Feasibility of Capacity Spectrum method for analysis of low damage structural systems with friction connections

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ABSTRACT

The use of friction connections for low damage design of new structures has been increasing over the past few years. These connectors have a high energy dissipation ratio and are relatively economical to fabricate and install. Furthermore, their performance has been the subject of many research projects in New Zealand that affirmed their efficiency and applicability. Accordingly, these devices have been implemented in many projects in Aotearoa and overseas. Given that buildings with friction connections are not covered by current building standards, they are generally classed as alternative solutions. Therefore, a robust and reliable approach is required so that the analysis can be performed and checked. One of the simple and efficient methods to analyse structural systems is the Capacity Spectrum method. This method, which is covered by many national and international guidelines, has been frequently used by the engineering community for seismic assessments and retrofit projects. However, little attention has been paid to its applicability for new builds. This paper investigates the feasibility of the capacity spectrum method for low damage structural systems with friction energy dissipators. The non-linearity of the systems is limited and localised within friction devices. Different structural types, such as rocking walls and braced frames, are modelled and subjected to non-linear analyses. The results show that this method can be simple and efficient for the analysis and design of such structures.

1 INTRODUCTION

In the past decade, there has been a surge in the uptake and implementation of friction connections for the construction of new structures. These connections enable structures to fall within the Low Damage Design (LDD) philosophy, where after the Canterbury earthquakes of 2010-2011, the need and implementation for such design philosophy was deemed necessary. After the Christchurch event, many of the multi-story structures were severely damaged and rendered unrepairable and were demolished. Some of those structures were designed and built with the ductile design method in mind, where controlled and predictable damage (yield) occur at particular members. Despite this, due to the experienced peak ground accelerations beyond the code and design levels, the extent of damage exceeded the ductile design, leading to excessive damage and in some cases, the collapse of structures. Therefore, along with the significant financial and human cost that earthquakes entail, it has motivated researchers to innovate and governments to invest and reconsider the ductile design towards a low damage design methodology. Designs are geared toward establishing systems that allow structures to remain functional or with nominal damage and must be sensibly economical to repair with minimal interruption of service after an earthquake. Current LDD aims towards limiting the damage and non-linearity within the Lateral Load Resisting System (LLRS), isolating and capacity protecting the gravity system. This is often achieved by adopting modern passive control systems, including LDD connections (joints) and utilising energy dissipation devices.

1.1 Low damage friction connections

The concept of a friction energy dissipator was initially introduced by (Pall et al. 1980) and later proposed by (Popov et al. 1995) for steel moment resisting frames using symmetric slotted bolted plates. Since then, numerous friction damper configurations have been created, including the asymmetric sliding hinge joint (Clifton et al. 2007). A symmetric slip-friction joint consisting of slotted sliding steel plates clamped via bolts and Belleville disks was introduced and experimentally tested by (Loo et al. 2014b; Loo et al. 2014a). It provides substantial energy dissipation while maintaining strength and stiffness. However, conventional friction dampers do not possess any self-centring capabilities and most often require a secondary mechanism or element to provide that, such as post-tensioned cables, amongst others. Hence the development of innovative and modern friction energy dissipators in which combine both mechanisms together. The friction rings spring developed and manufactured by Ringfeder Power Trans-Mission Gmbh (Helm et al. 2022) consists of two sets of machined outer and inner springs with tapered mating surfaces cramped into a pistol. Friction ring springs provide adequate energy dissipation independent of loading rate while providing self-centring. The Resilient Slip Friction Joint (RSFJ) was introduced in 2015 (Zarnani and Quenneville 2015). This device can be tuned to different levels of force and displacements as required by design, owing to its scalable parts and properties. RSFJ consists of two grooved cap plates, two grooved middle plates, disk springs, and pre-stressed bolts (or rods). Energy is dissipated via friction in between the sliding clamped plates, while the semi-prestressed disc springs, in conjunction with the grooved profile of the plates, restore the device to its original position (see Figure 1).

