# Adapting DDBD for the design of Frame-Wall Structures with Hybrid-Rocking Joints

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**SUMMARY:**
Recent and ongoing studies of the Direct Displacement-Based Design (DDBD) method for seismic design of structures have shown the satisfactory performance of the approach for a wide range of structural configuration and materials. In parallel to this, new technologies are emerging, such as the PRESSS technology that utilises hybrid-rocking joints to damp energy and minimise residual deformations. The objective of this research is to identify how the existing DDBD methodology for traditional RC Frame-Wall systems can be adapted for the design of frame-wall structures with hybrid rocking joints at beam-column joint locations and at wall-bases. This paper outlines the fundamentals of the procedure and the proposed methodology is explained through a 10-storey design example. The performance of the design solution is gauged by running non-linear time-history (NLTH) analyses with spectrum-compatible accelerograms. The results of NLTH analyses indicate that the methodology provides very good control of peak displacements and storey drifts.

*Keywords: Direct Displacement-Based Design; Seismic Design; Dual System, Frame-Wall, Hybrid-Rocking Joints, Performance-Based Design, PRESSS.*

## 1. INTRODUCTION

With the development of Performance-Based Engineering and its application around the globe, high performance structural system and new technologies are emerging. Earthquake engineering should go towards reliable analysis methods that provide safe designs complying with the needed and expected performance that society demands. There is an on-going debate about the performance parameters that the design should control and currently the most accepted parameter worldwide appears to be the inter-storey drift. However, in the last decade or so, the engineering community has also accepted the importance of residual deformation and floor accelerations as performance parameters for buildings.

A structural system that traditionally has been widely utilized to resists lateral loads is the frame-wall system (also known as a Dual System). A conventional reinforced concrete (RC) frame-wall structure with monolithic joints can provide in general, good drift control due the stiff nature of the cantilever wall as well as significant dissipation of energy through the ductile frames. Performance-based design guidelines for RC frame-wall structures have been provided by Sullivan et al (2006) and analytically tested with success.

At the same time, the Precast Seismic Structural System (PRESSS) provides a high performance system to be used in high seismic hazard zones due to its self-centring and energy dissipation capability (Priestley et al. 1999). Due to the concentrated gap opening, the damage to structural elements is generally less than an equivalent conventional RC system and given that the system is self-centring, normally, the residual deformations are negligible.

Frame-wall structures present the peculiarity that both components of the systems have a different lateral deformation mechanism under lateral load. In structures where the floor area is constrained by rigid diaphragms, both components will deform following the same displacement shape. This characteristic has an important impact on the analysis and design procedure. Another important characteristic that makes frame-wall systems different than other structural typologies, is the fact that given that there are two different components supporting the lateral loads, the shear distribution between the components will vary during different stages of load, and will definitely be different in the linear and in the non-linear range.

On the other hand, the PRESSS system (with hybrid rocking moment-resisting connections) uses a combination of post-tensioned steel (PT steel) and ductile mild steel so that the PT steel helps to re-centre and diminish residual displacements and the mild steel works to provide energy dissipation. Depending on the PT-steel / mild-steel ratio, the performance of the joint (and the structure) will vary between increased re-centring capabilities or increased energy dissipation capacity.

A frame-wall structure with hybrid rocking joints in the frames and at the wall bases could result in a very efficient system with considerable energy dissipation and drift control through the optimization of the PT-steel – mild-steel ratio. **Figure 1.1** shows an example of frame-wall structure with hybrid-rocking joints and a representation of its deformed shape.

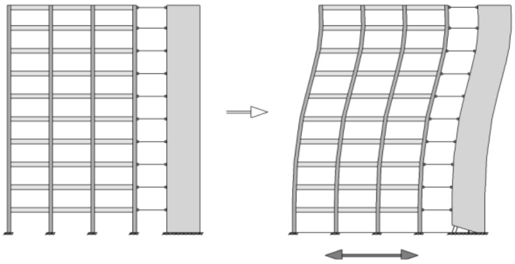


**Figure 1.1.** Frame-wall structure (left) and deformed frame-wall system with hybrid rocking joints (right).

Direct Displacement-Based Design from Priestley et al (2007) presents a comprehensive design method that aims to ensure that the required or needed performance will be fulfilled. Furthermore, since a rocking mechanism can be directly related to rotation, element deformations and material strains, it is well suited to a displacement-based design procedure like DDBD, where design deformation is the starting point of the procedure. The approach also gives the opportunity to arbitrarily assign the lateral strength distribution of the different system components (frames and walls) by the designer from the very beginning (see, Paulay 2002, Sullivan et al 2005 and Sullivan et al 2006).

