DEVELOPING THE DIRECT DISPLACEMENT-BASED DESIGN METHOD FOR RC STRONG FRAME – WEAK WALL STRUCTURES

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ABSTRACT

The Direct Displacement-Based Design of RC frame-wall systems in which the walls carry a greater proportion of the overturning moment has been extensively studied in the past. However, as the authors of the present research continue their efforts to develop a software for the Direct Displacement-Based Design of structures (DBDsoft), they are faced with the need of assessing the behaviour of a wider range of frame and wall strength proportions, and venture into the development of expressions for the displacement profile of frame-wall systems in which the frames carry a greater proportion of the total overturning moment than the walls. The performance of a newly-proposed approach for this kind of system is gauged by means of designing eleven case study frame-wall structures of 4-, 12- and 20-storeys and subjecting them to a series of non-linear dynamic analyses. Results obtained are promising and suggest that the use of the newly proposed approach for the determination of the displacement profile might be adequate for the design of strong frame – weak wall structures. It is also concluded that DBDsoft is able to significantly ease the design process and provides interesting possibilities for the development of refined seismic design solutions.

INTRODUCTION

Frame-wall structures are advantageous seismic-resistant structural systems, for they combine the benefits of each of their components. While frames tend to concentrate deformations at the lower storeys, walls deflect more at their top, and hence their working together leads to better controlled displacements along the whole height of the structure. Dual structures in general, and frame-wall systems in particular, are especially well suited to be designed with the Direct Displacement-Based Design method, given the known difficulties in accounting for the different deformation and ductility capacities within force-based methodologies (Priestley et al. 2007).

Sullivan et al. (2006) have proposed design guidelines for frame-wall systems that could be regarded as “weak frame – strong wall”, for they have been tested for cases in which walls carry 50% or more of the total base shear. The proportion of lateral load carried by the frames and the walls has been found to play a very significant role in the behaviour of the system. Due to the reduced deformability of frames in the upper storeys, walls are subjected to reverse bending moments and, thus, a contraflexure point develops, as shown in Fig.1. The stronger the frames, the lower the resulting contraflexure height. Sullivan et al. (2004) have found that the displaced shape of frame-wall structures is a function of the height of contraflexure in the walls which, as said, depends on the frame and wall strength proportions. In their work, Sullivan et al. (2006) propose the use of an inelastic displaced shape in which a cubic and a linear profile are used below and above the contraflexure

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height, respectively, implicitly implying that the displacements at the bottom are mostly controlled by the stiffer wall and that, in the upper storeys, frames restrict the deformation.

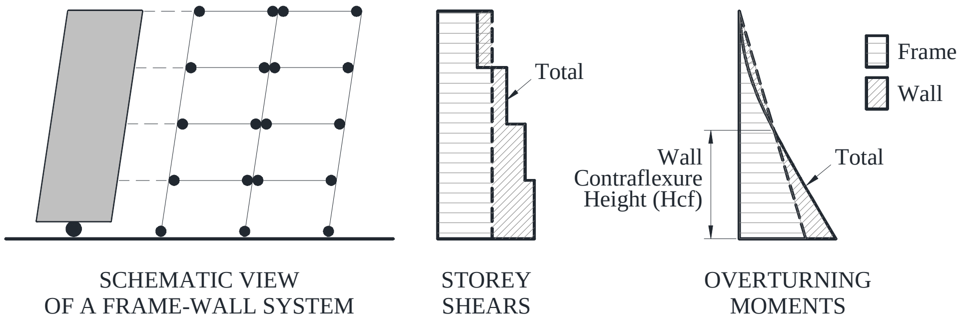

Figure 1. Relation between frame-wall strength proportions and the wall's contraflexure height

As the authors of the present research continue their efforts to develop a software for the Direct Displacement-Based Design of structures (Sullivan et al. 2014), they are faced with the need of assessing the behaviour of a wider range of frame and wall strength proportions, and venture into the development of expressions for the displacement profile of frame-wall systems in which the frames carry a significantly greater proportion of the total base shear than the walls.

PROPOSED MODIFICATION TO THE DESIGN DISPLACEMENT PROFILE

The work of Sullivan et al. (2006) suggests that the limit state displacement profile of a frame-wall system be calculated according to Eq.(1) and Eq.(2):

$$\Delta_{i,ls} = \Phi_{yW}\left(\frac{h_i^2}{2} - \frac{h_i^3}{6H_{cf}}\right) + \theta_{pFW}\cdot h_i \qquad \text{for } h_i \le H_{cf} \tag{1}$$

$$\Delta_{i,ls} = \Phi_{yW}\left(\frac{H_{cf}\cdot h_i}{2} - \frac{H_{cf}^2}{6}\right) + \theta_{pFW}\cdot h_i \qquad \text{for } h_i > H_{cf} \tag{2}$$

where Φ_{yW} is the yield curvature of the stiffest (usually the longest) wall, h_i and H_{cf} are the storey height and the contraflexure height, measured from the foundation level, and θ_{pFW} is the design plastic rotation, which can be controlled either by a non-structural interstorey drift limit θ_c or by limits imposed over material strains of the walls, expressed in the form of a limit state curvature Φ_{lsW}, as shown in Eq.(3):

$$\theta_{pFW} = \theta_c - \frac{\phi_{yW}\cdot H_{cf}}{2} \le \left(\phi_{lsW} - \phi_{yW}\right)\cdot L_p \tag{3}$$

where L_p is the plastic hinge length, approximated by Eq.(4), as suggested by Priestley et al. (2007):

$$L_p = k\cdot H_{cf} + 0.1\cdot L_w + L_{sp} \tag{4}$$

where k is a factor that depends on the ratio of ultimate (f_u) to yield (f_y) strength of the reinforcement, as shown in Eq.(5), L_w is the wall's length in the direction of analysis, and L_{sp} is the strain penetration length, estimated according to Eq.(6) as a function of the expected yield strength (f_{ye}) and the diameter (d_{bl}) of the reinforcement.