Figure 1: Low damage friction energy dissipators: (a) slip-friction joint; (b) friction rings springs; (c) resilient slip friction joint

1.2 Research intent

There are currently no building codes that cover buildings with friction connections, so designs incorporating friction connections are considered alternative solutions and subject to peer reviews. When it comes to evaluating and assessing the behaviour and performance of a non-linear multi-degree-of-freedom (MDOF) structures, (Priestley et al. 2007) encourage engineers and practitioners to focus their effort on Dynamic

Time History Analysis (DTHA) rather non-linear static methods such as Non-linear Pushover Analysis (NPA) or recently the centre of research multi-modal non-linear pushover analysis (MNPA). For some MDOF structures, non-linear static methods may not be able to predict important response parameters. NPA has a few critical limitations, such as: analysis is restricted to the fundamental mode response of the structure; as a result, it does not capture the dynamic effects and amplification of demands on the structure caused by higher mode effects. Moreover, the actual displacement demand of the structure cannot be determined; only the assumptions made in the Displacement Based Design (DBD) process and the assumed deformed shape can be verified. Although authors agree with the above statement, they yet argue that the time and computational effort required by DTHA can often be a potential barrier to adoption; hence a more simple and more efficient method utilising the NPA, which would also satisfy the above limitations, would be ideal. A robust and reliable approach is required so that the analysis can be performed and checked. Taking all these factors into consideration, the Capacity Spectrum Method becomes relevant.

2 CAPACITY SPECTRUM METHOD

One of the simple and efficient methods to analyse structural systems is the Capacity Spectrum Method (CSM). This method, which is covered by various national and international guidelines, has been frequently used for seismic assessments and retrofit projects. However, little attention has been paid to its applicability for new builds. The capacity spectrum method was initially codified and published in FEMA 356 and ATC-40 (as Coefficient Method), and later an improved procedure for CSM was published in FEMA 440. C2: Assessment procedures and analysis techniques by (Ministry of Business Innovation and Employment 2018) provide a similar and more simplified method of CSM for seismic assessment of existing buildings in New Zealand. CSM can be conducted by plotting the backbone curve of the system derived from NPA against the modified (reduced from 5% damping) Acceleration-Displacement Response Spectrum (ADRS) for the Serviceability limit state (SLS) and Ultimate limit state (ULS) (Figure 2). An ideal system should have its minimum yield point (device or fuse activation point) just above the SLS ADRS line and intersect the ULS ADRS line (performance point) above the design force and displacement levels. According to FEMA 440 and C2, higher modes are considered insignificant, and the NPA can be used standalone if the square root of the sum of the squares (SRSS) of shear on any one storey from modes that incorporate at least 90% of the mass does not exceed 130% of story shear from an analysis considering only the first-mode response of the structure. Additionally, C2 indicates that higher modes can be effective if the fundamental period of the structure exceeds approximately one second or ineffective if 60% or more of the structure's mass participation is captured by the first mode in a particular direction.

In structures that adopt energy dissipation devices as viscous damping, Rayleigh damping (EQ 1), where damping is dependent and proportional to a linear combination of its mass (M) and stiffness (K), is a suitable damping model and has been used in most commercially available software.

$$C = \alpha M + \beta K \tag{1}$$

where C = damping matrix; M = mass matrix; K = stiffness matrix; α = mass proportional Rayleigh damping coefficient; and β = stiffness proportional Rayleigh damping coefficient.

3 METHODOLOGY

In order to investigate the feasibility of the capacity spectrum method for new low damage structural systems with friction energy dissipators, different structural types have been selected and tabulated in Table 1. Each case study has been subject to NPA, and relative CSM is conducted and compared with its performance via DTHA.

