Effect of Serviceability and Over-strength Mechanism on Seismic Response of Low Damage Structures

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ABSTRACT

Low-damage structural systems are designed in a way that the damage because of the earthquakes is controlled and localised in designated components. These components are normally replaced, repaired or designed to remain functional after earthquakes. There are cases that the seismic actions are larger than design-level actions (e.g. earthquake intensities are higher than design level events). For such cases, low-damage components are designed in a way that can resist such actions. That is the reason why designers normally consider an upper bound for the design of such components. This upper bound is defined as the over-strength mechanism. However, to what extent these upper bounds should be considered in the design is still unknown for many of the low-damage systems. This study investigates the effect of the over-strength margin and serviceability limit state on the overall seismic performance of low-damage systems. Two types of generic structural responses are considered for the study, self-centring braced frames and self-centring rocking wall systems. Numerical models for both concepts are developed and subjected to nonlinear dynamic time-history analysis with different intensities up to large, rare events (MCE). The results show that braced frames' appropriate over-strength margin is relatively smaller than rocking walls. Also, raising the serviceability limit state level has a positive effect on response drifts of self-centring systems. The findings of this study help designers and engineers to perform more efficient and optimised designs for low-damage systems.

1 INTRODUCTION

The global interest in the low-damage design of structures in seismically active areas such as New Zealand has significantly increased in the last decade. A primary driver is the findings from the Canterbury earthquake sequence, where despite the acceptable seismic performance of many of the steel buildings, they had to be demolished because of the residual damage (Bruneau and MacRae 2019). These findings also highlighted the importance of low-damage design in a way that the integrity of the structures is maintained when exposed to design level earthquakes, and the building can be reoccupied after an initial inspection.

Accordingly, many new technologies have been introduced and implemented for reconstructing the city (Bruneau and MacRae 2017). Furthermore, findings from these events demonstrated the importance of self-centring behaviour, where the structure returns to its pre-event position at the end of the earthquake. Conventionally, a self-centring system combines an energy dissipation mechanism (friction connections, viscous dampers or yieldable steel components) with an elastic mechanism (e.g. post-tensioned elements). The result is a flag-shaped load-deformation relationship that the force is almost zero (or insignificant) at zero displacement.

Many different forms of self-centring systems have been introduced in the literature, and a limited number made their way into practice. For steel structures, many new concepts were introduced after the Northridge earthquake to improve the self-centring behaviour of steel structures and reduce the on-site work (Roeder and Venture 2000). Ricles et al. (2001) introduced the combination of high-strength steel bars and steel moment frames to self-centre the structure and reduce the residual damage. Kim et al. further improved the system and combined the PT frames with friction dampers installed on the top and bottom flanges of the beams (Kim and Christopoulos 2008).

For concrete structures, the combination of post-tensioned cables (up the height of the concrete walls or through the concrete beams) and the yielding of the steel components have been tested by many researchers.(Nakaki et al. 1999; Priestley et al. 1999; Bora et al. 2007). It was found that this type of wall assembly, known as the PRESSS (Precast Seismic Structural Systems) concept, has many advantages over conventional concrete walls and can outperform them in terms of lateral performance, while the extent of damage under large earthquakes is significantly less. For timber structures, a concept similar to the PRESSS system has been adopted. Sarti et al. (2014) provided comprehensive research on testing and design of such system with timber elements and highlighted the significant advantages over conventional timber systems, including a fully self-centring behaviour. Hashemi et al. (2017) introduced a novel friction connection (the Resilient Slip Friction Joint (RSFJ)) that can provide damping and self-centring in one assembly. The concept was later tested for rocking Cross Laminated Timber (CLT) (2017) walls and LVL walls (Hashemi et al. 2020) to demonstrate that by implementing this connection system, a reliable self-centring performance can be achieved without the need for post-tensioned elements.