$$k = 0.15\cdot\left(f_u \;/\; f_y - 1\right) \le 0.06 \tag{5}$$

$$L_{sp} = 0.022\cdot f_{ye}\cdot d_{bl} \tag{6}$$

Eq.(1) and Eq.(2) lead to a displacement profile as the one shown in Fig.2(a). As it has been mentioned, the stronger the frames, the lower the contraflexure height results. As opposed to this shape, a pure frame tends to deform more at the bottom and less at the top, and hence a quadratic equation like the one suggested in the DBD12 Model Code (Sullivan et al. 2012) and transcribed here as Eq.(7) is more appropriate for its definition:

$$\Delta_{i,ls} = \theta_c \cdot h_i \cdot \frac{\left(4H_n - h_i\right)}{\left(4H_n - h_1\right)} \tag{7}$$

where H_n and h_1 are the total height of the building and the height of the first storey, respectively.

If a specific frame-wall system is considered, in which the proportions of overturning moment carried by the walls and the frames are continuously varied so that the frames take more and more of the total demand, a point will be reached in which the frames will resist the full seismic action, and thus the structure will be expected to behave similarly to a pure frame. It could thus be expected as well that there exists a transition range of proportions for which the system behaves somewhere in between. This is the underlying idea behind the approach to be proposed, and it is schematically represented in Fig.2.

Given the proven importance of the contraflexure height in the behaviour of frame-wall systems (Sullivan et al. 2006), the transition range will be characterized by the contraflexure height rather than the strength proportions directly. In contrast, the use of the proportion of base shear carried by frames and walls would be the least effective parameter to be used for this characterization, for it not only depends on the relative contribution of each sub-system to the global overturning moment, but it is also sensitive to the distribution of strength within the members of the frames.

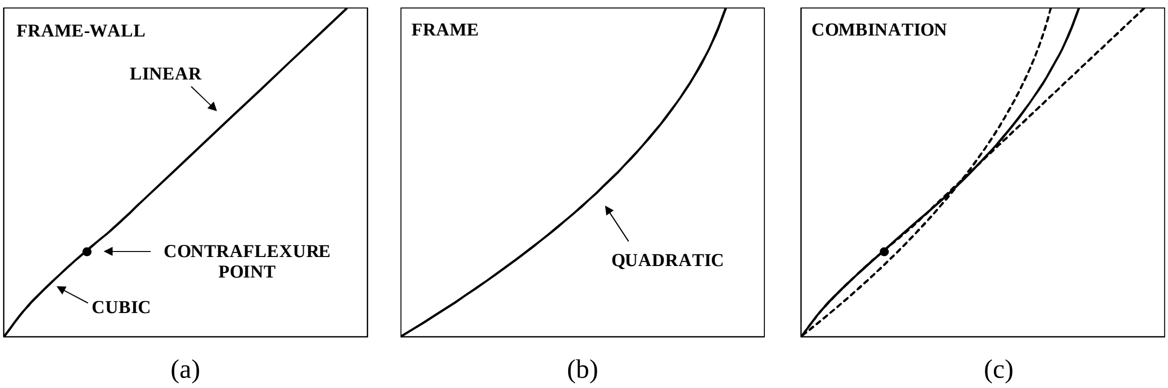

Figure 2. Limit state displacement profiles for (a) a frame-wall designed according to Eq.(1) and Eq.(2), (b) a pure frame, and (c) a strong frame - weak wall as proposed herein

In order to gauge the limits of the ranges for which the structure will be expected to behave closer to a pure frame, a strong frame-weak wall or a weak frame-strong wall, a series of 4-, 12- and 20- storey structures are designed with the DDBD method (Priestley et al. 2007, Sullivan et al. 2012) for a large range of moment-resisting strength proportions and for two different local strength distributions of beams and columns within the frames, using two types of displaced shapes:

- Cubic + Linear ("CL"): the "standard" weak frame-strong wall shape, defined by Eq.(1) and Eq.(2).

- Cubic + Quadratic ("CQ"): a combination of Eq.(1) up to the contraflexure point and a quadratic expression from this point onwards. For the latter, an adaptation of Eq.(7) is used, as shown in Eq.(8), where h_{av} is used instead of h_1, and it is calculated as the average storey height of the storeys above the contraflexure height:

$$\Delta_{i,ls} = \theta_c \cdot \left(h_i - H_{cf}\right) \cdot \frac{\left[4\left(H_n - H_{cf}\right) - \left(h_i - H_{cf}\right)\right]}{\left[4\left(H_n - H_{cf}\right) - h_{av}\right]} \quad (8)$$

All the frames are designed with the aid of DBDsoft (Sullivan et al. 2014), adapted where needed to allow for the use of the two alternative displacement profiles. Fig.3 presents the range of strength proportions (represented by the proportion of overturning moment carried by the frames, β_{fr}) and contraflexure heights investigated. As it can be observed, there is a discontinuity in the relation between these two parameters, given by the fact that the contraflexure height cannot lie within the top storey of the frame-wall structure.