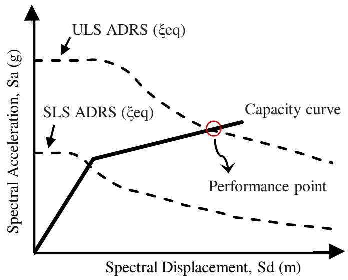

Figure 2: Capacity spectrum method (CSM)

All case studies employ self-centring friction energy dissipators and the non-linearity of the systems is limited and localised within them. Numerical analyses are carried out in ETABS (Computers and Structures Inc), and the friction devices are modelled using "Damper-friction spring" link properties. This modelling technique has already been verified by several earlier studies (Hashemi et al. 2020b; Hashemi et al. 2020a). All structures are assumed to be located in Wellington CBD, with ULS design considerations; annual probability of exceedance of 1/500 years, site soil classification of D, near fault factor of 1, and maximum allowable drift limit of 1.5%. SLS design considerations; annual probability of exceedance of 1/25 years, site soil classification of D, near fault factor of 1, and maximum allowable drift limit of 0.33%. The case study structures are summarised in Table 1 and illustrated in Figure 2. The structural performance factor (S_P) has been taken as a conservative value of 0.85 for the purpose of this study. While it can be argued that since the non-linearity is solely confined to the friction connections, which would provide predictable and reliable behaviour, an S_P value of 0.70 would suffice. Nevertheless, to preserve the ground motion accelerations and provide a moderate set of records for studying the full impact of the DTHA on the structures, the value of 0.85 has been considered to be a conservative value as it represents the middle of the range of 0.7 to 1.0.

Table 1: Summary of case study structures

Case Study	**Code**	**Number connections**	**Height (m)**
Single Degree of Freedom	SDOF	1	4.5
Simple Brace Frame	SBF	1	4.5
Rocking Wall with friction connection hold-downs	RW	2	8.0
Three-storey Braced Frame	BF-3	3	11.5
Five-storey Braced Frame	BF-5	5	18.5

For all models, the Displacement-Based Design (DBD) was used to determine seismic demand (Priestley et al. 2007). Detailed seismic design procedure for LDD structures with resilient friction dampers using DBD methodology is provided in (Hashemi et al. 2020b; Hashemi et al. 2020a). Based on the obtained backbone curves from cyclic pushover analyses, Jacobsen's method (hysteresis area) is used in order to calculate hysteretic damping for each system (Chan et al. 2021). As can be seen the Figure 4, A_1 is the hysteresis area enclosed within the flag-shaped loop, and A_2 is the total seismic energy input.

$$\xi_{hysteresis} = \frac{2A_1}{\pi A_2} \tag{2}$$

CSM capacity curves are obtained from NPA of each system based on the maximum drift of 1.5 percent used during the DBD procedure. To perform DTHA, ten ground motion records are selected and scaled according to NZS1170.5 (Standards New Zealand 2004), and the analysis variables from each case study are compared using the 'mean of more than 7 records' approach (Bradley 2014). Note that this is different to the 'max of three' approach that the New Zealand standard has prescribes but should well capture the aspects of the performance that this research is intended for. The case study structures are presented in Table 1 and illustrated in Figure 3.

