engineering community, even though the number of related studies has significantly increased in recent years. With the purpose of assessing the applicability of the DDBD procedure and its recommendations on how to account for higher mode effects, two 12-storey steel moment resisting frames with two levels of setbacks are designed and subjected to a series of non-linear time-history analyses for increasing levels of intensity. Some preliminary comparisons with research results in the literature suggest that existing expressions developed to estimate the expectable effect of higher modes over interstorey drift amplification of steel moment resisting frames with setbacks could be adapted for the DDBD of such structures. Furthermore, the potential need to account for the influence of ductility demand and the characteristics of ground motion on higher mode amplification is discussed.

INTRODUCTION

Recognized as very promising within the context of the Performance Based Design philosophy, the Direct Displacement-Based Design method (Priestley et al., 2007) has undergone a fast increase in its level of development in the last years (Sullivan et al. 2013, Maley et al. 2010, Khan et al. 2014, Sullivan and Lago 2012). However, vertically irregular structures are still under study, and a greater effort is needed to successfully extend DDBD to these and all kinds of irregular structures in general. Due to architectural and urbanistic constraints, moment resisting frames with setbacks are quite common, even though their seismic behaviour has not yet been fully understood. As Karavasilis et al. (2008) and De Stefano and Pintucchi (2007) point out, different authors working on the subject have systematically arrived to different conclusions.

Within the important number of studies that have been conducted recently in the area, the work of Karavasilis et al. (2008) is of particular interest, for it extensively investigates the seismic response of 120 steel frames with setbacks, subject to a set of 30 ground motion records, at four performance levels. The authors have found that the maximum interstorey drift can be estimated with a reasonable level of accuracy as a function of the number of storeys and a couple of geometric indices, previously defined by Mazzolani and Piluso (1996).

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As the DDBD method is based on the response of a structure's fundamental mode of vibration, the effect of higher modes on the behaviour of the structure is currently accounted for by means of a higher mode reduction factor (ω_θ) applied to the design displacement profile. In this way, the frame is actually designed for a reduced maximum drift ($\theta_{max\ design}$), which is expected to lead to a design in which the maximum code drift (θ_c) will occur, due to the combined effect of the first and the higher modes, as shown in Fig.1(a). The DBD12 Model Code (Sullivan et al., 2012, termed "DBD12" hereafter) currently suggests a tri-linear relationship between the higher mode reduction factor and the number of storeys of the structure, as shown in Fig.1(b), to be used with regular frame structures.

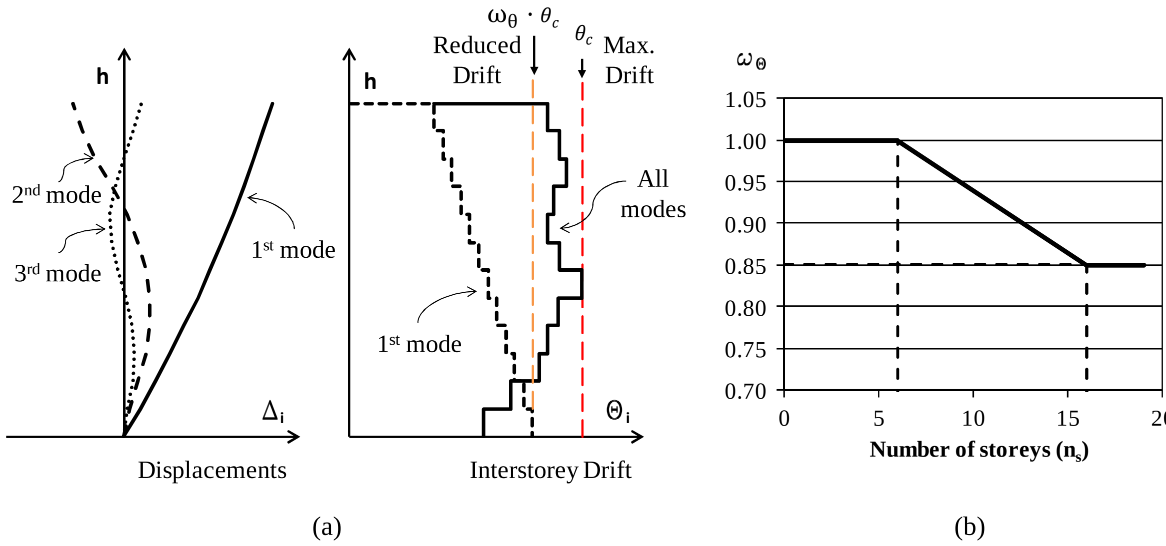


Figure 1. Definition of ω_θ for regular frames according to DBD12

Given the need to verify the applicability of current recommendations to account for higher mode effects on steel moment resisting frames with setbacks, this paper describes the main steps of the DDBD process that require special attention when dealing with this type of structures and analyzes the possibility of modifying the higher mode reduction factor to be used. With this purpose, the design of two 12-storey steel moment resisting frames with two levels of setbacks according to the DDBD procedure currently prescribed for regular structures in DBD12 is described. The frames will be later subjected to a series of non-linear time-history analyses to gauge the performance of the DBD solutions developed, and the results will be confronted with those of Karavasilis et al. (2008), with the aim of elaborating recommendations for the adjustment of the higher mode reduction factor to be used with steel moment resisting frames with setbacks.