2 THE SERVICEABILITY LIMIT STATE (SLS) CRITERIA AND OVER-STRENGTH MECHANISM

The New Zealand standard for earthquake actions (NZS 1170.5 (New Zealand Standards 2004)) generally defines two limit states for earthquake analysis and design. The first is the Serviceability Limit State (SLS) that; the structure is aimed to remain functional when subjected to small and frequent seismic events. The second is the design level earthquake known as Ultimate Limit State (ULS) in which the structure is aimed to remain stable without collapse and life safety is maintained when subjected to major but rare seismic events (known as design level earthquakes). The standard defines these two limit states via the Return Period Factor (R_u or R_S)). For normal residential and commercial structures (e.g. with Importance Level 2 (IL2)), R_s=0.25 and R_u=1.0, which means for a non-ductile structure, the seismic actions for ULS are expected to be four times larger than those for the SLS. The standard also defines a second type of SLS for high importance (IL3) and post-disaster buildings (IL4) that is named SLS2.

Although the standard does not explicitly define a limit state for Maximum Considered Event (MCE), it implies the response drifts for MCE should be limited to %3.75 while for ULS, this value is limited to %2.5 (similar to other international design standards such as ASCE 7 (American Society of Civil Engineers 2016)).

Two different criteria are defined for the SLS1 and SLS2 limit states. For SLS1, the standard implies that the response drifts should not exceed %0.33. This is defined to restrict the damage in the non- and secondary-structural elements so the building can remain serviceable. For SLS2, insignificant and minimal damage may be accepted providing it does not affect the performance of the building.

The low damage performance criteria for designing structures demand the design to consider an appropriate over-strength mechanism for the system to avoid collapse and maintain its performance in case the earthquake-induced actions are larger than the actions considered for the design. This requirement originates from adopting the capacity design principle that is well-defined within the New Zealand standards and guidelines. This principle requires the structural system to develop a hierarchy of strengths so that the 'weak links' are the first elements to fail before other structural members undergo failure (Priestley and Calvi 1991). The so-called 'weak links' are normally the lateral load resisting members or a specific component within them (e.g. braces in braced frames, beam-column joints in moment-resisting frames or hold-downs for shear walls). To properly adopt this approach, one needs to have a good understanding of the failure mechanism of the lateral load resisting systems so that an appropriate over-strength factor can be chosen for the non-yielding members.

There is no specific and reliable data available for self-centring low-damage structural systems when it comes to the over-strength mechanism. For these systems, not only the designer needs to consider an appropriate over-strength factor for the design but, more importantly, needs to specify a safe 'design displacement' for the system to not only control the response drift but also to control the performance of the structure beyond its design limit. The over-strength factor can be determined by assessing the forces developed in the lateral load resisting members when subjected to higher than design loads but the required extra 'displacement capacity' is harder to assess or predict given that for many systems, it is unknown how the performance of the lateral system affects the displacement demands. This paper seeks to address this issue, investigate and prescribe the appropriate design displacements (also known as 'over-strength margin') for self-centring structural systems.

3 THE STRUCTURAL SYSTEMS CONSIDERED FOR THE ANALYSIS

For this study, the two types of lateral load resisting systems are considered: braced frames and rocking shear walls. The reason for this selection is that most of the implemented self-centring systems belong to these two main categories. Therefore, they were deemed to be the most appropriate to conduct the investigation.

For the case study, a hybrid prototype building is considered. It was assumed that the structure is a four-story hybrid steel-timber building located in New Zealand (the city of Nelson) on deep soil (class D (New Zealand Standards 2004)). It was assumed that the height of the floors is 3.5 m for all storeys which means the building is 14 m high. The structure is considered a high-importance building with an importance level of 3.0. Fig. 1 illustrates the general configuration of the structures and arrangement of the lateral elements. Four brace lines/shear walls are considered at the corners of the building with the arrangement shown in the assumed dead loads (permanent loads), including internal walls and ceilings, which are 1.5 kPa for the first three floors and 1.2 kPa for the roof. The live loads (imposed loads) considered for design and analysis are 2 kPa, 1.5 kPa and 0.5 kPa for the first floor, second/third floors and the roof, respectively. Accordingly, the calculated seismic forces are 105 tonnes for floor 1, 96 tonnes for floors two to four and 67 tonnes for the roof.