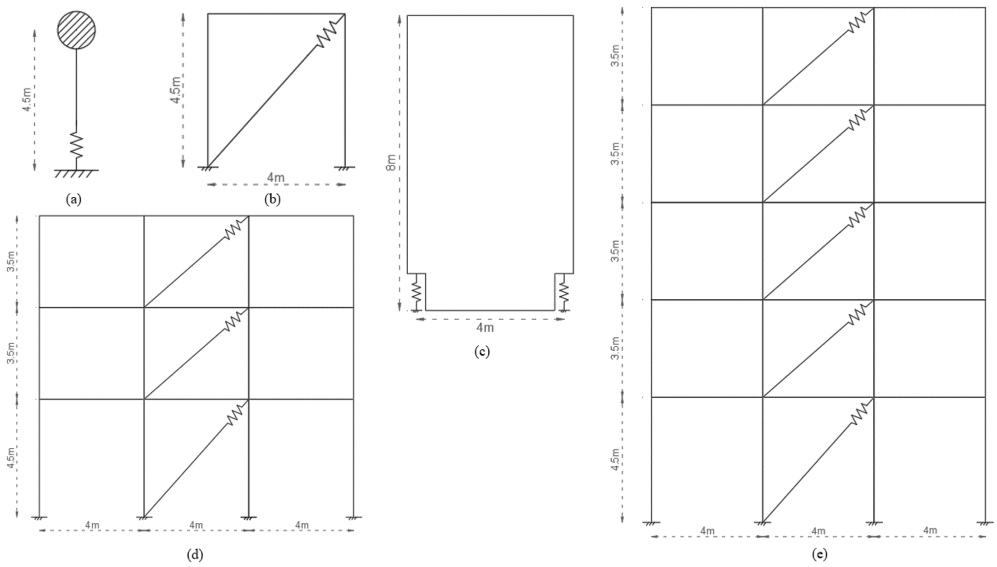

Figure 3. Case study structures: (a) SDOF; (b) SBF; (c) RW; (d) MBF-3; (e) MBF-5

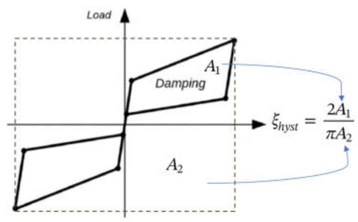

Figure 4: Hysteretic damping via simplified area method derived from flag-shaped cyclic pushover curves.

Table 2: Selected 10 ground motion records.

Event Name	Country	Year	Magnitude	Fault Mechanism
ChiChi	Taiwan	1999	7.62	Reverse Oblique
Christchurch	New Zealand	2011	6.2	Reverse Oblique
Darfield	New Zealand	2010	7.0	Strike Slip
El Centro	United States	1940	6.95	Strike Slip
Kaikoura	New Zealand	2016	6.2	Reverse Oblique
Kern County	United States	1952	7.36	Reverse
Kocaeli	Turkey	1999	7.51	Strike Slip
Lytle Creek	United States	1970	5.33	Reverse Oblique
Valparaiso (Llolleo)	Chile	1985	7.8	Strike Slip
Victoria (Chihuahua)	Mexico	1980	6.33	Strike Slip

4 RESULTS AND DISCUSSIONS

As a result of the above methodology, the results of each case study structure are presented individually in this section, which is then followed by a discussion whereby it is discussed whether CSM is feasible and applicable for the analysis of low damage structural systems with friction connections. A plot of CSM against DTHA is made by plotting the largest force-deformation cycle of each record acquired from DTHA against the capacity curve derived from NPA so as to determine the relationship between the two.

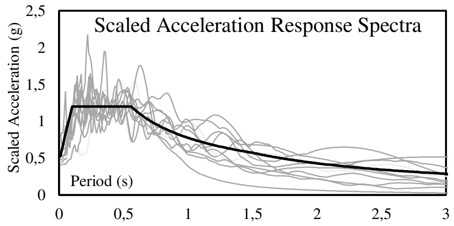

Figure 5: Selected 10 scaled ground motion records to NZS1170.5.

4.1 Single Degree of Freedom (SDOF) system

Considering that the DBD design consists of simplifying a MDOF structure into a SDOF, it is entirely logical that the capacity curve of a SDOF should coincide with that of the DBD design value, as is the case for this case study. Dynamic time history results, however, may fall short of design values, possibly due to the simplicity of the system. Based on the CSM curve, DTHA has been captured very well, and no records have reached or exceeded design levels (e.g. design drifts or base shears). Displacement demands achieved by performing DTHA are approximately half of the design drift of 1.5%.

