Elastic Design (μ=1.0) of Damage Avoidance Structures

Farhad Mohammadi Darani[1]

1. WSP in New Zealand

Pouyan Zarnani[2]

2. Auckland University of Technology (AUT)

Pierre Quenneville[3]

3. University of Auckland

ABSTRACT

Damage Avoidance Design (DAD) has been considered an efficient way for seismic design. Different solutions to achieve a robust DAD structure has been proposed and tested by researchers in the past. However, there is a lack of simplified and office-engineer friendly design procedure to design DAD structures. Despite the benefits of the DAD structures, design difficulty may result in a tendency amongst the engineers to avoid damage avoidance in their designs. One of the challenges is the adoption of a suitable ductility factor. Engineers may wonder how to adopt a ductility factor for their DAD designs. There has been ongoing research to develop damping-ductility relationships for DAD systems. However, a unique yet simple procedure has not been introduced. This paper aims to change the current mindset of ductile design of DAD structures. DAD structures can be easily looked at as elastic systems with ductility of one with a suitable damping ratio. This simple procedure has been introduced, studied and verified in this paper. The results have been compared to Nonlinear Time History Analysis (NLTHA) confirming the reliability of the proposed μ=1.0 procedure for the seismic design of DAD systems.

1 INTRODUCTION

Damage Avoidance Design (DAD) philosophy is the basis of the seismic design of the structures aimed not to suffer from damage due to Design Base Earthquake (DBE) [1]. The desired characteristics of damage avoidance systems are repeatable and predictable hysteresis damping and reliable self-centring. Different damage-avoidance systems have been introduced and tested successfully [2-7].

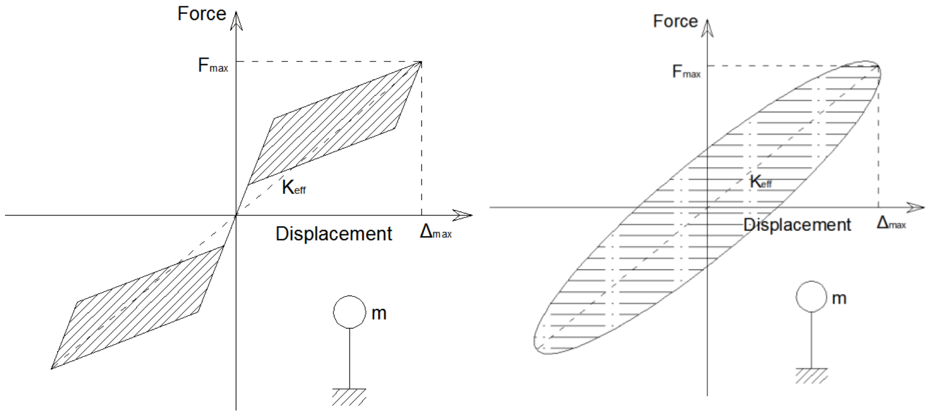
In order to achieve a structural system with advantages similar to RC walls, rocking concrete shear walls have been developed, tested and even used in real constructions [3, 8-11]. The damage is controlled in rocking concrete shear walls as the yielding of reinforcement and concrete crushing is not expected in these systems [3, 12-14]. Also, controlled self-centring could be provided by either unbonded post-tensioning [7, 11, 15-17] or self-centring connections [12]. Therefore, problems related to damage and residual deformation are almost overcome in rocking walls designed based on DAD philosophy.

Unbonded post-tensioning was used in pre-cast concrete walls in PREcast Seismic Structural Systems (PRESSS) in early 1990s [18]. Detailed analytical studies on the behaviour of Single Rocking Wall (SRW) systems was done by Kurama et al. [19]. They suggested to use additional Spiral reinforcement at the wall toes to mitigate the damage of concrete due to crushing. Kurama and colleagues [20] also performed a series of studies on the unbonded PT walls with additional supplemental damping such as yielding, friction or viscous damping. The results showed that the additional dampers effectively reduce the seismic demand if they are properly designed. Priestley [17] used U-shaped Flexural Plates (UFPs) in coupled rocking wall systems. Other efforts were made for using internal grouted mild steel reinforcement for additional hysteresis damping [9]. Externally mounted mild steel, Tension and Compression Yielding (TCY) dampers as well as viscous dampers were also designed and tested by Marriot et al. [9]. Sritharan and colleagues [8] suggested a PT rocking wall system as Pre-cast Wall with End Columns (PreWEC). In this paper, a rocking wall concept has been introduced in which post-tensioning tendons have been replaced with self-centring friction dampers as hold-down connections