For the rocking wall option, Cross Laminated Timber (CLT) panels made of five layers with 50 mm thickness for each layer have been adopted. Each panel has three longitudinal and two transverse layers. Resilient Slip Friction Joint (RSFJ) hold-downs are considered for the walls to utilise a rocking mechanism. It has been assumed that RSFJ hold-downs are located at the bottom corners of each wall (rocking toes) to provide the required over-turning moment capacity. This concept has been experimentally (Hashemi et al. 2017) and numerically (Hashemi et al. 2020) investigated and verified as an efficient concept for rocking mass timber walls. For the braced frame option, RSFJ timber braces have been considered. In this configuration, RSFJ devices are attached to a timber body (normally Glue-laminated timber members), forming a resilient self-centring brace. This concept has been experimentally (Yousef-Beik et al. 2021) and numerically (Hashemi et al. 2019) investigated and deemed as a feasible and efficient solution.

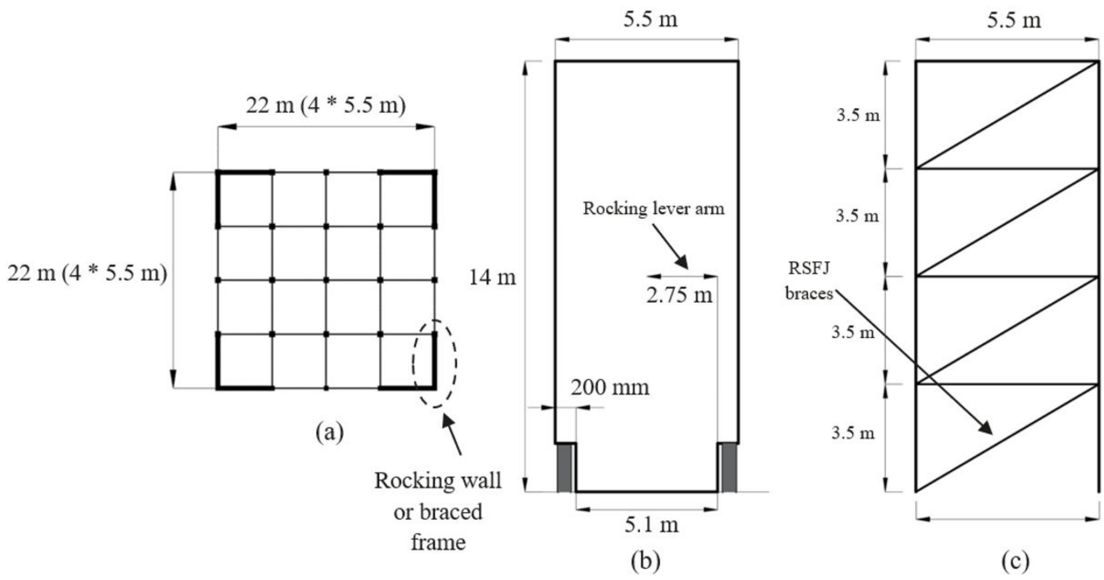

Fig. 1 General configuration of the case study structure: (a) plan view (b) rocking wall option (c) braced frame option

4 ANALYSIS OF THE STRUCTURES

The Direct Displacement-Based Design (DDBD) approach has been recognised to be an efficient method for the type of structure and type of devices considered in this research. The following steps are used to analyse the system and assess the seismic demand:

1- Determine the maximum allowable lateral drift:

Most international building standards allow for a maximum of 2.5% lateral inter-story drift for design levels seismic actions. However, for this study, a target drift limit of 1% is considered to reduce damage to the structure.