DDBD OF CASE STUDY BUILDINGS

The two 12-storey moment resisting frames selected for this study present two levels of setbacks, a constant interstorey height of 3.50 m and a constant bay length of 6.00 m, as shown in Fig.2, and they are assumed to be responsible for stabilizing 122.67 t of seismic mass per bay, at all storeys. Three-dimensional effects are not accounted for, vertical irregularity is assumed in only one direction, and the lateral resisting systems of the buildings are assumed to be independent in the two main directions of the building plan.

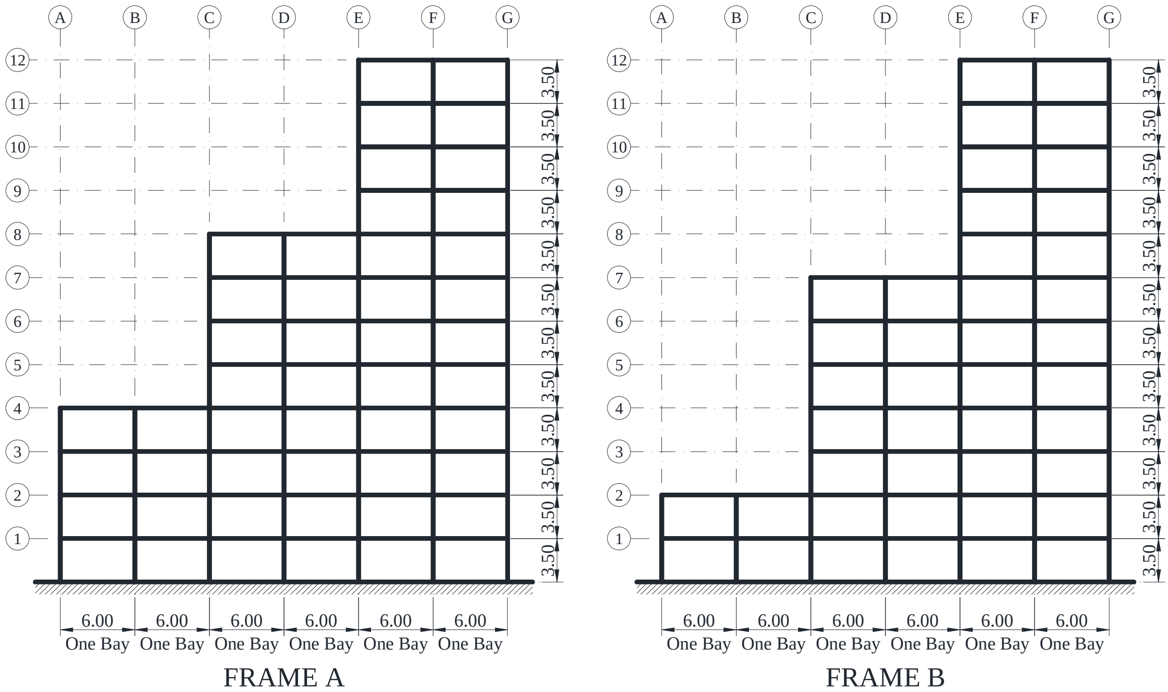


Figure 2. Case study buildings elevations

Design is conducted according to DBD12, Eurocode 3 (CEN, 2010) and Eurocode 8 (CEN, 2004). Wherever there exists overlap between DBD12 and Eurocode 8 (CEN, 2004), specifications from DBD12 are considered to prevail. General (non country-specific) suggested parameters are adopted for Eurocode 3 (CEN, 2010) and Eurocode 8 (CEN, 2004). A type 1 design spectrum from Eurocode 8 (CEN, 2004) is assumed for a design peak ground acceleration on rock of 0.3g, soil type A and an elastic damping of 3%, but using a displacement spectrum corner period T_D of 8.0 s instead of 2.0 s, due to the possible underestimation of spectral displacements caused by the use of 2.0, as reported by Priestley et al. (2007) and Boore and Bommer (2005). A maximum drift limit (θ_c) of 2.5% is assumed for the damage control limit state, which proves to be more restrictive than a maximum drift limit of 1.0% for the serviceability limit state, but is anyway later reduced to 1.90% and 1.88% (for Frame A and B, respectively) in order for the P-Delta stability coefficient not to exceed the limit of 0.30 specified in DBD12. Commercial HE-M, HE-B and IPE sections are used. The steel is assumed to be S450, with a yield strength of 440 MPa, ultimate strength of 550 MPa, expected yield strength of 484 MPa and maximum feasible yield strength of 572 MPa.