For DAD systems, the damage control is the main design objective. Therefore, Performance Based Earthquake Engineering (PBEE) methods are the preferred design approaches when it comes to low-damage structures [21]. Usually, maximum drift ratio is selected as the design objective. The Direct Displacement Based Design (DDBD) has been used for seismic design of rocking wall structures and the results were evaluated using experimental investigations [22]. In this paper, a DDBD approach will be outlined and used for seismic design of DAD structures and will be verified by designing a prototype structure. In this study, the seismic performance and design of self-centring systems is discussed. DDBD is implemented while the concept of damping ratio and effective stiffness are used rather than the ductility factor. The performance of each structural configuration is discussed and verified as well. The prototype structure is designed based on the proposed design method. Finally, the performance of the structure is verified using the Nonlinear Time History (NLTH) analysis through the comparison of performance measures with the primary design objectives.

2 CONCEPTUAL FRAMEWORK

Generally, self-centring structures can be considered as elastic structures having a bilinear performance. Accordingly, the seismic design of such structures is less complex than in-elastic or ductile structures. Such conclusion could be made based on the following two points. First, in self-centring structures, the residual displacement is not expected, and the structure will be at its initial position at the end of loading. The second point refers to the fact that self-centring systems show a repetitive cyclic behaviour without strength or stiffness degradation. As can be seen from Fig. 1, a Single Degree of Freedom (SDOF) self-centring system which is a system with bi-linear elastic behaviour and non-viscous damping which is usually produced by yielding or friction (Fig. 1a), can be considered equal to an elastic system (μ=1.0) with viscous damping.



(a) **(b)**

Figure 1. Load-deformation response of two different SDOF systems: a) Bi-linear elastic system with additional non-viscous damping (self-centring system) b) Elastic system with viscous damping.

Based on the graphs shown in Fig. 1, a self-centring structure is an equivalent of an elastic structure with similar effective stiffness when the damped energy in both systems are equal at the desired displacement demand of Δ_{max}. When dealing with an elastic system, it is not required to consider any ductility for calculating the seismic demand. The code specified design spectra which is usually developed for 5% viscous damping is the basis of the elastic design. Therefore, self-centring systems are not inherently ductile when ductility is defined as irrecoverable plastic deformation. Having said that, when using code design spectra, elastic (non-ductile, μ=1) behaviour is considered with a suitable damping. Therefore, the only requirement for the design of the system mentioned in Fig. 1a is to adopt a suitable viscous damping to be used for adjusting the code 5% damped design spectra.

Equivalent viscous damping ratio (ξ_{eq}) is the combination of hysteretic damping ratio (ξ_{hys}) and the inherent viscous damping of the system at the period of vibration (ξ_v). The concept of hysteretic damping ratio (ξ_{hys}) is illustrated in Fig. 2. It can be expressed by Eq. 1 which is a function of the hysteretic damped energy to elastic restored energy.

$$\boldsymbol{\xi_{hys} = \frac{1}{4\pi}\frac{E_D}{E_S}} \qquad (1)$$

$$\xi_{eq} = \xi_{hys} + \xi_v \qquad (2)$$

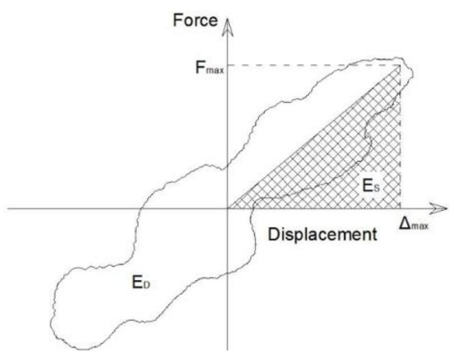


Figure 2. Hysteresis dissipated energy and equivalent viscous damping concept.