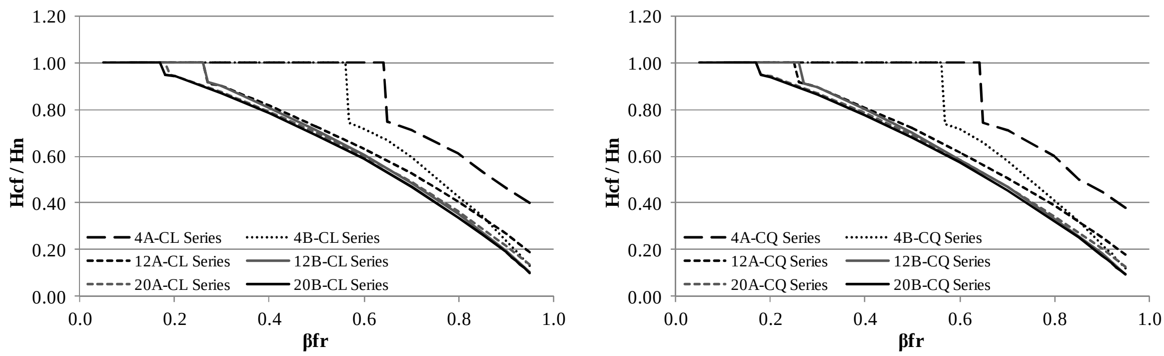

Figure 3. Range of contraflexure heights and strength proportions assessed within the first part of this study

Fig.4 shows the variation of the ratio of the design base shear obtained using the Cubic + Quadratic profile to that obtained using the Cubic + Linear profile, with respect to the contraflexure height. As it can be observed, the variation increases as the contraflexure height goes lower, that is, as the frames carry a greater proportion of overturning moment.

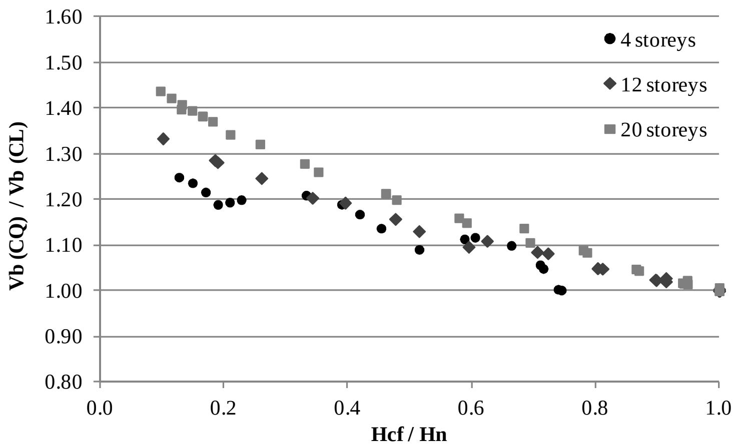

Figure 4. Ratio of design base shear values obtained using the CQ and the CL profiles, as a function of the ratio of the contraflexure height to the total height of the buildings designed within the first part of this study

Based on the results from this preliminary study, it is proposed that the following approach be followed to determine the kind of displacement profile to be used, as a function of the contraflexure height:

- For values of H_{cf} / H_n smaller than or equal to 0.2, use the Cubic + Quadratic profile.
- For values of H_{cf} / H_n greater than or equal to 0.5, use the Cubic + Linear profile.
- For values of H_{cf} / H_n in between 0.2 and 0.5, used a combined profile, as follows:

$$\Delta_{i,ls\ combined} = F_{CL} \cdot \Delta_{i,ls\ CL} + F_{CQ} \cdot \Delta_{i,ls\ CQ} \quad (9)$$

$$F_{CL} = \frac{10}{3} \cdot \left(\frac{H_{cf}}{H_n} - 0.2 \right) \quad (10)$$

$$F_{CQ} = 1 - F_{CL} \quad (11)$$

where $\Delta_{i,ls\ CL}$ and $\Delta_{i,ls\ CQ}$ are the Cubic + Linear and Cubic + Quadratic displacement profiles, respectively, and F_{CL} and F_{CQ} are their respective weighting factors.

Since the location of the contraflexure height is a function of the moment profile of the walls which is, in turn, dependent on the lateral force distribution and, thus, the displacement profile, an iterative procedure is adopted here since the program DBDsoft (Sullivan et al. 2014) is used for design. If hand calculations are to be done, an approximate non-iterative approach could be used, as described in Sullivan et al. (2006).

PERFORMANCE OF THE PROPOSED DESIGN DISPLACEMENT PROFILES

The performance of the proposed displacement profiles of new structures for DDBD is gauged by means of designing eleven case study frame-wall structures of 4-, 12- and 20-storeys and subjecting them to a series of non-linear response history (NLRH) analyses. The structures are designed according to the DBD12 Model Code (Sullivan et al. 2012) for the damage control limit state, assuming a 2% maximum drift limit for non-structural elements, a 10% ultimate strain limit for the longitudinal steel reinforcement of the walls, and a limit of 0.3 for the P-Delta stability coefficient calculated on a storey-by-storey basis. The elastic design displacement profile (Fig.5) is constructed based on a Eurocode 8 type I spectrum for soil type C, with a corner period (T_D) of 8 seconds, and peak ground acceleration on rock of 0.4g, and 10 selected natural accelerograms (Maley et al. 2013) are scaled to match, in average, this displacement profile. An expected concrete compression strength of 30 MPa and an expected yield strength of the flexural reinforcement of 400 MPa are used. Elastic moduli of concrete and steel are assumed to be 27.4 GPa and 200 GPa, respectively.