## 2. PROPOSED METHODOLOGY

The methodology proposed for the seismic design of frame-wall structures is based on the DDBD method proposed by Sullivan et al (2006). The fundamentals of the DDBD method are shown in **Figure 2.1**.

$$\Delta_D = \frac{\sum_{i=1}^{n} m_i \Delta_{Di}^2}{\sum_{i=1}^{n} m_i \Delta_{Di}} \qquad m_e = \frac{\sum_{i=1}^{n} m_i \Delta_{Di}}{\Delta_D} \qquad H_e = \frac{\sum_{i=1}^{n} m_i \Delta_{Di} H_i}{\sum_{i=1}^{n} m_i \Delta_{Di}}$$

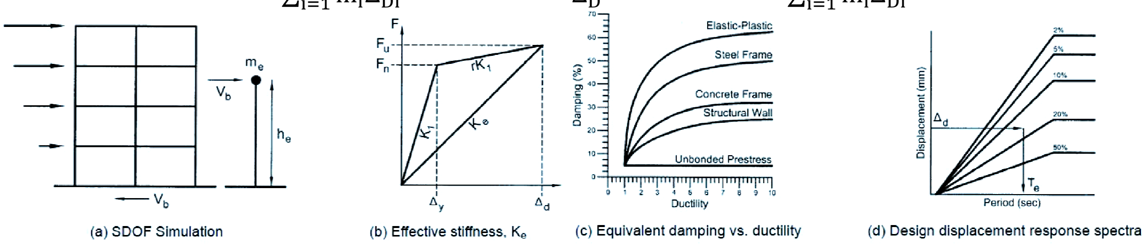



**Figure 2.1.** Fundamentals of Direct Displacement-Based Design (Priestley et al. 2007)

As pointed by Paulay (2002) the design strength for a mixed system can be decided at the beginning of the design process. **Figure 2.2** shows a simplified flowchart of the method with the main steps to be followed. The details of the method are explained in the next section with reference to a design example.

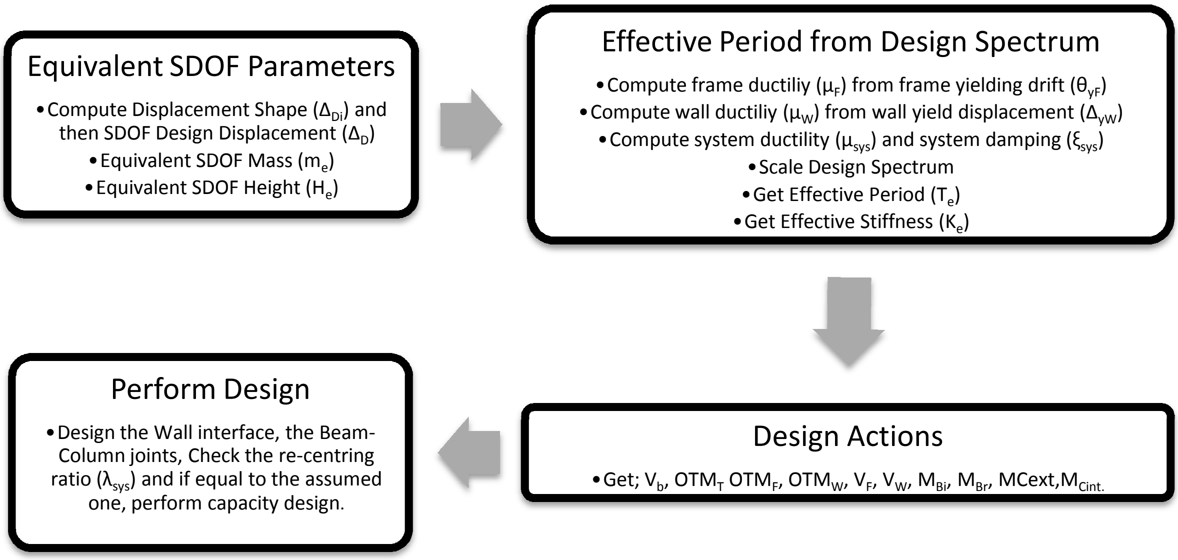



**Figure 2.2.** Flowchart of the proposed method.

## 3. DESING EXAMPLE

In order to show how the methodology can be applied to a building a design example is presented. The structure is a 10-storey office building where the lateral load resisting system is formed by two frame-wall systems at the building perimeter, as shown in **Figure 3.1**. Floor diaphragms are assumed to be stiff and resistant enough and with adequate system connections to transfer the entire floor inertia forces to the perimeter frame-wall system.