From Fig. 1, the maximum expected displacement, Δ_{max}, should be defined by the designer as a design objective. Δ_{max} or the displacement demand can be linked to the spectral acceleration based on Eq. 3.

$$\Delta_{max} = \boldsymbol{S_d} = \frac{\boldsymbol{T_n^2}}{\boldsymbol{4\pi^2}} \boldsymbol{S_a R_\xi} \tag{3}$$

$$R_\xi = \left(\frac{0.07}{0.02 + \xi_{eq}}\right)^{0.5} \tag{4}$$

where S_d is spectral displacement, S_a is spectral acceleration and R_ξ is the spectrum modification factor for the damping ratio being considered.

the ultimate force at the considered seismic hazard level can be obtained using Eq. 5 and Eq. 6.

$$\boldsymbol{k_{eff}} = \frac{\boldsymbol{4\pi^2}}{\boldsymbol{T^2}} \boldsymbol{m} \tag{5}$$

$$F_{max} = k_{eff} \Delta_{max} \tag{6}$$

where, k_{eff} is the effective stiffness of the system at the ultimate displacement as mentioned in Fig. 4.1, m is the seismic mass of the system and F_{max} is the design seismic force. When F_{max} and Δ_{max} are known, the self-centring system can be designed based on the flag-shaped response parameters mentioned in Fig. 4.3. The details of this step are discussed as follows.

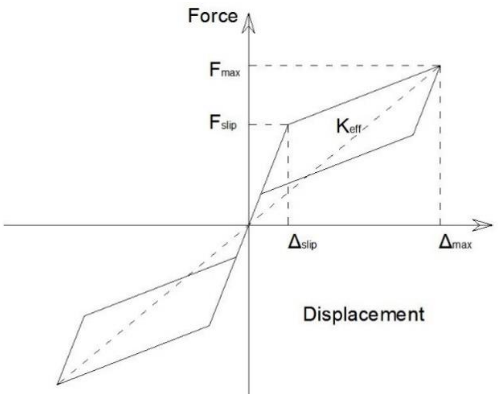


Figure 3. Design parameters of a SDOF self-centring system.

When the required parameters are assumed based on Fig. 3, the system ultimate and slip forces and displacements should be redistributed in the structure in order to quantify the force and displacement demands of the self-centring connection components. Then, each component will be designed based on the criteria required. At this stage, the preliminary design of the system is completed, and the design outcome should be verified.

In order to verify the primary design, a cyclic pushover analysis should be carried out. The structure should be laterally loaded up to the design displacement and then unloaded to its initial position under a reversed cycling loading for at least one full cycle. ξ_{hys} is calculated using Eq. 1 based on the obtained force-displacement response. When the obtained ξ_{hys} from the graph is equal to the adopted ξ_{hys}, the design and analysis process has been converged and the design is completed.

3 SELF-CENTRING STRUCTURAL CONNECTOR (SSC)

There are several ways to provide the self-centring ability in a structure. It is important to note that a suitable self-centring system should provide enough damping in addition to its restoring capacity. Friction dampers have been widely studied and applied in earthquake resisting structures [23-25]. When the energy dissipation through friction is combined with self-centring capacity, a robust device could be achieved in terms of both re-centring and repetitive cyclic energy dissipation. There have been several devices developed by incorporating these characteristics [26-28]. the Self-Centring Structural Connector (SSC) is used to represent a tension-only hold-down connection [29, 30]. The performance of this damper is introduced as follows.

As shown in Fig. 4, one of the parts of the SSC is the friction discs. These discs along with the tube take the major role of energy dissipation in the damper. The normal force, which is perpendicular to the surface of the tube, is applied to these discs. The friction will be developed as the result of relative movement between the tube and the discs. The earthquake force is applied to the friction discs through rods.

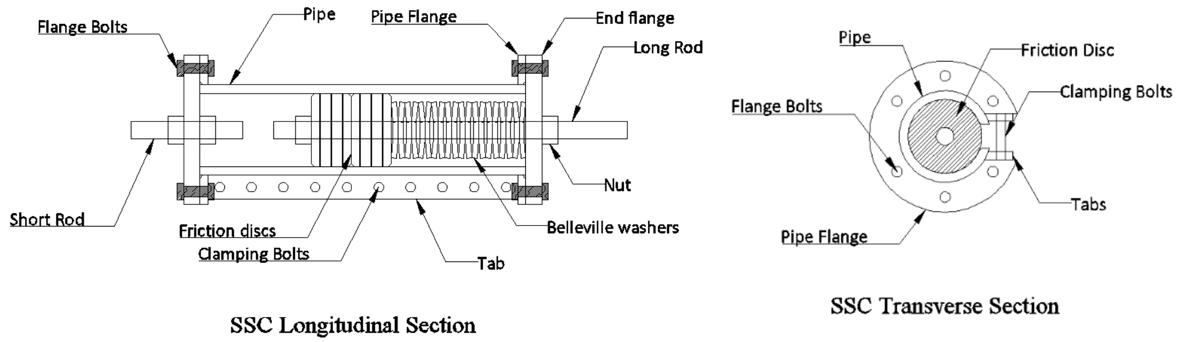


Figure 4. Self-centering structural connector.