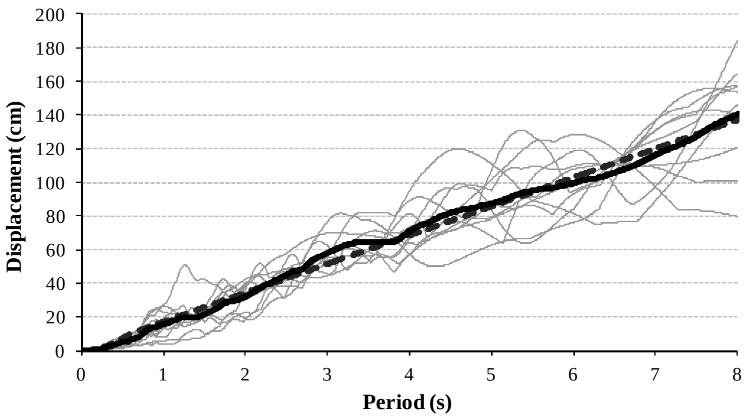

Figure 5. 5% elastic damping design displacement spectrum (thick gray dashed line), scaled 5% elastic damping displacement spectra of the 10 records used for the NLRHAs (thin gray lines), and average spectrum of all scaled records (thick black solid line)

The case-study frame-wall structures are considered to be part of a floor plan similar to the one shown in Fig.6, in which lateral resisting systems are assumed to be independent in the two perpendicular horizontal directions. Full rigid diaphragm action is assumed for each floor slab, and a seismic mass of 1900 t is considered to be present at each storey. Due to symmetry, torsion is deemed as negligible and thus the total mass is distributed equally between the two frame-wall systems marked in Fig.6. Columns and beams shown in Fig.6 other than the frame-wall systems under study are assumed to carry only gravity loads (and thus are designed simply to “follow” the movement of the building during a seismic event but not to resist its actions). A constant inter-storey height of 3.6 m is considered. Section dimensions for the case study structures are shown in Table.1.

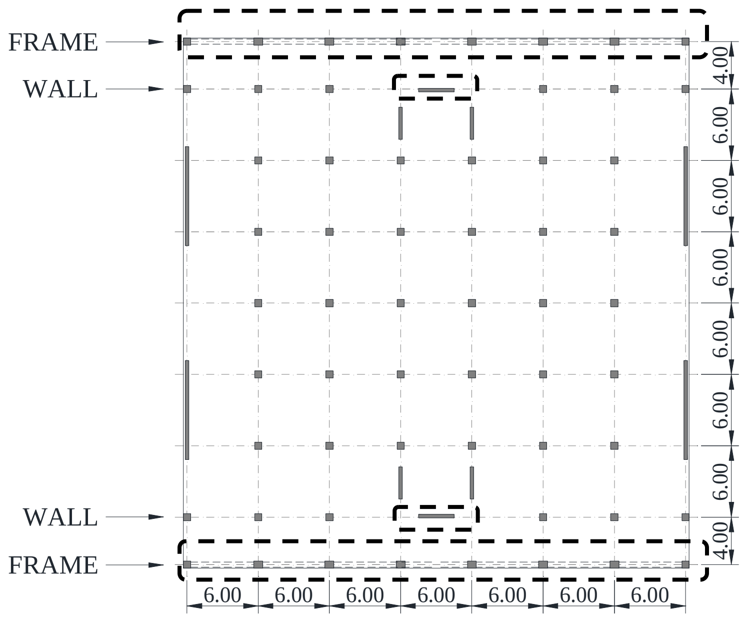

Figure 6. Floor plan of case-study structures

Design base shear demands and local strengths are obtained for all case-study structures following the recommendations of the DBD12 Model Code (Sullivan et al. 2012), with the aid of DBDsoft (Sullivan et al. 2014). Table.2 shows the main design parameters for each case. The first and second rows show the proportion of overturning moment ($\beta_{fr\ (OTM)}$) and base shear ($\beta_{fr\ (Vb)}$) carried by the frames, which are chosen at the beginning of the design process. Note that these values are not the same, for the latter depends as well on the relative strength distribution among the bases of the ground floor columns and the beams within the frame. The third and fourth rows present the maximum design drift ($\theta_{max\ des}$), and the factor that dominates the definition of the latter: the limit state curvature of the wall (WC, calculated from the approximate expressions of Priestley et al. 2007), the 2% non-structural drift limit (NSD), or P-Delta effects (P-Δ). Note that these values include the reduction to account for higher mode effects, as suggested by the DBD12 Model Code (maximum drifts are reduced by 1.00, 0.91 and 0.85 for the 4-, 12- and 20-storey structures, respectively). Rows 5 to 8 present the ratio of the contraflexure height to the total height of the building (H_{cf}/H_n), the equivalent viscous damping of the system (EVD), the design effective period (T_e), and the design base shear (V_b), already adjusted to account for P-Δ effects. Note that these values correspond to only one frame-wall system per structure (i.e. half of the storey plan).