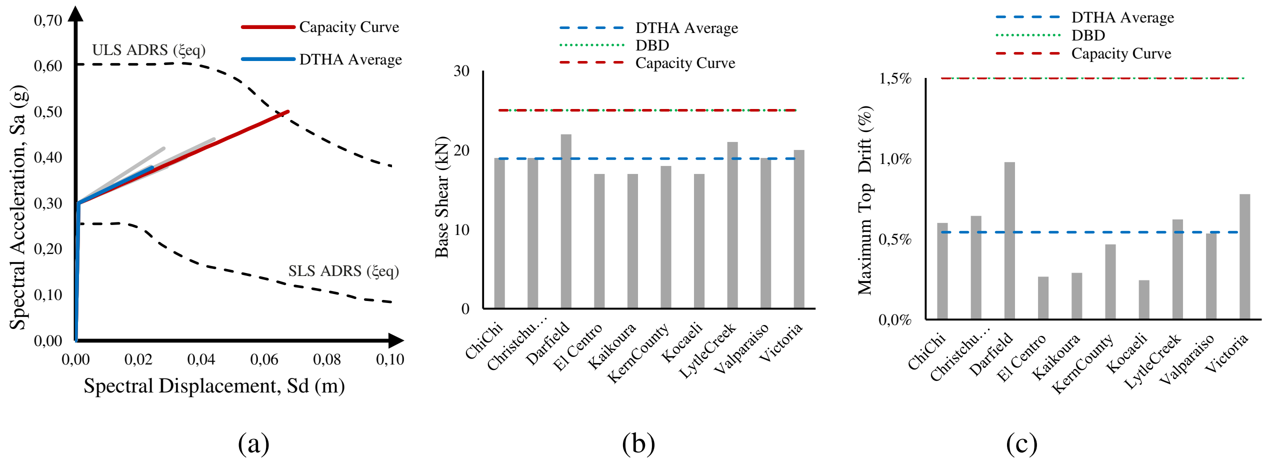

Figure 6: SDOF case study: (a) CSM against DTHA; (b) base shear comparison (c) maximum top drift comparison

4.2 Simple Brace Frame (SBF) system

Under both the NPA and DTHA, the behaviour of the structure is very consistent. In other words, the secondary stiffness of the system has been maintained and the structure has behaved consistently across all records. The CSM curve captures DTHA very well, and due to simplicity and the absence of dynamic effects, no records have exceeded the design levels. On average, the force demands of DTHA is 14% lower than the ultimate force of the CSM. Only two records have approached the design drift limit; the rest of the records are well within the displacement demand limits, with an average drift of 0.9%.