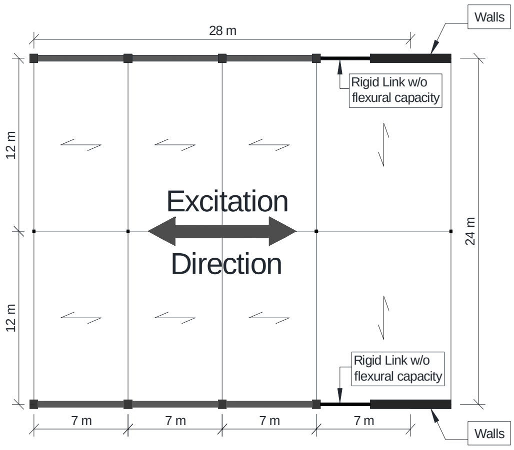
The structure under consideration is regular in plan and elevation. The inter-storey height is 3.8m for all levels; the seismic masses are 530T per storey except at the roof level which has a seismic mass of 450T. Since there are two parallel systems and a rigid diaphragm, each frame-wall system takes half of the total floor mass. The materials used for the design are shown in **Figure 3.1** and correspond to values typically found in building practice.

The building is designed using a Linear Displacement Spectrum for soil C of the EC8 (CEN 2004). The spectrum is linear up to 8s (corner period). An $a_g$ of 0.4g is selected (following the EC8) a displacement at 8s equal to 1.37m for the displacement design spectrum.

### 3.1. Initial design decisions

Three design choices should be made at the beginning of the design method; a) Design Drift Limit, $\theta_d$; b) Frame overturning-moment ratio, $\beta_F$ and c) the system's Re-centring Ratio, $\lambda_{sys}$. The Design Drift Limit is normally dictated by a National Code to ensure strain limits and/or non-structural limit states and for this case study a limit of 2.0%. The Frame Overturning-Moment (OTM) Ratio corresponds to the percentage of the total overturning moment resisted by the frames and it was selected as 0.50 (i.e. 50% of the total OTM will be support by the frames). A minimum value for the System Re-centring Ratio can be specified in a National Code in order to avoid residual deformations and following the state-of-the-art practice a value of 1.25 was selected.

| Property | Mild Steel | PT Steel | Concrete |
|---|---|---|---|
| $E_s$ or $E_c$ (GPa) | 200 | 200 | 27.4 |
| $f_y$ (MPa) | 400 | 1560 | - |
| $f_u$ (MPa) | 500 | 1860 | - |
| $f'_c$ (MPa) | - | - | 30 |
| $\varepsilon_y$ *(mm/mm)* | 0.002 | - | 0.0078 |
| $\varepsilon_{sh}$ *(mm/mm)* | 0.04 | - | - |
| $\varepsilon_u$ *(mm/mm)* | 0.06 | - | - |
| $\varepsilon_{c\text{-}max}$ *(mm/mm)* | - | - | 0.0035 |



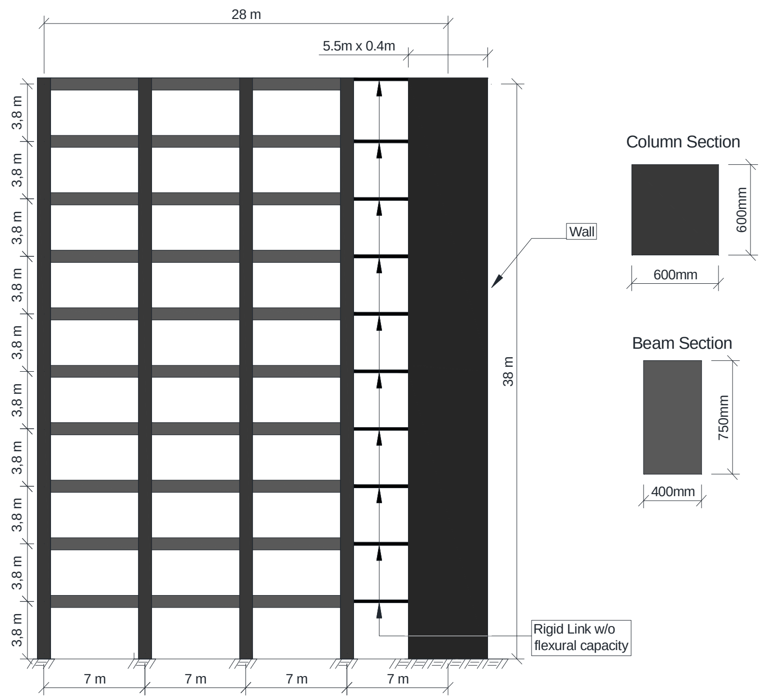




**Figure 3.1.** Layout and Elevation of the frame-wall structure designed following the proposed methodology.

Another important decision that should be addressed at the beginning is the beam strength distribution in the frames. For constructability and since the wall help to control drift, a constant beam strength distribution is recommended and selected for this example. The beam and column dimensions are shown in **Figure 3.1**.