The design process is carried out following DBD12 and Priestley et al. (2007). Interested readers can refer to Nievas and Sullivan (2014) for further details. Table 1 summarizes the properties calculated for the equivalent single-degree-of-freedom system: maximum design interstorey drift ($\theta_{max\ design}$), characteristic design displacement (Δ_d), effective mass (m_e), effective height (h_e), displacement ductility demand (μ_{sys}), equivalent viscous damping (ξ_{eq}), inelastic reduction factor (η_{in}), effective period (T_e), effective stiffness (K_e), increment in base shear due to P-Delta effects ($V_{P\text{-}\Delta}$), and design base shear (V_b).

Table 1. Equivalent Single-Degree-of-Freedom System Properties:

Frame	$\Theta_{max\ design}$ **(rad)**	Δ_d **(mm)**	m_e **(Tn)**	h_e **(m)**	μ_{sys}	ξ_{eq}	η_{in}	T_e **(s)**	K_e **(kN/m)**	$V_{P\text{-}\Delta}$ **(kN)**	V_b **(kN)**
A	0.0190	388	4538	24.138	1.059	5.49%	0.959	4.857	7595	730	3677
B	0.0188	391	3905	24.677	1.032	5.26%	0.977	4.803	6682	620	3232

As it can be observed, ductility demands are relatively low, due to the flexible nature of steel moment resisting frame systems. The low design ductility demands imply that some members of the structure are not expected to yield at peak response for the design intensity. Priestley et al. (2007) do not elaborate much on the design of these members, and thus a decision is made based on the fact that the DDBD method is an effective stiffness-driven design process: storey shear stiffness needs to account for the effects of ductility demand and, therefore, the required storey yield shear ($V_{i,y}$) of storeys not expected to yield is bigger than the required storey shear (V_i, calculated from the distribution of base shear up the height of the structure), and it is determined simply by dividing the latter by the expected ductility demand of the storey (μ_i), as shown in Eq.1. For the case of storeys expected to yield, the required storey yield shear is instead smaller than the required storey shear, due to the effects of strain hardening (represented by the post-stiffness ratio *r*), as shown in Eq.2. Note that Roldán et al. (2014) have recently proposed a promising alternative, but this is not examined in this work.

$$V_{i,y} = V_i/\mu_i \qquad \text{for } \mu_i < 1.0 \tag{1}$$
$$V_{i,y} = V_i/[1 + r(\mu_i - 1)] \qquad \text{for } \mu_i \geq 1.0 \tag{2}$$

As suggested by Priestley et al. (2007), the designer can have an active role in the distribution of actions in members if an equilibrium approach is used instead of undertaking a conventional structural analysis. Any strength distribution is acceptable, as long as external actions can be resisted and displacement compatibility is respected. This approach is adopted here, and hence it is decided to distribute storey shear to individual columns as a function of the number of beams supported by the columns; that is, it is decided that external columns carry half the shear (and thus half the moment) of internal columns, for the nodes of the first receive only one beam, while those of the latter receive two. While deciding the beams' and columns' relative strength, implicit decisions are made with respect to the contraflexure height of columns. In this work, it is assumed that ground columns present their contraflexure point at 0.6 times the storey height above the base level, while a mid-height point of contraflexure is assumed for all the other columns.

The decision made with respect to the distribution of storey shear among internal and external columns implicitly assumes that all beams of the same storey are assigned the same beam section. This is true for all storeys except for those in which the setbacks occur, because if the assumed distribution of storey shear among columns is maintained above and below this level, then not all the beams at the setback level present the same moment demand. This situation is exemplified in Fig.3 for the case of level 8 of Frame A, in which M_{bR8} and M_{bL8} are the plastic hinge moments for the beams to the "right" and "left" of the setback, respectively, and $M_{c\,int\,i}$ and $M_{c\,ext\,i}$ represent the moment demand in internal and external columns of level *i*. The equilibrium equations to be satisfied are:

$$M_{bL8} = M_{c\,ext\,8} = M_{c\,int\,8}/2 \tag{3}$$
$$2 \cdot M_{bR8} = M_{c\,int\,8} + M_{c\,int\,9} \tag{4}$$
$$M_{bL8} + M_{bR8} = M_{c\,int\,8} + M_{c\,ext\,9} \tag{5}$$

Seismic axial force demands on columns can be easily calculated by summing the shear demands associated with flexural yielding of beams up the height of the frame. Note that seismic axial demand is only generated in columns located on axes A, C, E and G (see axis notation in Fig.2), and that verification of the columns must be carried out for seismic excitation in both directions, due to the asymmetry of the building.

An approximate capacity-design procedure from Priestley et al. (2007) is applied to assure that a weak beam – strong column mechanism is developed. For the calculation of capacity-protected actions, strain hardening of the provided beam resistances is neglected in view of the low ductility demands expected. Dynamic amplification of shear forces is also undertaken, but shear capacity of steel sections is rarely critical and thus does not represent an issue for this study.