Springs are responsible for the self-centring capability. In the SSC, Belleville washers are used for the spring members. In the design of SSC, the springs have been placed in the series layout. The number of these springs are selected based on the required displacement capacity of the SSC.

As can be seen from Fig. 5., the developed damper in this study has a flag-shaped force-displacement response. The reader is referred to [31] for more details.

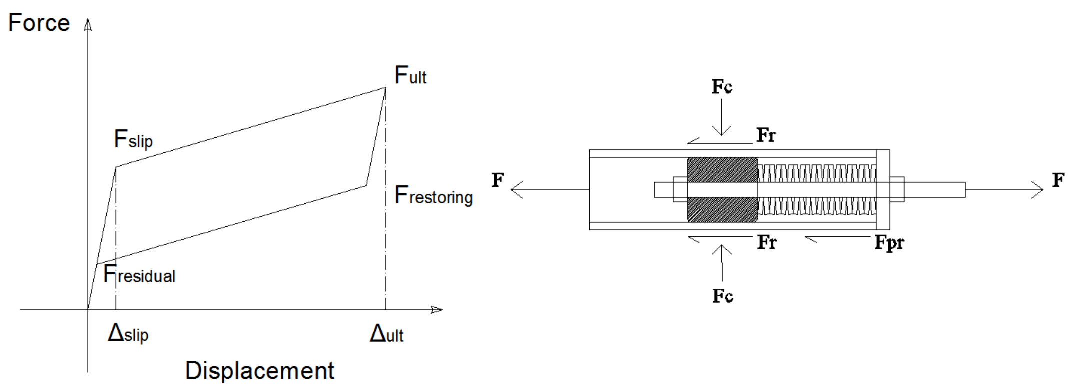


Figure 5. Free body diagram and force-displacement response.

4 PROTOTYPE STRUCTURE

In this section, a five-story structure is analysed and designed based on the procedure described in the previous section. The plan and elevation dimensions of this structure is shown in Fig. 6.

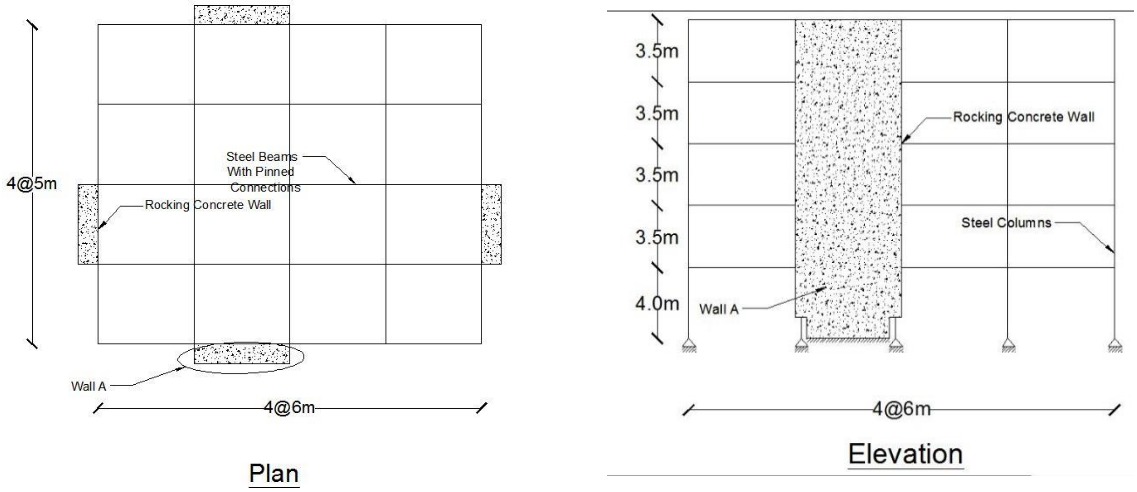
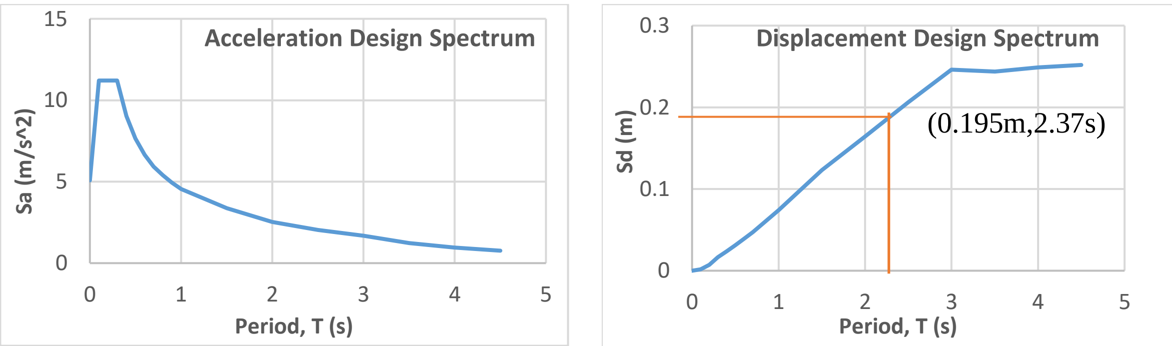