### 3.2. Equivalent SDOF characteristics

One of the key calculations in DDBD is the equivalent SDOF Design Displacement, computed through the design displacement shape from the MDOF. The MDOF design displacement shape is calculated from the design drift, which can be modified in order to take into account higher mode effects following recommendations in Sullivan et al. (2006). Eqn.3.1 is proposed for the calculation of the structure displacement shape. The yield displacement computation has two different equations since it is calculated assuming a linear curvature profile from the base up to the height of contraflexure (the point where the bending of the walls reverses), and curvatures are taken as zero above the point of contraflexure. The height of contraflexure can be approximated using charts from the DBD Model Code (Sullivan et al. 2012).

$$\Delta_{Di} = \Delta_{yi} + \left(\theta_d - \left(\frac{3}{8}\frac{\phi_{yw}}{\gamma}H_n + \theta_{yw}\right)\right)H_i \tag{3.1}$$

$$for\ \ H_i \le H_{CF}:\ \ \Delta_{\mathrm{yi}} = \frac{1}{2}\frac{\phi_{yw}}{\gamma}\left(H_i^2 - \frac{H_i^3}{2H_{CF}} + \frac{H_i^5}{20H_{CF}^3}\right) + \theta_{yw}H_i \tag{3.2a}$$

$$for\ \ H_i > H_{CF}:\ \ \Delta_{\mathrm{yi}} = \Delta_{y-H_{CF}} + \theta_{H_{CF}}(H_i - H_{CF}) \tag{3.2b}$$

where the $\theta_d$ is the design drift (amplified for higher mode effects if needed), $\Phi_{yw}$ the wall yield curvature from Eqn.3.3, γ is the wall curvature factor (set as 1 for this example), $\theta_{yw}$ is the wall yield rotation given by Eqn.3.4, $H_n$ is the total height, $H_i$ the height of the level *i*, $H_{CF}$ is the height of contraflexure and $\Delta_{y\text{-}HCF}$ is the yield displacement at height of contraflexure (using $H_{CF}$ instead of $H_e$ in Eqn.3.2a).

$$\phi_{yw} = 2\frac{\varepsilon_y}{\mathrm{L_W}} = 2\frac{0.0022}{5.5} = 0.0081 \tag{3.3}$$

$$\theta_{yw} = \frac{\varepsilon_y(\ell_{ub}+2\Delta_{sp})}{l_w(1-\nu)-d_{AS}} = \frac{0.0022(0.35+2(0.022\cdot400\cdot.025))}{5.5(1-0.2)-1.32} = 0.000596 \tag{3.4}$$

Table 3.1 presents the results of the displacement calculation for the 10-storey building using Eqns.3.1 to 3.4.

**Table 3.1.** Results for the displacement profiles for a frame-wall structure.

| Level (i) | $m_i$ (T) | $H_i$ (m) | $\Delta_{y\,i}$ (m) | $\Delta_i$ (m) | $m_e\Delta_i$ (T·m) | $m_e\Delta_i^2$ (T·m$^2$) | $m_e\Delta_iH_i$ (T·m$^2$) |
|---|---|---|---|---|---|---|---|
| 0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| 1 | 265 | 3.8 | 0.0076 | 0.048 | 12.7 | 0.6 | 48.3 |
| 2 | 265 | 7.6 | 0.024 | 0.106 | 28.1 | 3.0 | 213.5 |
| 3 | 265 | 11.4 | 0.048 | 0.170 | 45.1 | 7.7 | 513.6 |
| 4 | 265 | 15.2 | 0.076 | 0.239 | 63.3 | 15.1 | 962.7 |
| 5 | 265 | 19.0 | 0.107 | 0.31 | 82.2 | 25.5 | 1560.9 |
| 6 | 265 | 22.8 | 0.139 | 0.383 | 101.5 | 38.9 | 2314.1 |
| 7 | 265 | 26.6 | 0.172 | 0.458 | 121.4 | 55.6 | 3228.4 |
| 8 | 265 | 30.4 | 0.204 | 0.53 | 140.5 | 74.4 | 4269.7 |
| 9 | 265 | 34.2 | 0.234 | 0.601 | 159.3 | 95.7 | 5446.9 |
| 10 | 225 | 38.0 | 0.264 | 0.672 | 151.2 | 101.6 | 5745.6 |
| | | | | $\sum =$ | 905.1 | 418.1 | 24303.6 |

Having established the displacement profile and the summations of the last three columns from Table 3.1, the well known equivalent SDOF procedure from **Figure 2.1,** is applied to calculate the equivalent SDOF system design displacement as 46.2cm.