Table 1. Dimensions of members (width *w*, depth *d*, length *l*, all in mm) for the case-study structures

Case		4-storey frames			12-storey frames				20-storey frames			
		4-A	**4-B**	**4-C**	**12-A**	**12-B**	**12-C**	**12-D**	**20-A**	**20-B**	**20-C**	**20-D**
beam	**w**	450	450	450	550	550	450	450	450	450	450	450
	d	950	850	750	950	850	850	850	850	850	750	750
roof beam	**w**	400	350	400	550	450	400	350	350	300	400	350
	d	950	850	750	950	850	850	850	850	850	750	750
int. col. base	**w**	600	600	450	500	500	500	450	600	500	500	450
	d	950	850	850	950	750	750	750	650	650	600	600
ext. col. base	**w**	700	600	600	600	500	500	450	500	450	450	450
	d	700	600	600	600	600	600	600	500	450	450	450
wall base	**w**	300	350	350	300	350	350	350	300	300	300	300
	l	3500	3500	4000	6000	6000	6000	7000	6000	8000	8000	9000

Table 2. Design parameters of the case-study structures and estimated required nominal flexural strength of members at columns centrelines (for the case of beams) and columns' and walls' bases

Case		4-storey structures			12-storey structures				20-storey structures			
		4-A	**4-B**	**4-C**	**12-A**	**12-B**	**12-C**	**12-D**	**20-A**	**20-B**	**20-C**	**20-D**
$\beta_{fr\,(OTM)}$		89.9%	81.8%	72.3%	89.6%	83.7%	73.8%	69.2%	88.9%	79.6%	70.5%	64.8%
$\beta_{fr\,(Vb)}$		74.1%	61.9%	53.8%	62.2%	55.7%	51.9%	49.8%	70.8%	61.5%	56.6%	54.2%
$\theta_{max\ des}$		1.58%	1.82%	1.82%	1.41%	1.72%	1.82%	1.82%	1.34%	1.31%	1.37%	1.39%
		WC	WC	WC	WC	WC	NSD	NSD	P-Δ	P-Δ	P-Δ	P-Δ
H_{cf} / H_n		0.288	0.394	0.508	0.187	0.273	0.420	0.487	0.204	0.344	0.464	0.535
EVD (%)		15.08	15.68	15.52	13.46	14.61	15.28	15.45	11.53	12.09	11.47	11.65
Te (sec)		1.39	1.69	1.79	3.07	4.05	4.59	4.66	6.64	7.04	7.22	7.30
Vb (kN)		9956	7916	7392	14132	10379	8783	8515	8090	7196	6884	6693
Required Nominal Strength (kNm)	**beam**	1509	1161	960	2085	1445	1089	995	1203	985	849	762
	roof beam	898	601	557	1443	964	645	512	629	435	483	383
	int. col.	1842	1035	830	2060	1272	1056	1002	1685	1276	1159	1117
	ext. col.	978	600	481	931	625	519	494	669	497	474	451
	wall	8388	12468	17794	36154	43038	62467	70782	40332	68934	99196	115340

Rows 9 to 13 of Table.2 show the required nominal flexural strengths of the members of each case-study structure, calculated in an approximate fashion, assuming a storey post-yield force-displacement stiffness ratio of 0.05. Reinforcement ratios for the sections are not reported, because design solutions are only developed to the point that non-linear models can be constructed for non-linear response history (NLRH) analyses. Note that, if reinforcement contents were to be checked, section sizes might need to be adjusted to respect code reinforcement and axial load ratio limits.

Two-dimensional models of the case-study structures are developed using Ruaumoko 3D (Carr 2011). All elements are modelled at their centrelines, using one component Giberson elements for the beams, columns at the ground floor and walls at the ground floor, and using elastic elements for the rest of the members which are expected to behave elastically due to capacity design provisions. Beams are modelled up to the columns' centrelines, without rigid offsets or specific modelling of panel zone deformations, as recommended by Priestley et al. (2007). Plastic hinge regions are assigned the moment capacity corresponding to the values shown in Table.2, assuming that reinforcement quantities would have been provided to exactly match the design actions, and a moment-curvature post-yield stiffness ratio determined from basic lumped plasticity mechanics theory, assuming a storey post-yield force-displacement stiffness ratio of 0.05. The length of the plastic hinges is estimated with the approximate formulae of Priestley et al. (2007). Distributed loads over beams and columns, and point loads over walls and a dummy gravity column (used to include the gravity load acting over non-modelled gravity columns) are used. Bending moment-axial load interaction is not modelled in the columns, as Priestley et al. (2007) report that the variation in flexural strength that occurs in columns due to varying seismic axial loads does not have a significant influence on the system response, which is the focus of this study. The inertia of the elements is calculated based on the nominal moment, the elastic modulus of the concrete and the approximate yield curvature, estimated using the expressions of Priestley et al. (2007).

The hysteretic behaviour of plastic hinges is characterized by the Takeda model (Otani 1981), with a post-yield stiffness ratio of 0.05 and the unloading model of Emori and Schonbrich (1978). An unloading stiffness factor (α) of 0.50 is used for walls and columns, while a factor of 0.25 is used for beams. A reloading stiffness factor (β) of zero and a reloading power factor of 1.0 are used for all members. Tangent stiffness proportional Rayleigh damping is used with 5% elastic damping specified for the second mode, while an artificially low elastic damping is estimated as per Priestley et al. (2007) as a function of the system's ductility to be specified for the first mode, with the aim of avoiding over-damping of higher modes of vibration. All the analyses are run with allowance for large displacements, using the Newmark constant average acceleration integration method, with an integration step of 0.005 seconds.