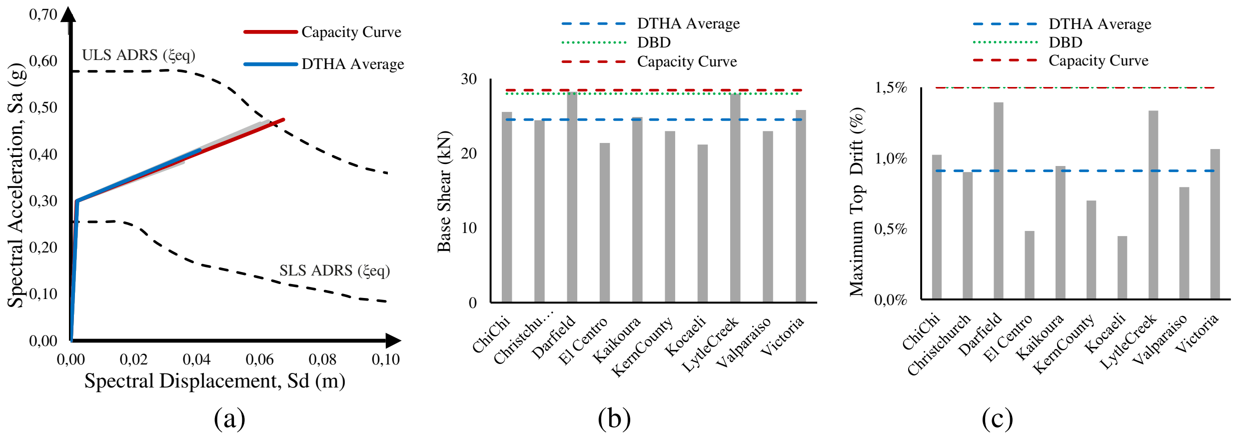

Figure 7: SBF case study: (a) CSM against DTHA; (b) base shear comparison (c) maximum top drift comparison

4.3 Three-storey Braced Frame (BF-3)

As the structures become taller and more complex, the presence of dynamic effects becomes apparent. The secondary stiffness of the system also begins to vary with each ground motion. The secondary stiffness of a system is affected by the magnitude, frequency, and duration of the ground motion excitation. Consequently, the structural response of taller and more complex structures is more likely to be influenced by these parameters and therefore to exhibit a varying structural response. Average DTHA is approaching the capacity curve peak; however, based on the CSM, it is apparent that the demands are adequately captured. A single record (Kaikoura) has exceeded the force demand beyond the capacity curve prediction by a very narrow margin of 3% which is neglectable (see Figure 8b). DTHA's force demands are on average 10% lower than the CSM's ultimate force. Despite one record (Victoria) exceeding the design drift limit by 0.1% (equivalent to 11mm), the remaining records are well within the displacement demand limits, with an average drift of 1.15% (Figure 8c). As this study takes a 'mean of records' approach, the small number of records exceeding the CSM with narrow margins is not considered critical. However, even if the maximum of responses are considered, CSM managed to capture the actual dynamic behaviour by a very small margin. Therefore, for the design to be conservatively on the safe side, an extra displacement capacity of %10 can be considered. Note that this number (%10) is less than the over-strength margin normally considered for the design of such structures.

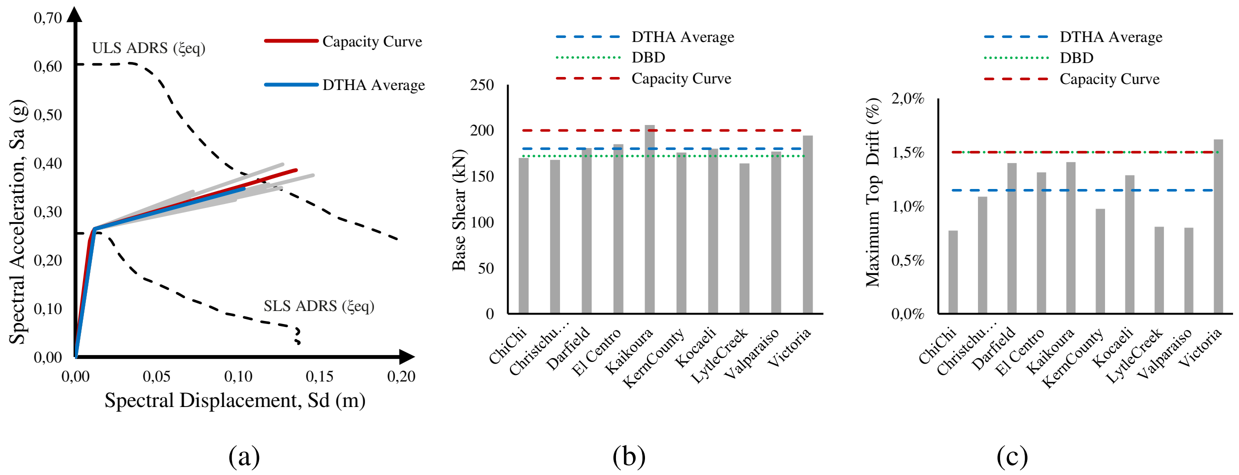

Figure 8: BF-3 case study: (a) CSM against DTHA; (b) base shear comparison (c) maximum top drift comparison