In addition to the equivalent SDOF design displacement, the energy dissipation of the MDOF system needs to be accounted for. Therefore, the ductility needs to be computed as well as the equivalent viscous damping (EVD) in order to reduce the design spectrum and obtain the effective period, $T_e$, of the equivalent SDOF.

The ductility of frame-wall systems can be computed by combining the frame and wall components of ductility $\mu_F$ and $\mu_W$ respectively, using the proportions of the overturning moment selected at the start of the design process (Sullivan et al, 2006). The system ductility is thus obtained from Eqn.3.5. Since $\beta_F$ was set as 0.50, $\beta_W$ (the OTM taken by the walls) will be 0.50 as well.

$$\mu_{sys} = \mu_F\beta_F + \mu_W\beta_W \tag{3.5}$$

The computation of the frame wall ductility components will depend on the yield displacements of the individual components. The frame yield displacement is computed using Eqn.3.6 following the recommendations from the PRESSS Design Handbook (NZCS 2010).

$$\mu_F = \frac{\Delta_D}{\frac{0.24\varepsilon_y L_{beam}}{d_{beam}}H_e} = \frac{0.462}{\frac{0.24\cdot0.0022\cdot7.0}{0.75}26.9} = 3.385 \tag{3.6}$$

The wall yield displacement is computed using Eqn3.2 at the effective height (of the equivalent SDOF system). After the yield displacement computation, one should divide the equivalent SDOF design displacement by the wall yield displacement to find the ductility as show in Eqn.3.7.

$$\mu_W = \frac{\Delta_D}{\Delta_{y\_He}} = \frac{0.462}{0.173} = 2.67 \tag{3.7}$$

Finally the system ductility, $\mu_{sys}$, is calculated with Eqn.3.5 and for the case study building this gives $\mu_{sys}$ =3.08.

### 3.3. Equivalent Viscous Damping and Spectral Reduction

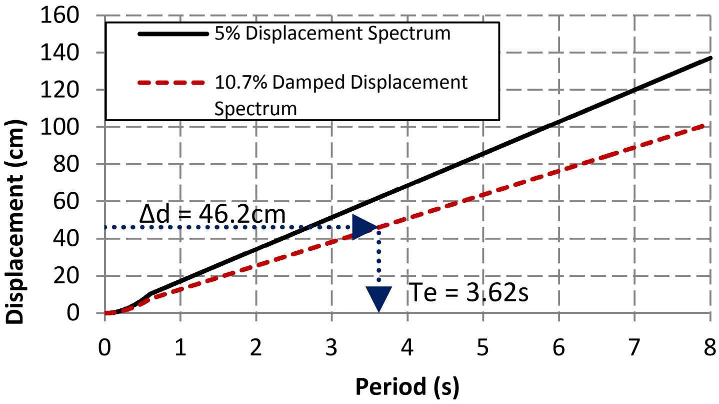
The EQV of systems having flag-shape hysteresis is an on-going research topic. There are some recommendations in the literature (Ceballos et al 2006, Dwairi et al 2007, NZCS 2010, Pennucci et al 2009) that can be followed. If for the spectral reduction, the “displacement reduction factor” (Pennucci et al, 2011) is preferred, then the specific damping computation is not needed anymore and the displacement reduction factor can be computed using only the ductility and the corrected reduction expression depending on energy dissipation characteristics of the building. This Design Example follows Priestley et al. (2007) DDBD approach and therefore the damping is needed and since this design is intended to be code complaint, Eqn.3.8 from the New Zealand Standard (NZS3101.1 2006) is followed.

$$\xi_{sys} = 0.05 + 0.3\frac{\left(1-\frac{1}{\sqrt{\mu_{sys}}}\right)}{\lambda_{sys}+1} = 0.05 + 0.3\frac{\left(1-\frac{1}{\sqrt{3.08}}\right)}{1.25+1} = 0.107 \tag{3.8}$$

Accordingly, the spectral reduction factor is computed with Eqn.3.9.

$$\Delta_{(T,\xi)} = \Delta_{(T,5\%)}\left(\frac{0.07}{0.02+\xi_{sys}}\right)^{1/2} = \Delta_{(T,5\%)}\left(\frac{0.07}{0.02+0.107}\right)^{1/2} = \Delta_{(T,5\%)}\cdot 0.742 \tag{3.9}$$

Then, after reducing the design spectrum, the effective period, $T_e$ is obtained in **Figure 3.2**, where the SDOF results are summarized as well.




| Characteristic | Value |
|---|---|
| $\Delta_d$ (m) | 0.462 |
| $m_e$ (T) | 1959 |
| $H_e$ (m) | 26.9 |
| $\mu_F$ | 3.49 |
| $\mu_W$ | 2.671 |
| $\mu_{Sys}$ | 3.08 |
| $\varepsilon_{Sys}$ | 0.107 |