Given the relevant number of design factors that are taken into consideration in the present study (higher mode effects, P-Delta instability), results from the NLRH analyses need to be assessed from multiple perspectives. The simplest outcome is obtained for the 4-storey structures, for which neither higher modes nor P-Delta effects represent a problem. As it is exemplified in Fig.7 for the case of building 4-B, the proposed displacement profile is able to predict the displacements and effectively control the interstorey drifts for this kind of structure.

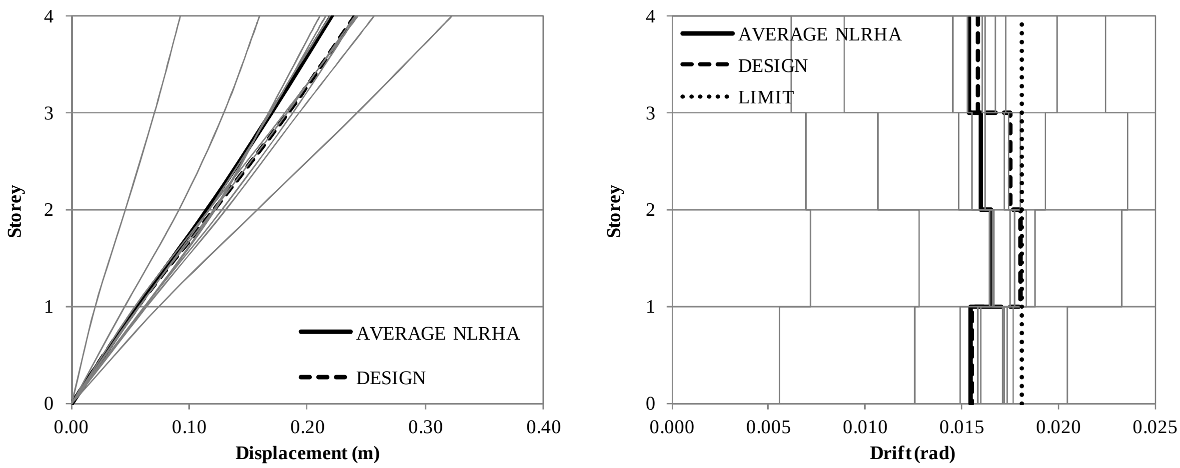

Figure 7. Peak displacements (left) and peak inter-storey drifts (right) for building 4-B: thin gray lines represent results from individual records, thick solid lines show their average, and thick dashed lines show the design profiles