4.4 Five-storey Braced Frame (BF-5)

For BF-5 case, similar to the case of BF-3, the dynamic effects are evident and the average DTHA is much closer to the peak of the capacity curve; however, the CSM has been able to adequately capture and predict the structure's seismic demand. On average, DTHA's force demands are about 5% lower than the CSM's maximum force (Figure 9b). However, two records of Christchurch and Darfield have exceeded the CSM peak by about 9% and 4%, respectively. There are no concerns regarding the performance of the structure due to the limited number and margin of exceedance, in addition to the fact that the average is still below the peak of the CSM. Moreover, since the structure is capacity protected with an appropriate overstrength factor, it must be able to withstand reasonable excess demand, where the excess demand is taken up by LDD connections that undergo yielding or enter the secondary fuse phase (over-strength mechanism). The displacement demands all fall under the CSM 1.5% limit line, and no records have surpassed it (Figure 9c). Similar to the BF-3 case, a %10 extra displacement capacity can be considered for safety.

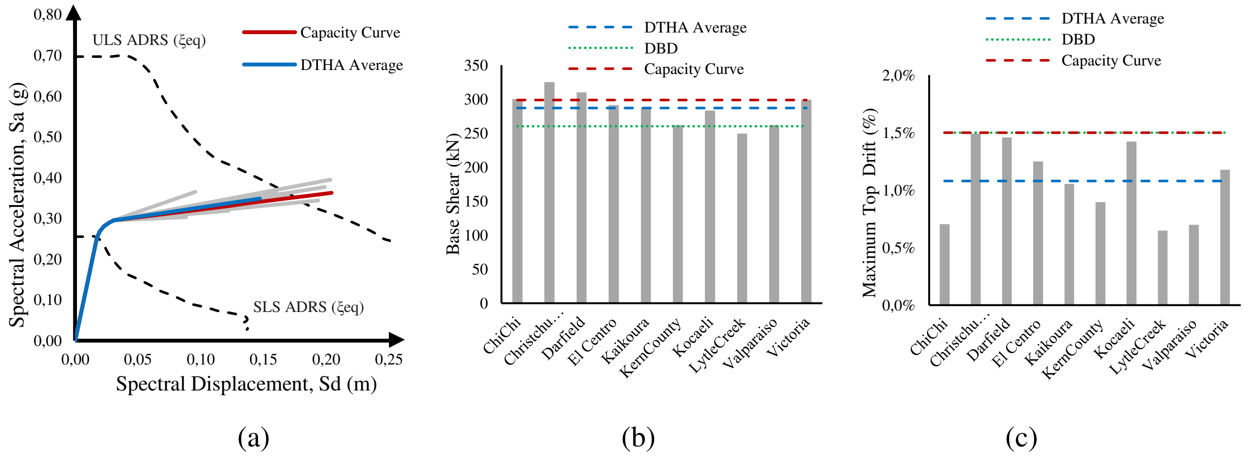

Figure 9: BF-5 case study: (a) CSM against DTHA; (b) base shear comparison (c) maximum top drift comparison

4.5 Rocking Wall system with friction connection hold-downs (RW)

This case study analysis demonstrates that walls are more susceptible to higher mode effects and increased demands due to dynamic amplification. As a result, the behaviour of the system is not as uniform as that of braced systems and is more likely to be affected by the characteristics of the ground motion. It may be noted that the CSM has adequately captured the behaviour of the system, as the DTHA average is just below the CSM line (Figure 10b). This is despite the fact that a few records have exceeded the capacity curve force by a small margin. Among the five records exceeding the CSM ultimate force, the maximum exceedance was found under Lytle Creek ground motion, where it exceeded by 9%. Furthermore, contrary to other cases, three records exceeded the maximum drift limit set by DBD design, where Darfield, Kocaeli, and Kaikoura reached maximum drifts of 1.8%, 1.8%, and 1.7%, respectively. It is evident from this that when using LDD friction connections as wall hold-downs, an appropriate overstrength factor needs to be considered. Furthermore, the friction connection should be able to accommodate a secondary mechanism or reserved displacement capacity in the exceptional event that the displacement demand reasonably exceeds the drift limit set by design. In other words, for wall structures, an extra %20 may be required to cover rare cases when the demands are higher than the design.