**Figure 3.2.** Response spectra at 5% and 10.7% damping (left) used for the case study and SDOF results (right).

Once the effective period has been calculated the rest of the SDOF parameters can be computed following the Standard DDBD approach via Eqn.3.10.

$$K_e = \frac{4\pi^2 m_e}{T_e^2} \rightarrow V_B = K_e\Delta_d \rightarrow OTM_{TOTAL} = V_B H_e \tag{3.10}$$

The effective stiffness, $K_e$ results to be 5902kN/m, the design base shear for the whole system is 2727kN and the system OTM is equal to 73349kNm.

### 3.4. Base Shear Distribution and Element Forces

The next important calculation to be addressed is the shear and moment distribution over the height of frames and walls. The OTM of the system is equal to the base shear multiplied by the effective height, and the OTM distribution between components is calculated from the OTM ratio. Remembering that 0.50 was selected for this case study, (i.e. the frame and the walls take 50% of the total OTM each).

In line with the assumption of a constant beam strength distribution, the frame overturning moment profile can be approximated as triangular and thus, the frame shear is computed by dividing its OTM by the total height of the frame. The wall shear is then the difference of the frame shear and the total shear. Furthermore, since there are equal beam strengths up to the roof level, the structural analysis procedure from Priestley et al (2007) shown in **Figure 3.3** can be followed and therefore the beam design moments, $M_{bi}$, for all beams (except the roof beams) can be computed using Eqn.3.11.

$$M_{bi} = \frac{V_F H_i}{n_{pl}} \tag{3.11}$$

where $V_F$ is the frame shear and $n_{pl}$ is the number of expected plastic hinges to be formed in the storey (for this specific case $n_{pl}$=6 for each frame, since there are 3 bays and 2 plastic beams per bay are expected). The roof's beams moment should be half of the moment computed with Eqn 3.11.

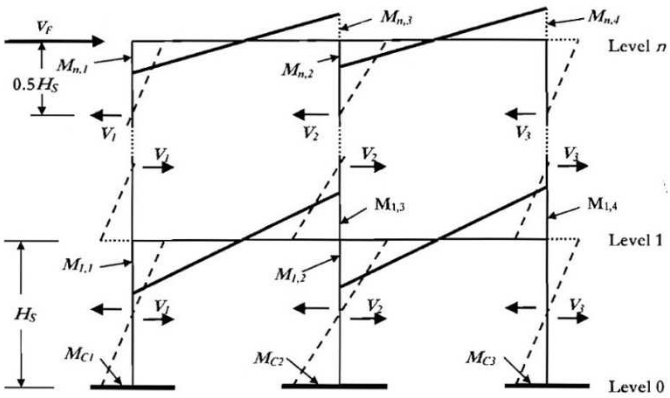



**Figure 3.3.** Shear and Moments in the frames when constant beam strength is assumed (Priestley et al, 2007).

Following the moment distribution structural procedure in **Figure 3.3**, the moment demand in the internal and external columns is computed with Eqn.3.12.

$$M_{ci} = V_{col} H_i - 0.5 \sum M_{bi} \tag{3.12}$$

Finally, the design results from the structural analysis are presented in Table 3.2. Note that in line with the recommendations from Priestley et al. (2007) all values besides $OTM_W$ and $M_b$ should be modified after performing capacity design.

**Table 3.2.** Final Design Results for the 10-storey hybrid frame-wall building.

| **SYSTEM COMPONENTS** | | | **BEAMS** | | **BASE COLUMNS** | |
|---|---|---|---|---|---|---|
| | **Shear (kN)** | **OTM (kNm)** | | **Moment (kNm)** | | **Moment (kNm)** |
| Total | 2728 | 73348 | Intermediate | 611 | External | 306 |
| Frame | 965 | 36674 | Roof | 306 | Internal | 611 |
| Wall | 17616 | 36674 | | | | |

### 3.5. Flexural Design of Precast Hybrid Rocking Connections

The complete design of the hybrid rocking joints is undertaken following the PRESSS Design Handbook (NZCS 2010). Here only the main results are presented and explained. The design of the hybrid rocking wall-foundation interface is presented in some detail and the procedure can be extrapolated for the beam-to-column interface just by setting the gravity load to zero.

Firstly the actual rotation (or gap opening) should be computed, in order to carry out the design procedure with the actual gap opening. The actual wall rotation is made up of a rigid body rotation (gap opening) and an elastic deformation (wall deformation).