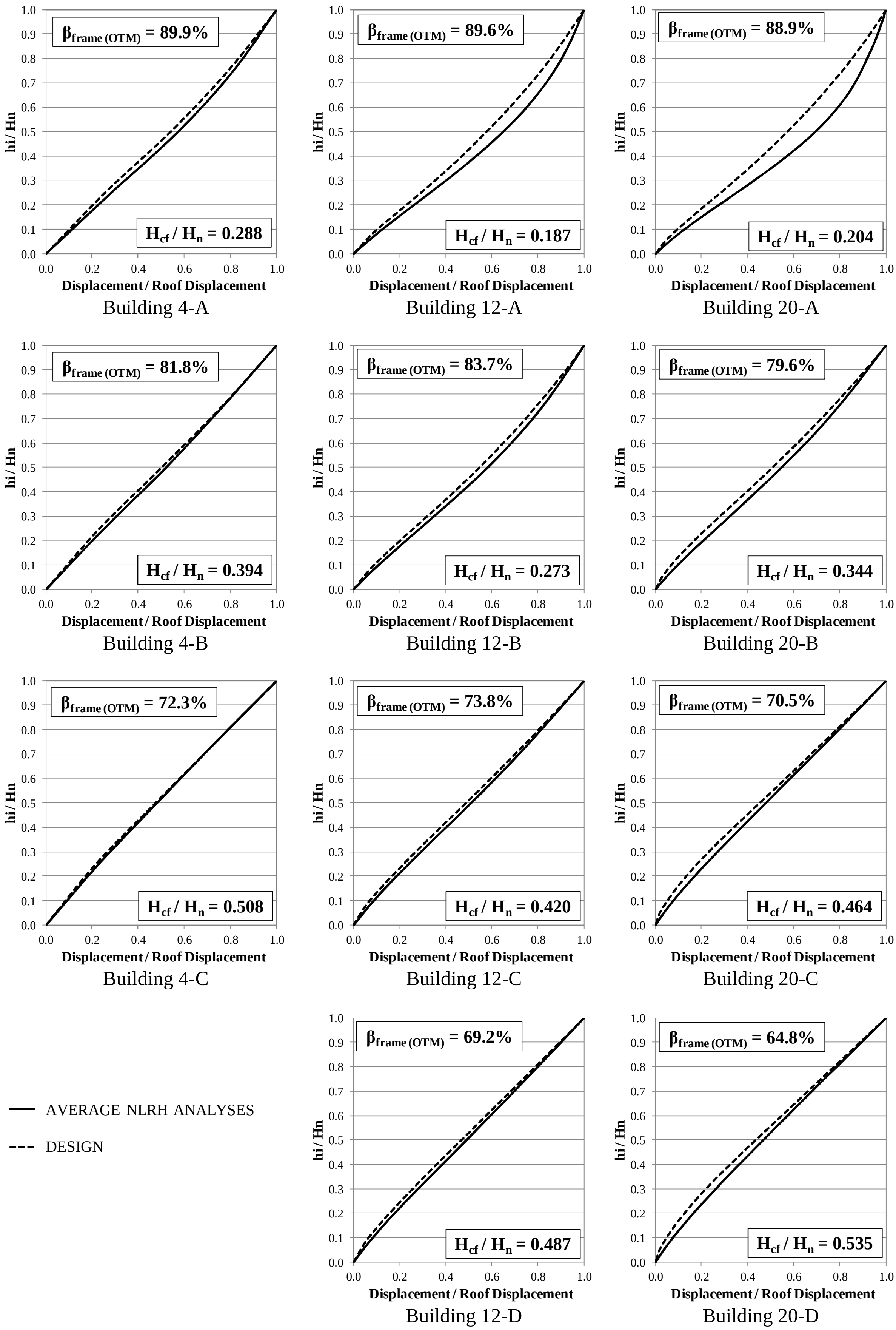

Figure 8. Design and average peak displacements from NLRH analyses, normalized with respect to the corresponding roof displacement

Fig.8 shows the design and average peak displacement profiles from NLRH analyses for the various case-study structures, normalized with respect to their respective roof displacements, to ease the comparison process. Observe that good matches are found in all cases with values of H_{cf}/H_n between 0.27 and 0.54, but not for buildings 12-A and 20-A, whose H_{cf}/H_n ratio is around 0.20. Observation of the average peak displacement profiles of these two structures shows that the average behaviour from the NLRHAs is quite close to that of a pure frame, as expected. However, even the expression for pure frames (Eq.(7)) cannot accurately represent it, as shown with the gray line in Fig.9 (left), though it could certainly describe better the higher drifts at the bottom storeys. Fig.9 (right) shows the inter-storey drifts obtained for building 12-A. It is clear that the maximum drift limit (corresponding, in this case, to the limit state wall curvature) is exceeded, which can be attributed to the limited accuracy of the displaced shape equation. The possibility of directly using Eq.(7) for frame-wall structures with a theoretical contraflexure height ratio smaller than 0.20 could be considered as part of future research.

Despite the good match between the normalized displaced shapes shown in Fig.8, maximum design interstorey drifts are (to a smaller or greater extent) exceeded for all the 12- and 20-storey buildings, possibly due to P-Delta amplification of displacements. The interstorey drifts obtained for the analyses run (for comparison) without allowance for large displacements are found to be significantly smaller. For this reason, it is believed that more attention should be paid to the influence of P-Delta effects, as will be elaborated upon in the upcoming section.

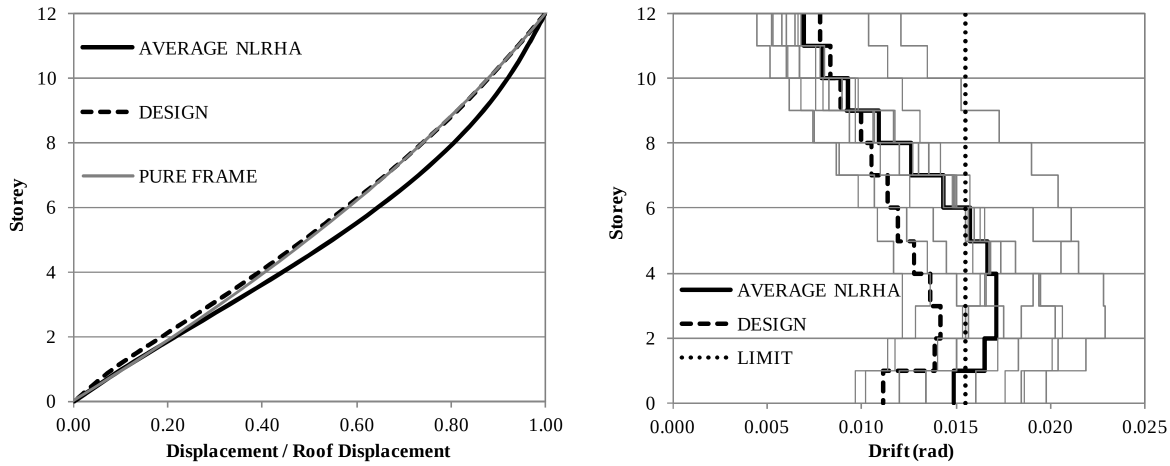

Figure 9. Design and average peak displacements, normalized with respect to roof displacement (left), and peak inter-storey drifts (right) for building 12-A: thin gray lines represent results from individual records, thick solid black lines show their average, and thick dashed lines show the design profiles

HIGHLIGHTING THE APPARENT IMPORTANCE OF P-DELTA EFFECTS

P-Delta effects prove to have a significant influence in the results obtained for all the 12- and 20-storey structures. Collapses are observed for one record for buildings 20-A and 20-B, clearly attributable to P-Delta dynamic instability. Fig.10 shows the effect of P-Delta amplification of drifts over some of the case-study structures, by comparing the average interstorey drifts obtained from the NLRH analyses run with and without allowance for large displacements. It is relevant to notice that, while studies related to other types of structures (Maley et al. 2013) have found the current approach to avoid P-Delta amplification (DBD12, Sullivan et al. 2012, Priestley et al. 2007) to be conservative, this does not appear to be the case for the structures investigated in the present study. This suggests that further research is required on the control of P-Delta effects for frame-wall structures, as well as on the complex interaction between P-Delta, higher modes and spectral shape (as highlighted, for example, in Nievas and Sullivan 2014).