Figure 6. Structural plan and elevation of the modelled building.

The structure is located in Christchurch, New Zealand on soil type class C or shallow soil site as per NZS 1170.5 [32]. The hazard factor, Z, for the Christchurch region is 0.3. A design working life of 50 years is adopted for this structure and the importance level of this building is considered to be 3. 1.5% maximum inter-story drift for the ULS earthquake is selected for the building modelled in this paper. The S_a spectrum is mentioned in Fig. 7. The displacement spectrum, S_d, is derived using Eq. 3 and Eq. 4 and is defined in Fig. 7b.



(a) **(b)**

Figure 7. Acceleration design spectrum (Left) and Displacement design spectrum (Right).

Based on the calculated Δ_{max}, the natural period $T_n = 2.37$s is obtained from the design displacement spectrum as mentioned in Fig. 7b. The effective stiffness and ultimate base shear can be calculated using Eq. 5 and Eq.6.

$k_{eff} = 5685\ kN/m$

$F_{max} = 1107\ kN$

The base shear is distributed along the height of the structure and the structure can be analysed accordingly. When the analysis is finished, the forces and displacements in the self-centring connections are determined and the connections can be designed for the demand actions.

5 ROCKING WALLS WITH TENSION-ONLY HOLD-DOWNS

The free body diagram of rocking walls with tension-only hold-downs is mentioned in Fig. 4.12. Self-centring Structural Connector (SSC) is used for representing the tension-only hold-down. There is a main difference between tension-only and double-acting system. In tension-only systems, the governing equations are the same for before and after the onset of rocking motion (Eq. 7 and Eq.8).

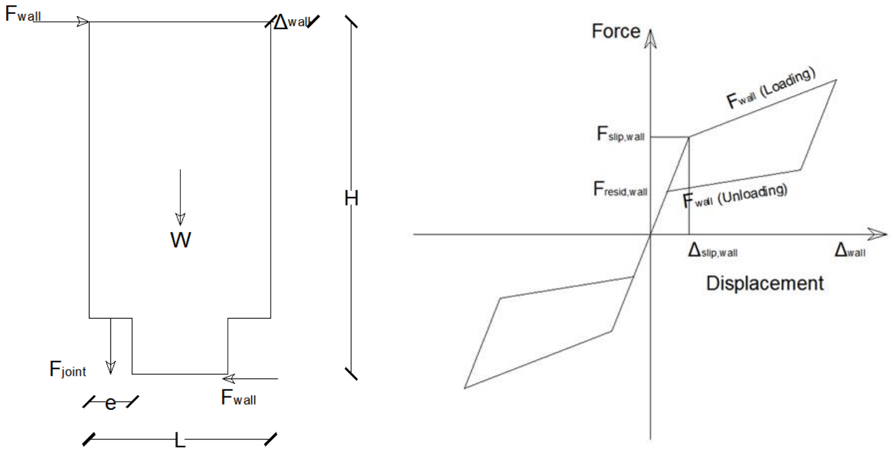


Figure 8. Free-body diagram of rocking walls equipped with tension-only self-centring connections.

$$\boldsymbol{F_{wall}} = \frac{\mathbf{1}}{\boldsymbol{H}}\left[\boldsymbol{F_{joint}}(\boldsymbol{L} - \frac{\mathbf{3}\boldsymbol{e}}{\mathbf{2}}) + \boldsymbol{w}\frac{\boldsymbol{L} - \mathbf{2}\boldsymbol{e}}{\mathbf{2}}\right] \tag{7}$$

$$\frac{\Delta_{rot}}{H} = \frac{\Delta_{joint,t}}{L - \frac{3e}{2}} \tag{8}$$

The developed equations are verified through experimental investigation. The test demonstrations are illustrated in Fig. 9. the reader is referred to [33] for more details about the experimental testing and verification of the models developed. As can be seen from the graph, the load displacement performance of the walls equipped with SSCs are accurately predicted by the developed equations.