The PRESSS Design Handbook (NZCS 2010) goes through the equations and methodology for the complete design. One of the most important concepts presented, is the “monolithic beam analogy“which provides an analytical procedure for the calculation of strains in the concrete section. Using the PRESSS Design Handbook (NZCS 2010) the wall is designed for a rotation of 1.07%, a Moment Demand of 36674 kNm, yield rotation of 0.00062 and using a minimum re-centring ratio of 1.25. **Figure 3.4** shows the final reinforcement details (mild steel and post-tensioned steel) of the hybrid wall section after performing design following the PRESSS Design Handbook recommendations. Details of the beam-to-column reinforcement were also found using the PRESSS Design Handbook but are not shown here due to space limitations. Interested readers should refer to Roldán et al (2012)

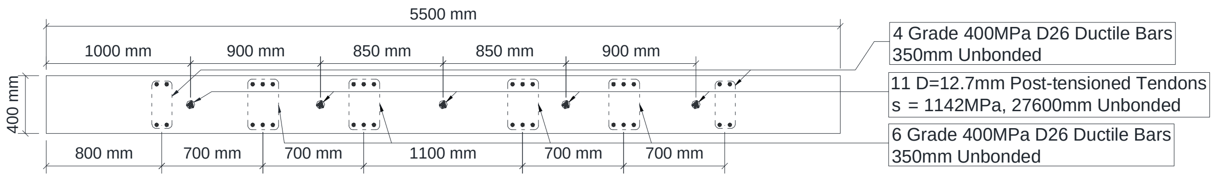



**Figure 3.4.** Wall Section with steel details after design.

## 4. DESIGN ASSESSMENT USING WITH NON-LINEAR TIME HISTORY ANALYSES

The method is analytically tested by comparing the predicted displacement shape and inter-storey drifts design with non-linear time-history analysis results obtained using the software Ruaumoko2D (Carr, 2007). Ten spectrum-compatible acceleration time-histories from soil type C and a Corner Period equal to 8s from the PEER data base were scaled to match the design spectrum; Table 4.1 presents the records and their characteristics. The original selection of accelerograms was made by Maley et al (2012)

**Table 4.1.** Record Set linear displacement spectra Type soil C (adapted from Maley et al. 2012).

| Earthquake Number | PEER Number | Earthquake name | M | Distance | Scaling Factor |
|---|---|---|---|---|---|
| EQ1 | 1233 | Chi-Chi, Taiwan | 7.62 | 36 | 2.1 |
| EQ2 | 1153 | Kocaeli | 7.51 | 127 | 7.9 |
| EQ3 | 851 | Landers | 7.28 | 157 | 4.0 |
| EQ4 | 1810 | Hector | 7.13 | 92 | 2.9 |
| EQ5 | 1629 | St Elias, Alaska | 7.54 | 80 | 1.5 |
| EQ6 | 777 | Loma Prieta | 6.93 | 28 | 1.8 |
| EQ7 | 1043 | Northridge-01 | 6.69 | 52 | 5.8 |
| EQ8 | 728 | Superstition Hills-02 | 6.54 | 13 | 2.3 |
| EQ9 | 172 | Imperial Valley-06 | 6.53 | 22 | 5.1 |
| EQ10 | 2615 | Chi-Chi, Taiwan-03 | 6.2 | 40 | 5.6 |

The program Ruaumoko2D (Carr, 2007) utilises a concentrated plasticity model for the representation of the element zones presenting potential inelastic behaviour. As such, beams, columns and walls (except for ground floor columns and walls) were modelled as line members connected by points and following the capacity design principles, linear elastic stiffness was assigned to these elements, since they are not intended to yield. The beam-to-column joints were modelled following the recommendation from Pampanin et al. (2001) using 2 rotational springs in parallel as shown in **Figure 4.1**. One spring represents the ductile steel with a “fat” Takeda hysteresis rule with the same parameters as for the base column plastic hinges but with a reloading stiffness factor of 0.2 and the computed post-yielding stiffness (represented by the spring number 2 in **Figure 4.1**). The other spring has the characteristics of the post-tensioning steel and is modelled by a bi-linear elastic hysteresis rule with characteristics that will be explained below (represented by spring number 1 from **Figure 4.1**).