2- Represent the building as an equivalent Single Degree of Freedom (SDOF) structure:

A key assumption in the DDBD methodology is that every structure is simplified and represented by an equivalent Single Degree of Freedom (SDOF) structure. Following the recommendations made by Priestly et al., the design deflection value Δ_d=0.11 m, effective mass m_e=300 tonnes and effective height H_e=9.95 m are determined.

3- Specify the equivalent viscous damping ratio of the building

For this study, an elastic damping value of ξ_{el} = %2 is considered for the building. Previous studies demonstrated that the hysteretic damping value of self-centring systems (e.g. systems with flag-shaped hysteretic response) might vary between 8% to 20%. There should be values outside of this range as well, but this range should give a good starting indication. Therefore, a hysteretic damping value of ξ_{hyst} =15% is assumed at this step. The following equation is used to calculate the spectral scaling factor where A_1 is the energy preserved by the structure, and A_2 is the input seismic energy to the system. The scaling factor is then calculated as R_{eq}= 0.61. Consequently, the effective period is found as T_e = 0.98 seconds.

$$\xi_{hyst} = \frac{2\,A_1}{\pi\,A_2} \qquad (1)$$

4- Determine the design base shear

Following the DDBD procedure, effective stiffness and the design base shear are K_e = 12312 kN/m and V_b =1226 kN. The design base shear is distributed up the height of the structure using the distribution equation provided in (NZS 1170.5 2004).

5 NUMERICAL MODELLING

5.1 Scenarios for investigation

For this study, five levels of serviceability thresholds have been investigated. The serviceability threshold refers to the minimum resistance that the structure needs to remain linear and elastic while the lateral response drifts are also kept in the elastic range (%0.33 drift). The first level is the conventional definition of SLS1 in the standard (New Zealand Standards 2004). As mentioned in the first section of the paper, this limit is the minimum requirement that the structures need to have to be able to meet the serviceability performance requirements. As mentioned in section 4, the ULS and SLS1 design base shears for the structure are 1226 kN and 722 kN, respectively. This means the ratio of SLS1 demand over ULS demand is 722/1226= 0.59. This ratio is named as the Serviceability Ratio (SR) for simplicity. For this paper, four other serviceability ratios are defined and for each one, SR is incrementally increasing. These ratios are SR=0.65, 0.70, 0.75 and 0.80. This means that for each of the investigated systems (e.g. rocking shear wall and braced frames), the F_{slip}/F_{ult} of the devices should be the same as the respective SR. Also, note that for simplicity, structural models with rocking walls are named as RW and the braces frames as BR. Table 1 summarises the structural systems, including the parameters used to model the devices as link elements.

Table 1: Lateral elements.