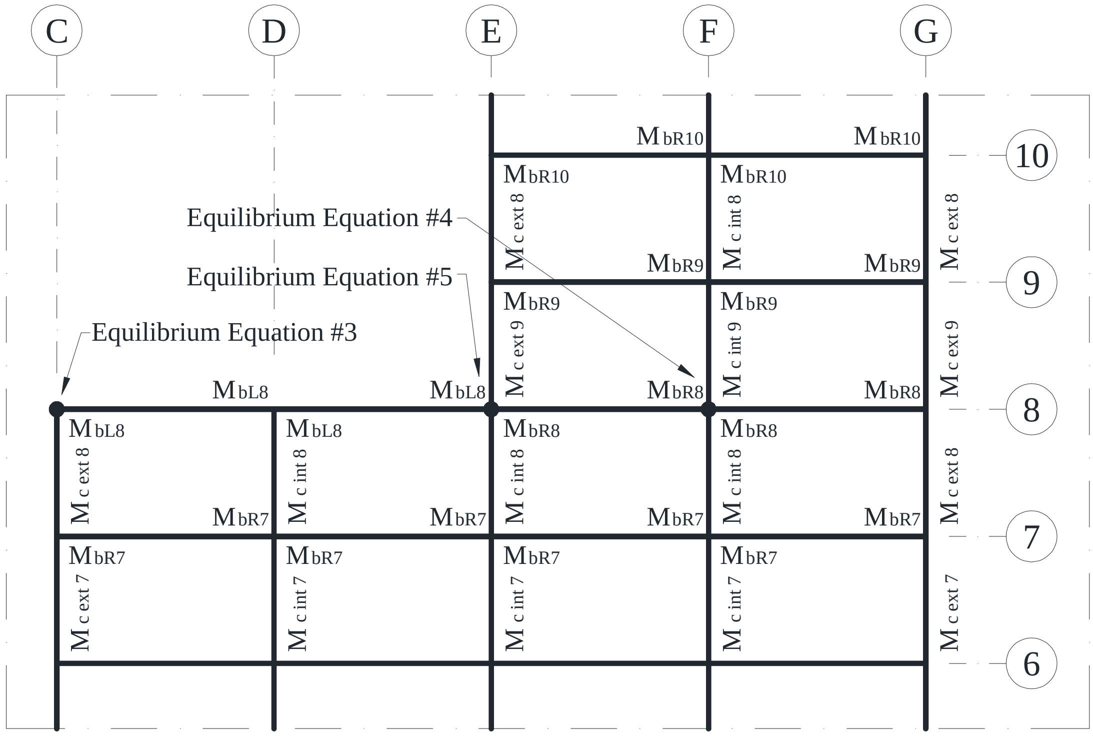


Figure 3. Detail of moment distribution at Level 8 of Frame A

The final adopted sections for the beams and the columns of Frame A and Frame B are summarized in Table.2 and Table.3. In both cases, beam section sizes do not follow the strength reduction with height expected for regular frames, due to three main reasons:

- The final storey yield shear profile ($V_{i,y}$) calculated by means of Eq.1 and Eq.2 turns out to be more uniform in height than the ductility-independent storey shear profile (V_i).
- The summation of the required plastic hinges strengths increases from top to bottom, but so does the number of bays at each setback level.
- Plastic hinges required strength is reduced for the beams of the first storey with the aim of facilitating the capacity design of ground storey columns. This is implicitly done by assuming that the latter present their contraflexure point at 0.6 times the interstorey height above the base level.
- The decision made regarding the distribution of strength at setback levels (as described earlier) leads to beams on the "right" of the setback to need to make up for beams on the "left" being smaller.

Table 2. Frame A: column and beam sections resulting from application of DDBD procedure

Storey	Column Sections							Beam Sections					
	A	B	C	D	E	F	G	A-B	B-C	C-D	D-E	E-F	F-G
12					HE320B	HE300M	HE320B					IPE400	IPE400
11					HE320B	HE300M	HE320B					IPE500	IPE500
10					HE280M	HE320M	HE280M					IPE550	IPE550
9					HE280M	HE320M	HE280M					IPE550	IPE550
8			HE300B	HE300M	HE300M	HE320M	HE280M			IPE360	IPE360	IPE550	IPE550
7			HE300B	HE300M	HE300M	HE320M	HE280M			IPE500	IPE500	IPE500	IPE500
6			HE340B	HE300M	HE360M	HE320M	HE300M			IPE500	IPE500	IPE500	IPE500
5			HE300M	HE320M	HE360M	HE320M	HE300M			IPE550	IPE550	IPE550	IPE550
4	HE300B	HE300M	HE300M	HE320M	HE360M	HE320M	HE300M	IPE360	IPE360	IPE500	IPE500	IPE500	IPE500
3	HE300B	HE300M	HE300M	HE320M	HE360M	HE320M	HE300M	IPE500	IPE500	IPE500	IPE500	IPE500	IPE500
2	HE320B	HE300M	HE300M	HE320M	HE360M	HE320M	HE300M	IPE500	IPE500	IPE500	IPE500	IPE500	IPE500
1	HE320B	HE300M	HE300M	HE320M	HE360M	HE320M	HE300M	IPE500	IPE500	IPE500	IPE500	IPE500	IPE500