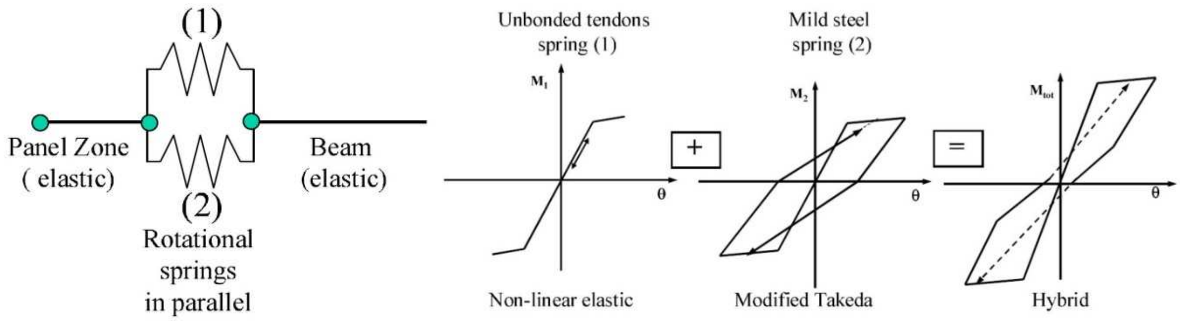



**Figure 4.1.** Analytical model of hybrid connections and hysteresis rules used (NZCS 2010).

A Rayleigh Damping with Tangent damping matrix as Secant damping matrix stiffness is used in Ruaumoko for the analysis as recommended by Priestley and Grant [refer Sullivan et al, 2006]. The elastic damping of the first mode was set as 2.1% following the approach from the recommendation from Grant and Priestley (refer to Sullivan et al, 2006). A time step equal to 0.005s has been adopted for the dynamic equation integration in Ruaumoko and the masses were assigned as lumped masses at the nodes. The analyses were carried out assuming small displacement in RUAUMOKO2D since previous analyses proved that for this case study, the P-Δ effect has no a major influence in the overall response. No strength degradation was modelled. Interested readers should refer to Roldán et al (2012).

The comparisons of the predicted displacements and drift from the proposed method with NLTHA are shown in **Figure 4.2.** It is clear from the Figure that the predicted displacements and drifts response from the proposed method matches the mean of the ten records very well. It can be also noted that mean inter-storey drift didn't overpass the design drift limit and also match with the expected values. These findings indicate that the proposed Direct DBD methodology is very effective for frame-wall structures with hybrid rocking joints.

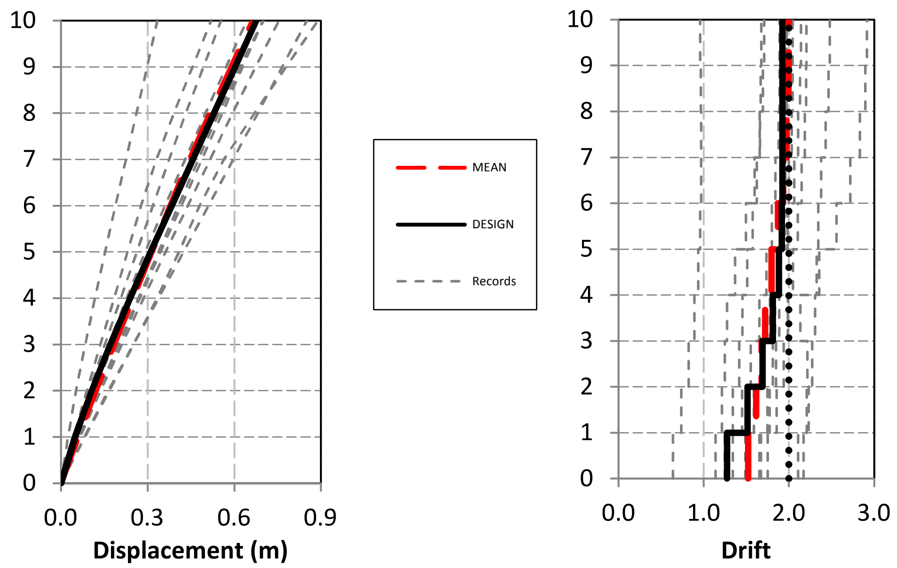



**Figure 4.2.** Maximum recorded displacement (left) and maximum recorded drift (right) from NLTHA compared with computed displacement and drift from proposed methodology for the case study.

## 5. CONCLUSIONS

A displacement-based method for the analysis and design of frame-wall structures with hybrid-rocking connections was introduced through applications to a 10-storey case study building. The method was assessed by subjectivity an accurate model of the design solution to NLTH analyses. The results demonstrate that the proposed methodology for frame-wall structures with hybrid-rocking connections can control the maximum displacements and storey drifts accurately.

The finding displacement profile proposed by Sullivan et al. (2006) for frame-wall structures is valid also when a structure has hybrid-rocking connections. Important variations were introduced in order to take into account the rigid-body rotation of rocking systems. The assumption of having a linear curvature profile from the base up to the point of contraflexure and then zero curvature appears to be valid for frame-walls with hybrid-rocking joints but for structures with small frame OTM ratio this approach may be conservative and future research could investigate the use of a non-linear curvature profile for a more efficient procedure.

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