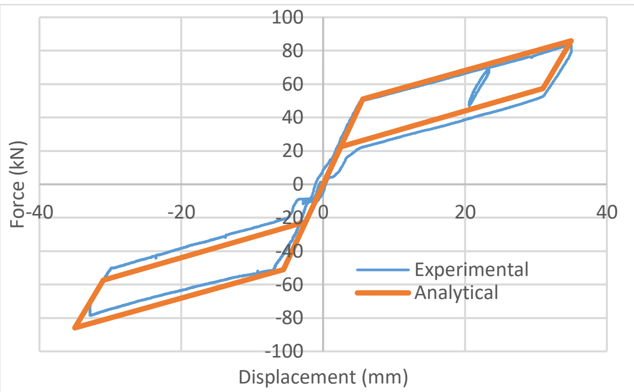


Figure 9. Experimental Verification, SSC shear wall.

6 DESIGN OF PROTOTYPE BUILDING

For the prototype structure illustrated in Fig. 6, the force and displacement demands on the connections of Wall A can be calculated based on the developed equations. The required parameters for wall performance calculations are summarised in Table 1. The design and analysis parameters for tension-only configuration is summarised in Table 2. Accordingly, a Self-centring Structural Connector (SSC) [29] is designed to represent the behaviour of the tension-only hold-down for Wall A.

Table 1. Geometrical and mechanical properties of Wall A.

H (mm)	18000
h_e (mm)	13000
L (mm)	6000
e (mm)	300
W (N)	810000
E (Mpa)	27500
I (mm^4)	5.40E+12

Table 2. Force and displacement parameters calculated for Wall A with tension-only joints.

F_{wall}	560
$\Delta_{elastic}$	2.76
Δ_{wall}	195
Δ_{rot}	192
$\Delta_{joint,t}$	82
$F_{slip,wall}$	373
$F_{slip,joint}$	480
$F_{joint,t}$	918

Force: kN
Displacement: mm

At this stage, a numerical model of the structure is made and analysed under cyclic pushover analysis. The modelled structure was loaded under gravity loads and then analysed under cyclic pushover analysis for seismic actions. The pushover hysteresis curve is shown in Fig.10. It should be noted that the vertical distribution of the base shear obtained in previous sections was used to perform the pushover analysis.

The load-displacement graph is shown in Fig. 10. The ξ_{eq} of 12% is calculated for this structure which is less than the primary assumption of 15%. There are two approaches for the next design attempt. First, the previous steps can be repeated with new ξ_{eq} of 12% and continue until reaching a convergence. Second, the design of the dampers can be reconsidered to achieve a higher damping ratio. Then, the cyclic pushover analysis and the measurement of ξ_{eq} should be repeated until convergence. In this paper, the second approach has been selected.

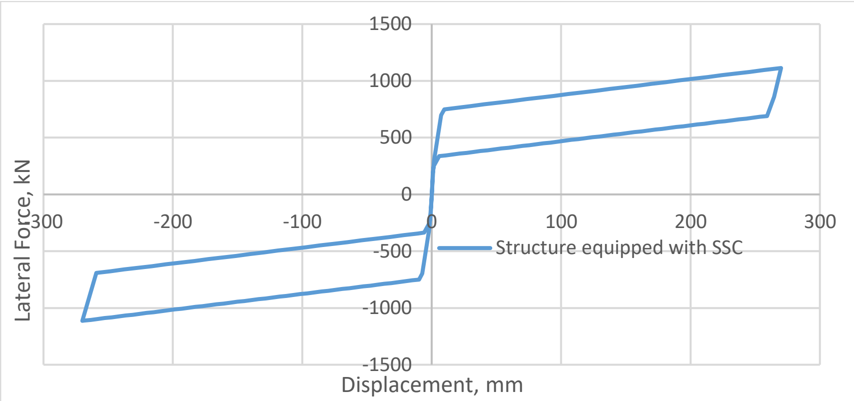


Figure 10. Overall load-displacement response of the structure.

7 NONLINEAR TIME-HISTORY ANALYSIS

In this section, the multi-story building designed based on the proposed design procedure in the previous section, is analysed under nonlinear time-history analysis. The selected ground motions and the scale factor of each ground-motion as per the requirements of NZS 1170.5[32] is summarised in Table 3. The scale factors are driven for each ground motion spectra and scaled to the code designed spectra within a specific period range. The period range and spectral acceleration magnitude were selected as per the code recommendations.