Model	SR ratio	Link name	F_{slip} (kN)	F_{ult} (kN)	$F_{restoring}$ (kN)	$F_{residual}$ (kN)	Initial stiffness (kN/mm)	Pre-compression displacement (mm)	Loading stiffness (kN/mm)	Unloading stiffness (kN/mm)	Stop displacement (mm)
RW-1	0.59	HD-1	295	500	150	89	200	-80	4.7	1.4	55
RW-2	0.65	HD-2	325	500	150	98	200	-90	4.0	1.2	55
RW-3	0.70	HD-3	350	500	150	105	200	-110	3.4	1.0	55
RW-4	0.75	HD-4	375	500	150	113	200	-146	2.8	0.8	55
RW-5	0.80	HD-5	400	500	150	120	200	-200	2.3	0.7	55
BR-1	0.59	Brace-4	82	140	113	66	16	-28	2.5	0.8	28
		Brace-3	144	245	74	43	29	-28	4.4	1.5	28
		Brace-2	186	315	95	56	37	-28	5.6	1.9	28
		Brace-1	221	375	113	66	44	-28	6.7	2.2	28
BR-2	0.65	Brace-4	91	140	42	27	16	-41	2.1	0.6	28
		Brace-3	159	245	74	48	29	-41	3.7	1.1	28
		Brace-2	205	315	95	61	37	-41	4.8	1.4	28
		Brace-1	244	375	113	73	44	-41	5.7	1.7	28
BR-3	0.70	Brace-4	98	140	42	29	16	-52	1.8	0.5	28
		Brace-3	172	245	74	51	29	-52	3.2	1.0	28
		Brace-2	221	315	95	66	37	-52	4.1	1.2	28
		Brace-1	263	375	113	79	44	-52	4.9	1.5	28
BR-4	0.75	Brace-4	105	140	42	32	16	-67	1.5	0.5	28
		Brace-3	184	245	74	55	29	-67	2.7	0.8	28
		Brace-2	236	315	95	71	37	-67	3.4	1.0	28
		Brace-1	281	375	113	84	44	-67	4.1	1.2	28
BR-5	0.80	Brace-4	112	140	42	34	16	-90	1.2	0.4	28
		Brace-3	196	245	74	59	29	-90	2.1	0.6	28
		Brace-2	252	315	95	76	37	-90	2.7	0.8	28
		Brace-1	300	375	113	90	44	-90	3.3	1.0	28

5.2 Nonlinear Static Pushover (NSP) analysis

To further investigate the performance of the systems described in the previous sections, two separate three-dimensional models have been developed in SAP2000 (see Fig. 2) and subjected to Nonlinear Static Pushover (NSP) analysis. The purpose of performing this type of analysis was to properly calibrate the models based on the information provided in Table 1 and to verify the assumption made in the DDBD exercise about hysteretic damping. Hold-downs are modelled by "Damper – Friction Spring" elements working in parallel with a gap element. The friction spring elements' parameters are specified per the information in Table 1 for RW-1 to RW-5. The gap element is used to define the rocking toe (e.g. the foundation). This modelling technique has previously been verified by comparing the numerical data with experimental results on actual RSFJ devices and on a rocking CLT wall with RSFJ hold-downs (Hashemi et al. 2017). The walls are decoupled from the columns and are attached to the beam with the vertical movement degree of freedom released. Furthermore, the horizontal degree of freedom of the nodes where the hold-downs are located is restrained to represent shear keys.

Fig. 3(a) and Fig. 3(b) respectively show the results for RW-1 and RW-5 models that have lowers and highest SR values. The damping ratio of the RW-1 model is calculated as %15.6 which is higher than the value assumed for the DDBD analysis (%15). Therefore, the assumption made is verified. Note that the hysteretic damping ratio of RW-2 to RW-5 models is relatively higher than RW-1 given that those systems have higher SR value, thus higher F_{slip} for the hold-downs. Fig. 3(c) and Fig. 3(d) respectively show the results for BR-1 and BR-5 models that have lowers and highest SR values. Similarly, the damping ratio of the BR-1 model using Equation 4 (see Fig. 3) is calculated as %15.1, which is higher than the value assumed in step 3 of section 2 (%15). The hysteretic damping ratio of BR-2 to BR-5 models is relatively higher given that those systems have higher SR value, thus higher F_{slip} for the hold-downs.