Table 3. Frame B: column and beam sections resulting from application of DDBD procedure

Storey	Column Sections							Beam Sections					
	A	B	C	D	E	F	G	A-B	B-C	C-D	D-E	E-F	F-G
12					HE300B	HE300M	HE300B					IPE400	IPE400
11					HE260M	HE300M	HE260M					IPE500	IPE500
10					HE260M	HE300M	HE260M					IPE550	IPE550
9					HE260M	HE320M	HE260M					IPE550	IPE550
8					HE280M	HE320M	HE280M					IPE550	IPE550
7			HE300B	HE300M	HE300M	HE320M	HE280M			IPE360	IPE360	IPE550	IPE550
6			HE300B	HE300M	HE320M	HE320M	HE280M			IPE500	IPE500	IPE500	IPE500
5			HE320B	HE300M	HE320M	HE320M	HE300M			IPE500	IPE500	IPE500	IPE500
4			HE280M	HE300M	HE360M	HE320M	HE300M			IPE500	IPE500	IPE500	IPE500
3			HE280M	HE300M	HE400M	HE340M	HE300M			IPE550	IPE550	IPE550	IPE550
2	HE280B	HE280M	HE300M	HE300M	HE400M	HE340M	HE300M	IPE360	IPE360	IPE500	IPE500	IPE500	IPE500
1	HE280B	HE280M	HE300M	HE300M	HE400M	HE340M	HE300M	IPE450	IPE450	IPE450	IPE450	IPE450	IPE450

NON-LINEAR TIME-HISTORY ANALYSES OF CASE STUDY BUILDINGS

The performance of the DDBD solutions adopted for the two case study frames is evaluated by subjecting two-dimensional lumped-plasticity models of the structures to a series of non-linear time-history analyses run at increasing intensities. *Ruaumoko 3D* (Carr, 2011) software is used for this purpose, using a time step of 0.005 seconds, a large displacements analysis regime and a set of ten spectrum-compatible accelerograms selected by Maley et al. (2013) and listed in Table.4, scaled to match the design spectral shape at design and increasing intensity levels.

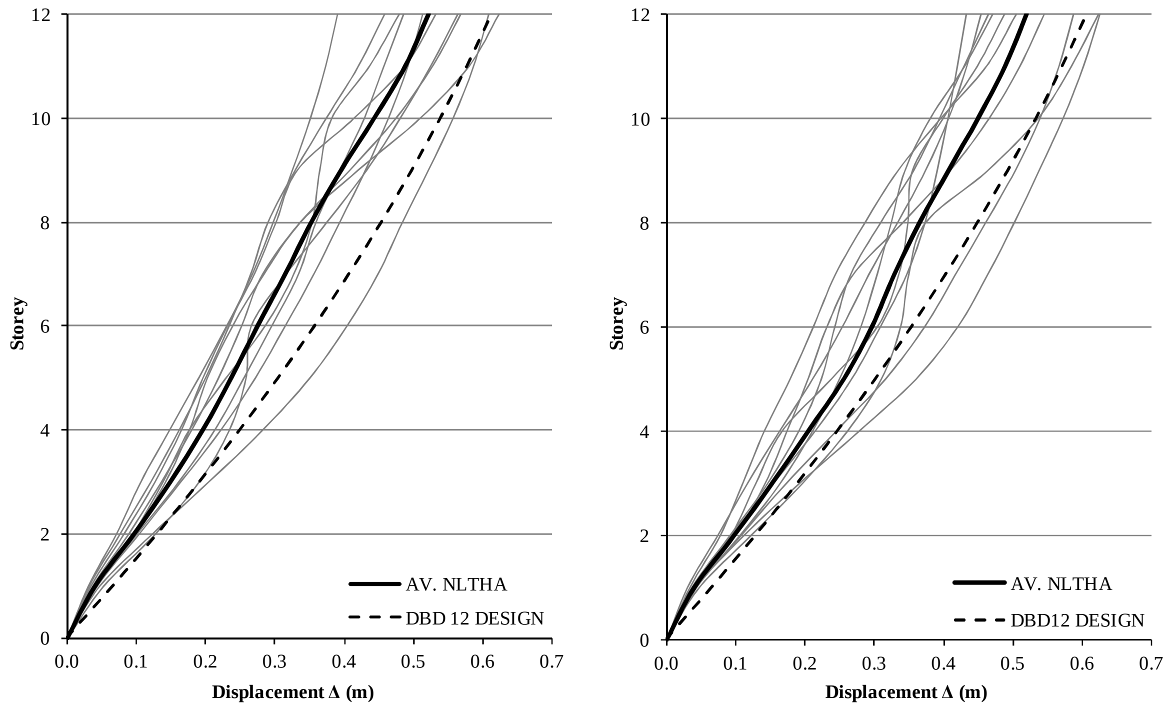
Table 4. Earthquake records used for non-linear time-history analyses