Table 3. Selected Ground-Motions and corresponding scale factors.

Year	Ground Motion	Station	Scale Factor
1979	Imperial Valley	El Centro	0.89
1978	Tabas	Tabas	0.47
2010	Darfield	Hororata School	0.48
1994	Northridge	Saticoy St.	0.58
1989	Lomaprieta	Saratoga	0.85
1995	Kobe	Amagasaki	0.69
1999	Duzce	Duzce	0.55
2011	Christchurch	Botanical Gardens	0.57

In the graph shown in Fig. 11, the maximum observed demand for the structure equipped with double-acting mechanism is 225mm and for the structure equipped with tension-only mechanism is 210mm both observed for the case of Tabas earthquake. The average of eight ground motions are also illustrated in the graph below which is 181mm and 139mm for the double-acting and tension only mechanisms, respectively.

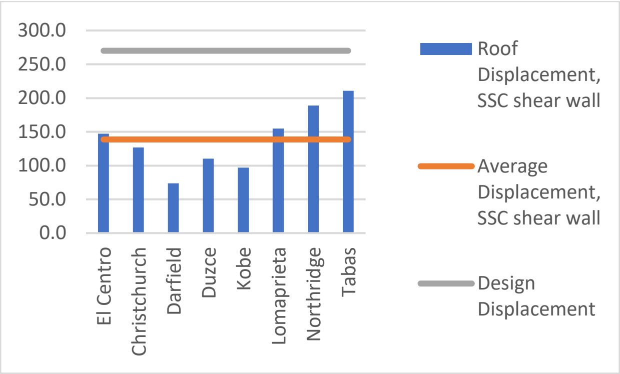


Figure 11. Comparison of maximum roof displacements obtained from NLTH analysis.

As obvious, the drift demand for all the selected ground motions is less than the 1.5% demand targeted as the primary design objective. Therefore, the time-history analysis verified that the design is capable to achieve the desired performance. However, the designer might decide to repeat the design process by selecting a smaller design drift such as 1.25% or even 1.0%.

8 CONCLUSIONS

In this paper, a design procedure was proposed based on Direct Displacement Based Design (DDBD) method. It was emphasised that self-centring structures can be compared to elastic structures with non-ductile (μ=1) given the residual displacement and ductility demand is not expected in such structures. Accordingly, the proposed design procedure was developed based on the equivalent damping ratio and effective stiffness instead of ductility and yielding concepts. A five-story building equipped with concrete rocking shear walls and new Self-centring Structural Connector was designed based on the proposed design procedure. The nonlinear Time-History analysis was carried out in order to verify the seismic performance of the prototype buildings. The structures performed as expected and the response parameters were in compliance with the maximum design limits. Therefore, the proposed design procedure can be used for seismic design of rocking wall structures equipped with self-centring dampers.

9 REFERENCES

1. Mander, J.B., *Beyond ductility.* Bulletin of the New Zealand Society for Earthquake Engineering, 2004. **37**(1): p. 35-44.
2. Holden, T., J. Restrepo, and J.B. Mander, *Seismic performance of precast reinforced and prestressed concrete walls.* Journal of Structural Engineering, 2003. **129**(3): p. 286-296.
3. Ajrab, J.J., G. Pekcan, and J.B. Mander, *Rocking wall–frame structures with supplemental tendon systems.* Journal of Structural Engineering, 2004. **130**(6): p. 895-903.
4. Mulligan, K.J., et al., *Semi-active rocking wall systems for enhanced seismic energy dissipation.* 2006.
5. Rodgers, G.W., et al., *Performance of a damage-protected beam–column subassembly utilizing external HF2V energy dissipation devices.* Earthquake Engineering & Structural Dynamics, 2008. **37**(13): p. 1549-1564.
6. Solberg, K., et al., *Performance of a damage-protected highway bridge pier subjected to bidirectional earthquake attack.* Journal of structural engineering, 2009. **135**(5): p. 469-478.
7. Hamid, N. and J.B. Mander, *Damage avoidance design for buildings.* KSCE Journal of Civil Engineering, 2014. **18**(2): p. 541-548.
8. Sritharan, S., et al., *Precast concrete wall with end columns (PreWEC) for earthquake resistant design.* Earthquake Engineering & Structural Dynamics, 2015. **44**(12): p. 2075-2092.