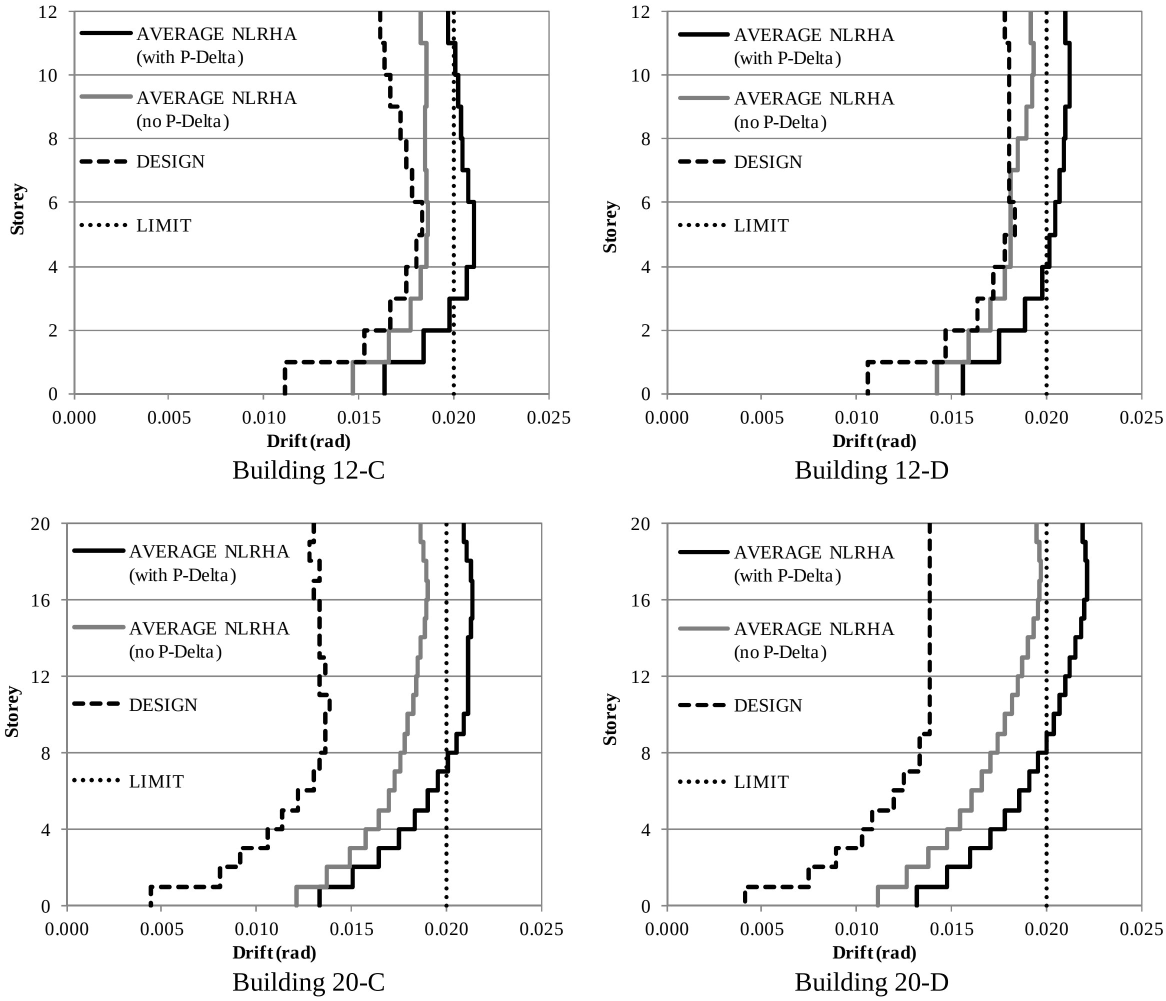

Figure 10. Peak inter-storey drifts for buildings 12-C, 12-D, 20-C and 20-D: solid black lines and solid gray lines show the average of envelopes for the NLTHAs run with and without allowance for large displacements, respectively, and dashed lines show the design profiles

CONCLUSIONS

This paper has proposed and evaluated an adaptation of the displacement profiles to be used for the design of RC strong frame – weak wall structures. A series of case-study buildings have been designed and subjected to NLRH analyses for this purpose. DBDsoft (Sullivan et al. 2014) has proven to be able to significantly ease the design process and provide interesting possibilities for the development of refined seismic design solutions.

Results obtained are promising and suggest that the use of the newly proposed displaced shape expressions might be adequate for the design of frame-wall systems in which frames carry a large proportion of the seismic load. However, a combination of additional factors such as higher mode effects and P-Delta amplification has been shown to significantly influence the response of this kind of system. Further research should focus on extending the number of case-study structures and adopting more refined modelling techniques, with the aim of better understanding the interactions between these various factors that can influence the response of frame-wall structures and arriving at conclusive recommendations for design.

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