ID	Earthquake	Year	Mw	Station	Component
LA1	Denali, Alaska	2002	7.90	R109 (temp)	5596-090
LA2	Chi-Chi, Taiwan	1999	7.62	TCU085	TCU085-E
LA3	Chi-Chi, Taiwan	1999	7.62	TAP065	TAP065-N
LA4	Chi-Chi, Taiwan	1999	7.62	KAU003	KAU003-N
LA5	Darfield	2010	7.10	Rata Peats	E
LA6	Loma Prieta	1989	6.93	So. San Fr., Sierra Pt.	SSF115
LA7	Loma Prieta	1989	6.93	So. San Fr., Sierra Pt.	SSF205
LA8	Irpinia, Italy-01	1980	6.90	Auletta	A-AUL000
LA9	Northridge-01	1994	6.69	Sandberg - Bald Mtn	SAN180
LA10	Northridge-01	1994	6.69	Antelope Buttes	ATB000

All the beams and columns are modeled at their centrelines as one component Giberson members, and rigid end blocks are used at all member ends. Deformation in panel zones and connections is not modeled because full-strength rigid joints are assumed. It is believed that the possible overestimation of the frames' stiffness derived from neglecting panel zone deformations is counterbalanced by the fact that the gravity (non-seismic) system is not being modeled. However, future research could investigate the effects of more refined modeling assumptions. No axial force-bending moment rule is assigned to the beams, while an adaptation of the interaction surface developed by Chen and Atsuta (2008) is used for the columns. Masses are lumped at beam-column intersections, and a dummy gravity column is used to represent the additional mass carried by the gravity system. A tangent-stiffness based Rayleigh damping model is specified, in accordance with the suggestions by Priestley et al. (2007).

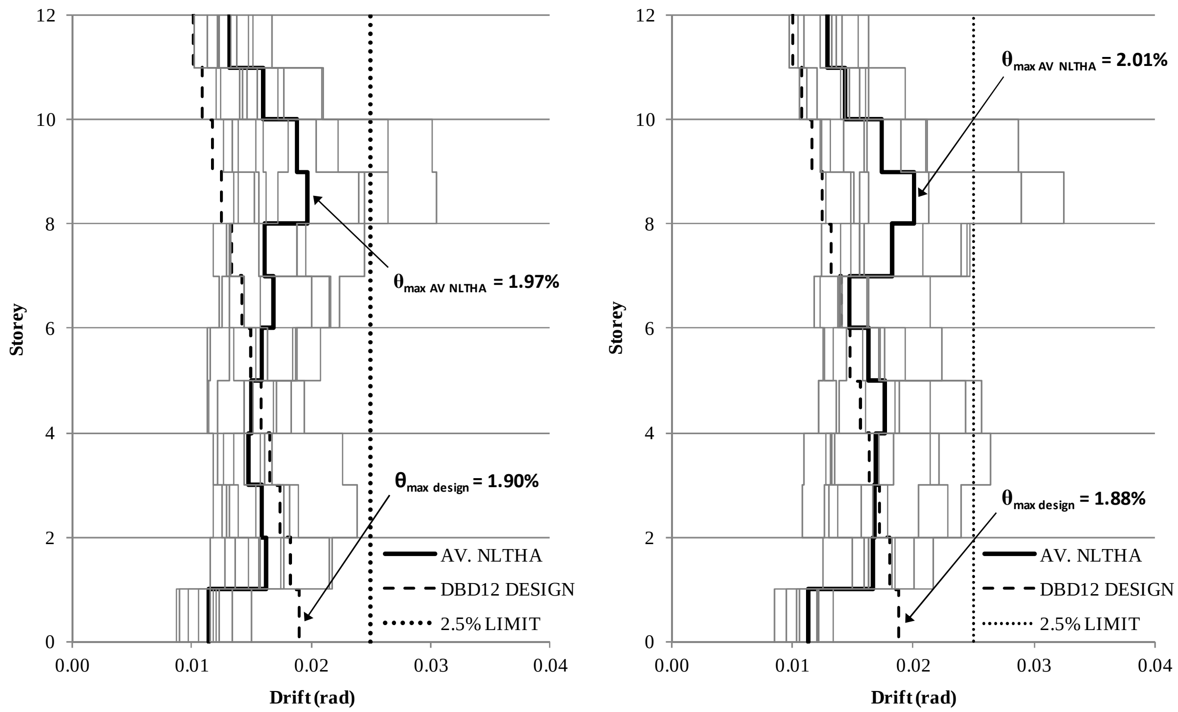
Fig.4 shows that the resulting displacements for the non-linear time-history analyses run at the design intensity level are, on average and also individually in their great majority, smaller than the design displacement profile. Fig.5, in turn, shows that the corresponding interstorey drifts easily meet the maximum drift criterion of 2.5%. However, it should be noted that this maximum design drift had actually been reduced to 1.90% (Frame A) and 1.88% (Frame B) due to P-Delta considerations, and therefore both frames actually present an amplification of the maximum design drift. Even though the problem is non-linear, an equivalent maximum drift ($\theta_{\text{equiv max av NLTHA}}$) that would have been obtained if both frames had been designed for a maximum drift of 2.28% (resulting from the product of 2.5% and a higher mode reduction factor ω_θ of 0.91, appropriate for a 12-storey frame according to DBD12) can be estimated as per Eq.6. This calculation results in an estimated 2.36% and 2.43% maximum interstorey drift for Frames A and B, respectively, which are clearly closer to the 2.5% limit.