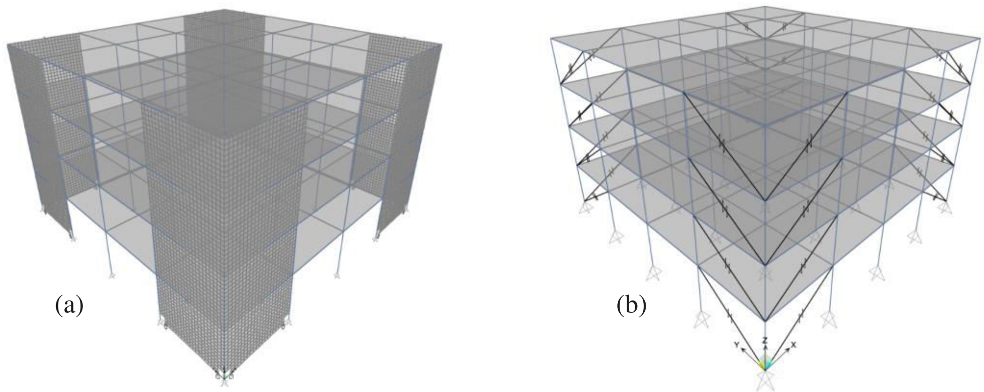

Fig. 2 *Numerical models: (a) rocking wall option (b) braced-frame option*

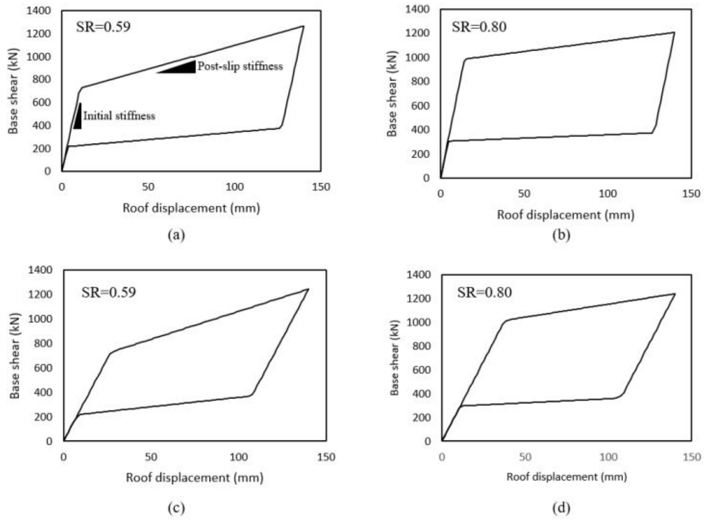

***Fig. 3** The results of the cyclic NSP analysis: (a) RW-1 (b) RW-5 (c) BR-1 (d) BR-5*

5.3 Nonlinear dynamic analysis

In this section, all the developed building models are subjected to nonlinear dynamic time-history simulations. Seven ground motions are selected and scaled based on the procedure outlined in the New Zealand standard (NZS1170.5 2004). These events are scaled for two limit states, ULS representing design level demands and MCE representing over-strength demands. The MCE level records are specifically used to determine the over-strength displacement requirements for each system and the effect that SR value has on the performance.

5.3.1 RW models

Fig. 4(a) displays the maximum inter-storey drifts recorded for the RW-1 to RW-5 models. The average response drift of the seven events is %0.74 for the RW-1 model and %0.55 for the RW-5 model. The fact that all average values are below the target design drift adopted for the DBD procedure (%1) shows that performed DBD exercise is valid and slightly on the conservative side. The average response drift for the RW-2 to RW-4 are respectively %0.67, %0.64 and %0.58. This shows that as the SR increases, response drifts decrease, which can be attributed to a higher damping ratio. Also, the higher post-slip stiffness of the RW-1 model (compared to the RW-5 model) does not seem to have a significant positive effect on the response. It is notably observable that for RW-1 with the minimum SR value (0.59), except for one event, all response drifts are higher than %0.33 drift (the maximum acceptable drift for SLS1 actions). For the RW-2 and RW-3 models, the response drifts are lower than %0.33 for two of the events. For RW-4, the response drifts are lower than %0.33 for three of the events and finally, for RW-5, the simulations show that for four of the events, drifts are lower than %0.33. As per the standard (NZS1170.5 2004), when the response drifts are lower than %0.33, it means the system remains serviceable. The fact that for the RW-5 model (with the SR=0.8), the structure remained serviceable under four events (compared to the RW-1 model, only one of the analysed cases showed this performance), demonstrates that as the serviceability ratio (SR) increases, the likelihood of severe damage decreases.