9. Marriott, D., et al., *Dynamic testing of precast, post-tensioned rocking wall systems with alternative dissipating solutions.* 2008.
10. Marriott, D.J., *The development of high-performance post-tensioned rocking systems for the seismic design of structures.* 2009.
11. Nazari, M., S. Sritharan, and S. Aaleti, *Single precast concrete rocking walls as earthquake force-resisting elements.* Earthquake Engineering & Structural Dynamics, 2017. **46**(5): p. 753-769.
12. Darani, F.M., et al., *Rocking concrete shear walls with self-centring friction dampers for seismic protection of building structuress.* 2018.
13. Toranzo, L., et al., *Shake-table tests of confined-masonry rocking walls with supplementary hysteretic damping.* Journal of Earthquake Engineering, 2009. **13**(6): p. 882-898.
14. Rodgers, G.W., et al., *Beyond ductility: parametric testing of a jointed rocking beam-column connection designed for damage avoidance.* Journal of Structural Engineering, 2016. **142**(8): p. C4015006.
15. Twigden, K.M., *Dynamic response of unbonded post-tensioned concrete walls for seismic resilient structures*. 2016, University of Auckland.
16. Mander, J.B. and C.-T. Cheng, *Seismic resistance of bridge piers based on damage avoidance design*, in *Seismic resistance of bridge piers based on damage avoidance design*. 1997. p. 109-109.
17. Priestley, M.N., et al., *Preliminary results and conclusions from the PRESSS five-story precast concrete test building.* PCI journal, 1999. **44**(6): p. 42-67.
18. Priestley, M.N. and J.R. Tao, *Seismic response of precast prestressed concrete frames with partially debonded tendons.* PCI journal, 1993. **38**(1): p. 58-69.
19. Kurama, Y.C., et al., *Seismic-resistant precast concrete structures: state of the art*. 2018, American Society of Civil Engineers.
20. Kurama, Y.C., *Simplified seismic design approach for friction-damped unbonded post-tensioned precast concrete walls.* Structural Journal, 2001. **98**(5): p. 705-716.
21. Aragaw, L.F. and P.M. Calvi, *Comparing the performance of traditional shear-wall and rocking shear-wall structures designed using the direct-displacement based design approach.* Bulletin of Earthquake Engineering, 2020. **18**(4): p. 1345-1369.
22. Nazari, M., *Seismic performance of unbonded post-tensioned precast wall systems subjected to shake table testing.* 2016.
23. Mualla, I.H. and B. Belev, *Performance of steel frames with a new friction damper device under earthquake excitation.* Engineering Structures, 2002. **24**(3): p. 365-371.
24. Grigorian, C.E., T.-S. Yang, and E.P. Popov, *Slotted bolted connection energy dissipators.* Earthquake Spectra, 1993. **9**(3): p. 491-504.
25. Pall, A. *Performance-based design using pall friction dampers-an economical design solution*. in *13th World Conference on Earthquake Engineering, Vancouver, BC, Canada*. 2004.
26. Christopoulos, C., et al., *Self-centering energy dissipative bracing system for the seismic resistance of structures: development and validation.* Journal of structural engineering, 2008. **134**(1): p. 96-107.
27. Gledhill, S., G. Sidwell, and D. Bell. *The damage avoidance design of tall steel frame buildings-fairlie terrace student accommodation project, victoria university of wellington*. in *New Zealand Society of Earthquake Engineering Annual Conference*. 2008.
28. Zhu, S. and Y. Zhang, *Seismic analysis of concentrically braced frame systems with self-centering friction damping braces.* Journal of Structural Engineering, 2008. **134**(1): p. 121-131.
29. Mohammadi Darani, F., P. Zarnani, and P. Quenneville, *A Structural Connector.* Patent No. WO2020008422A1, NZ IP Office, 2018.
30. Darani, F.M., P. Zarnani, and P. Quenneville, *Development of a New Self-Centring Structural Connector for Seismic Protection of Structures.* Journal of Constructional Steel Research, 2022. **189**: p. 107064.
31. Mohammadi Darani, F., P. Zarnani, and P. Quennville, *Development of a New Self-Centring Structural Connector for Seismic Protection of Structures*. Paper Under review, 2020.
32. Standards New Zealand, *Structural design actions Part 5: Earthquake actions.* NZS 1170.5, 2004.
33. Mohammadi Darani, F., P. Zarnani, and P. Quenneville, *Development of a New Rocking Concrete Shear Wall with Self-Centring Friction Connections, Experimantal Verification.* Paper Under review, 2020.