$$\theta_{\text{equiv max av NLTHA}} = \omega_\theta \cdot \theta_c \cdot \frac{\theta_{\text{max av NLTHA}}}{\theta_{\text{max design}}} \quad (6)$$



(a) Frame A (b) Frame B

Figure 4. Envelopes of peak displacements at damage control (design) limit state. Light gray lines represent results from individual records, the thick solid line shows the average of envelopes from all records, and the thick dashed line shows the design displacement profile



(a) Frame A (b) Frame B

Figure 5. Envelopes of peak drifts at damage control (design) limit state. Light gray lines represent results from individual records, the thick solid line shows the average of envelopes from all records, and the thick dashed line shows the design drift profile

RELATION WITH PREVIOUS STUDY BY KARAVASILIS ET AL. (2008)

Non-linear time history analyses are conducted for the case-study structures not only at the design intensity level, but for a range of intensities as well, allowing for comparison with the work carried out by Karavasilis et al. (2008). Based on the results from over 14 thousand analyses, they developed an expression that relates the maximum interstorey drift to the maximum roof displacement, as a function of the number of storeys and the frames' geometry. The latter is characterized by two factors, Φ_s and Φ_b, as defined by Mazzolani and Piluso (1996), and computed by means of Eq.7 and Eq.8, with L_i, H_i, n_b and n_s as per Fig.6. The lower the factors Φ_s and Φ_b are, the closer the frame's geometry is to being regular, with 1.00 being the lowest possible value for either of the two.

$$\phi_s = \frac{1}{n_s-1} \cdot \sum_{i=1}^{i=n_s-1} \frac{L_i}{L_{i+1}} \tag{7}$$

$$\phi_b = \frac{1}{n_b-1} \cdot \sum_{i=1}^{i=n_b-1} \frac{H_i}{H_{i+1}} \tag{8}$$

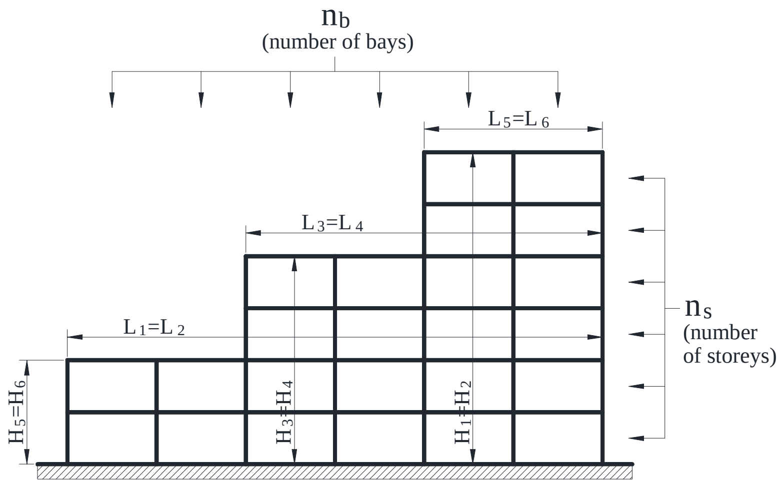


Figure 6. Definition of terms in Equations 7 and 8 (as in Karavasilis et al., 2008, and Mazzolani and Piluso, 1996)

Karavasilis et al. (2008) gave the name β to the ratio of the maximum roof displacement ($\Delta_{n\ max}$) to the product between the height of the frame (H_n) and the maximum interstorey drift ($\theta_{i\ max}$), as shown in Eq.9. By means of regression analysis, they found that this ratio is related to the number of storeys and the geometric indices through Eq.10 which, when applied to Frames A and B, yields β values of 0.507 and 0.491, respectively.

$$\beta = \frac{\theta_{roof\ max}}{\theta_{i\ max}} = \frac{\Delta_{n\ max}}{H_n \cdot \theta_{i\ max}} \tag{9}$$

$$\beta = 1.0 - 0.13 \cdot (n_s - 1.0)^{0.52} \cdot \phi_s^{0.38} \cdot \phi_b^{0.14} \tag{10}$$

Based on their series of analysis, Karavasilis et al. (2008) found that if the maximum interstorey drift is estimated using Eq.9 and Eq.10, with the maximum roof displacement known, then the median ratio of the approximate to real maximum interstorey drift was 1.00, and presented a standard deviation (defined in terms of their logarithmic ratio) of 0.17. This testing process is carried out for the case study frames of the present work and, even though the results are not smoothly distributed, it can be said that they appear to follow a distribution similar to that described by Karavasilis et al. (2008), with a median ratio of 1.05 and a standard deviation of 0.16. Fig.7 shows the number of repetitions obtained for ratio intervals of 0.05. The lack of smoothness can be easily explained by noting that only two geometrical configurations are being considered in the present study, while Karavasilis et al. (2008) considered a total of forty. It is also relevant to note that, following the suggestions of Karavasilis et al. (2008), results within the elastic range of response are excluded from this analysis.