Fig. 4(b) shows the response drifts for the MCE-scaled events. The average drift for the RW-1 model is %1.69, while for the RW-5 model is %1.39. This observation agrees with the findings from the ULS-scaled events. In other words, relatively higher hysteretic damping of the RW-5 model (compared to RW-1) negates the potential negative effect of the lower post-slip stiffness and results in lower drifts under MCE actions. It has been recommended that for systems with RSFJ devices, at least %50 extra displacement capacity is provided by the devices because of the secondary fuse function (refer to (Hashemi et al. 2018) for the details). This means if the target design drift is %1, the secondary fuse feature guarantees that there is displacement capacity available in the devices up to about %1.5. However, it can be seen that for the RW-1 model, at least for five of the analysed cased (Darfiled, Duzce, Northridge, Kobe and Christchurch), the maximum inter-storey value is beyond the assumed %1.5, and even the average is %1.69. As can be expected, the situation gets better as the SR increases. For example, the results for the RW-5 models show that only for two of the events the response drift is higher than the %1.5 (Northridge and Christchurch). Furthermore, in two of the cases for RW-1 analysis, the response drift is about %2. As a result, it is recommended for self-centring rocking wall systems that implement dissipative devices (such as RSFJs), an overstrength margin of two times the design displacement is considered for the design so that the system can tolerate higher than design actions (e.g. MCE actions) properly and avoid collapse.

5.3.2 BR models

Fig. 5 shows the response inter-storey drifts for ULS and MCE for the BR models. It is observable from the figures that generally, the brace frame option yielded smaller drifts compared to the shear wall options. The average drift for BR-1 and BR-5 models are, respectively %0.62 and %0.44 which are about %20 less than the drifts recorded for the RW-1 and RW-5 models. Additionally, as the SR value increases, the recorded response drift decrease that agrees with the findings from the previous section about the RW-1 to RW-5 models. The number of events that the recorded drift is under 0.33% is four for the BR-1 model, five for the BR-2 and BR-3 models and six for the BR-5 model. This means as the SR value increases, the number of design-level events that the structure can remain serviceable (as per the standard criteria (NZS 1170.5 2004)) increases.

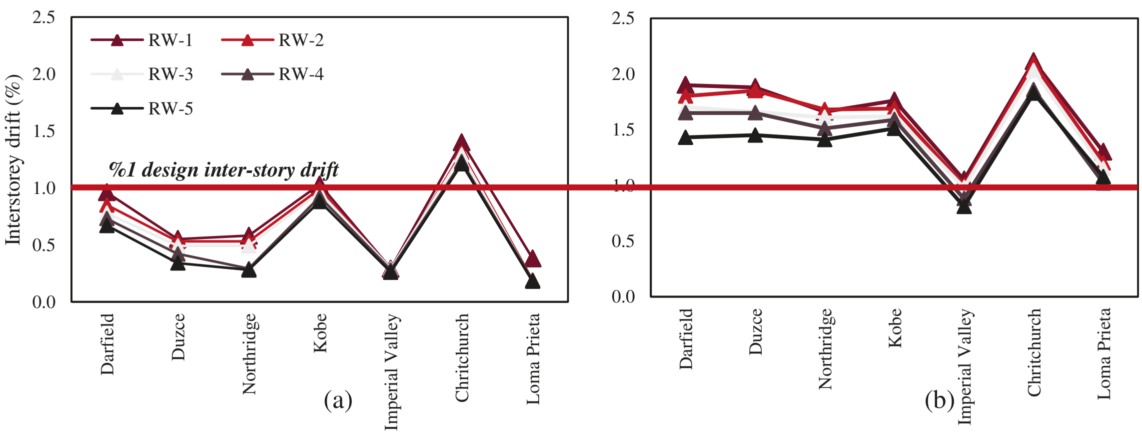

***Fig. 4** Response inter-storey drifts for the RW models: (a) ULS (b) MCE*

Fig. 5(b) provides the response drifts for the MCE-scaled events. The average drift for the BR-1 is %1.31, while for the BR-5 model is %0.9. These values are almost %25 less than the values achieved for the rocking wall option (RW-1 and RW-5 models). Furthermore, the average drift decreases as the SR increases. As expected, the largest recorded MCE drifts are related to the BR-1 model (with the lowest SR=0.59 value) that are %1.57 for Darfiled and %2.1 for Christchurch. This means that the over-strength margin required for the MCE events (e.g. displacement capacity available in the system) should be at least 2.0 to be on the safe side but can be reduced to a lower number when SR is increased above 0.65.

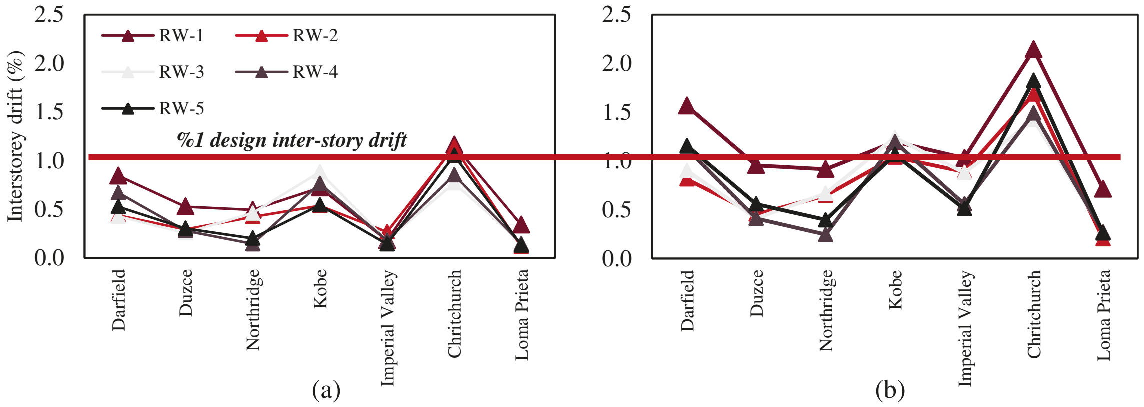

***Fig. 5** Response inter-storey drifts for the BR models: (a) ULS (b) MCE*

6 CONCLUSIONS

This study presents an investigation of the effect of over-strength margin and serviceability limit state on the overall seismic performance of low-damage systems under design level earthquakes (ULS) and collapse limit state (MCE actions). Two types of generic structural responses are considered for the study, braced frames and rocking shear walls. Numerical models for both concepts are developed and subjected to nonlinear dynamic time-history analyses. From the results of the analyses, the following concluding points can be drawn:

- The overall performance of both systems improved when the serviceability ratio (SR) increased. This can be related to higher resistance before the system mobilises.
- The braced frame option demonstrated about %20 less response drifts for ULS and about %25 less response drifts for MCE events. This can be attributed to the more uniform distribution of the lateral loads for the braced frames and/or the effect of higher modes on the rocking shear walls. There is potential for further investigations in this area.
- The design over-strength margin for selecting systems is recommended to be at least 2.0 for rocking shear wall systems, regardless of the SR value. It means two times the ULS design displacement should be available in the system before the lateral load resisting system fails. For braced frames, this value can be decreased to a lower number providing the SR value is above 0.65.
- The lower post-yield stiffness of the systems with higher SR (compared to systems with lower SR but higher post-yield stiffness) had no negative effect on overall seismic performance.

The findings of this study demonstrated different aspects of the seismic performance of self-centring systems. Therefore, there is merit to investigate this further and advancing this research.

7 ACKNOWLEDGEMENTS

The author would like to thank the faculty of engineering at The University of Auckland and Toka Tū Ake EQC (Earthquake Commission) for their support of this research.

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