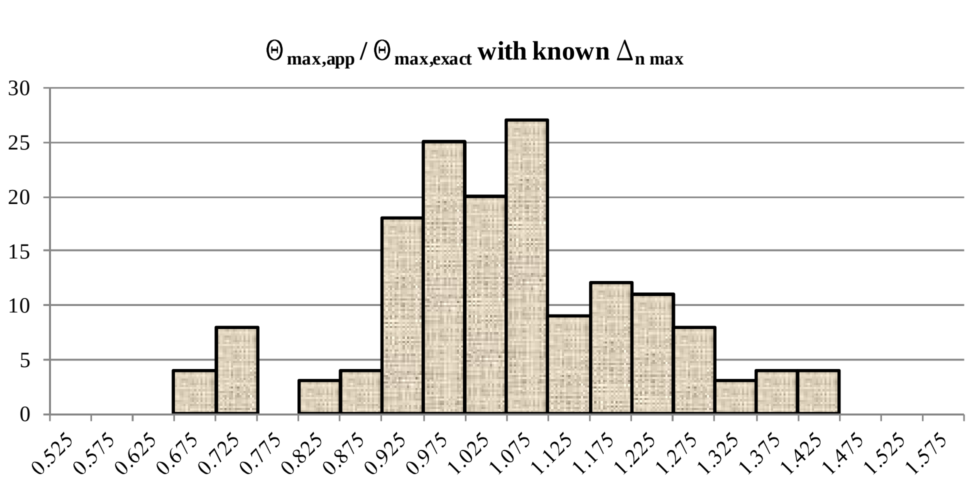


Figure 7. Distribution of approximate (Eq.9) to real maximum interstorey drifts for Frames A and B

MODIFICATIONS TO THE HIGHER MODE REDUCTION FACTOR

Eq.9 and Eq.10 can be thought of as being the higher mode reduction factor that should be used with a linear displacement profile, because the ratio of maximum roof displacement to the frame's total height would be the (constant) interstorey drift. Furthermore, Eq.9 and Eq.10 clearly show that the more irregular the frame is, the lower the β factor results and, therefore, the greater the amplification of the maximum interstorey drift obtained with respect to that corresponding to a linear profile. This is aligned with the results obtained for Frames A and B, for the former presents an amplification of its design drift of 3.8% and its corresponding β value is 0.507, while the latter presents an amplification of 7.0%, and its corresponding β value is 0.491, i.e. a greater amplification occurs for the frame with the lowest value of β. In this way, it can be assumed that irregular frames with the same number of storeys as Frames A and B but with bigger values of Φ_s and Φ_b would present higher amplification of drifts than the frames studied here, and thus the maximum drift of 2.5% could be exceeded in these cases. Based on this observation, Nievas and Sullivan (2014) propose that the higher mode reduction factor currently suggested for regular frames ($\omega_{\theta\ \mathrm{regular}}$) be scaled by the ratio of the β value of the irregular frame ($\beta_{\mathrm{setbacks}}$) and the β value that would correspond to a regular frame with the same number of storeys (β_{regular}), as shown in Eq.11.

$$\omega_{\theta\ \mathrm{setbacks}} = \omega_{\theta\ \mathrm{regular}} \cdot \frac{\beta_{\mathrm{setbacks}}}{\beta_{\mathrm{regular}}} \qquad (11)$$

The efficacy of Eq.11 is tested by Nievas and Sullivan (2014) who, by analyzing a broader series of frame configurations designed for three different intensity levels, arrive at the conclusion that a further adjustment is needed to account for the influence of the spectral shape, ductility demand and P-Delta effects on the amplification of interstorey drifts due to higher modes. In their work, the authors initially suspect a possible dependency of the expectable amplification on ductility demand, but then prove that this apparent dependency can be better explained in terms of the effective stiffness and, therefore, effective period of the system in relation to the characteristics of the ground motions' acceleration and displacement spectra.

CONCLUSIONS

This study has investigated the applicability of the DDBD method to steel moment resisting frames with setbacks, and has highlighted those aspects that are considered to require special attention. A special effort has been made to investigate the efficacy of higher mode reduction factors currently prescribed for regular frames which, thanks to a parallelism that can be established with a previous work conducted by Karavasilis et al. (2008), appear to be easily adjustable as a function of the frame's degree of irregularity. Furthermore, it is explained that more recent research by Nievas and Sullivan (2014) indicates that the higher mode amplification drift factor should also be adjusted to consider the shape of the design acceleration and displacement spectra, the ductility demand and the influence of P-Delta effects. The findings presented contribute significantly to the development of a more accurate way of accounting for higher mode effects in the design of steel moment resisting frames with setbacks.

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