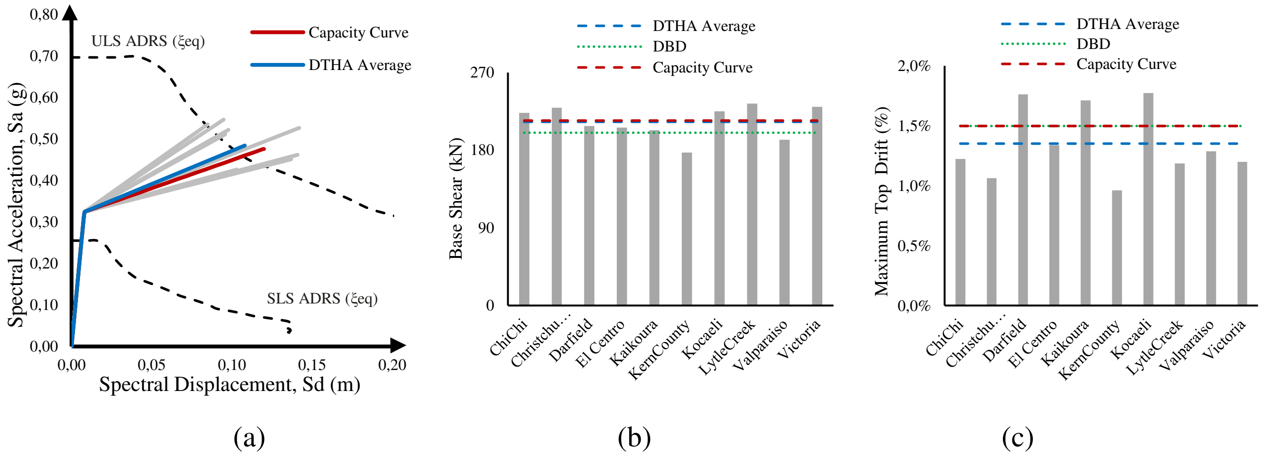

Figure 10: RW case study: (a) CSM against DTHA; (b) base shear comparison (c) maximum top drift comparison

4.6 Further discussion

In all case study structures, self-centring was observed, largely due to the reliable behaviour of flag-shaped friction connections as the only inelastic component within the models. As can be seen from the results of CSM, the capacity curve tends to provide an envelope for DTHA, which is a conservative approach. As a result, CSM can be utilised efficiently and confidently for the analysis of regular structures utilising LDD friction connections with hysteretic damping ranges between %5 and %15, such as those described in this paper. Consequently, area-based equivalent viscous damping ratios do not need to be corrected by correction factors suggested by Priestley (Priestley et al. 2007) for this range of hysteretic damping (5% to 15%). It is evident that the gap between average DTHA and CSM narrows as the height and complexity of the structure increase. Figure 11 illustrates the ratio of DTHA average over CSM peak seismic demand for each case study system. By examining the graph, one can conclude that CSM is an appropriate method for analysing brace and wall structures utilising LDD friction connections, as well as a method that will adequately predict and capture seismic demands. In the case of simple structures up to five storey bracing structure, CSM provides a conservative approach, whereas for wall structures, CSM provides an accurate yet marginal prediction of seismic demands. Having said that, to adopt a conservative yet safe design, an overstrength margin (e.g. extra lateral displacement available in the system) of %10 and %20 are suggested for braced frames and wall structures, respectively.

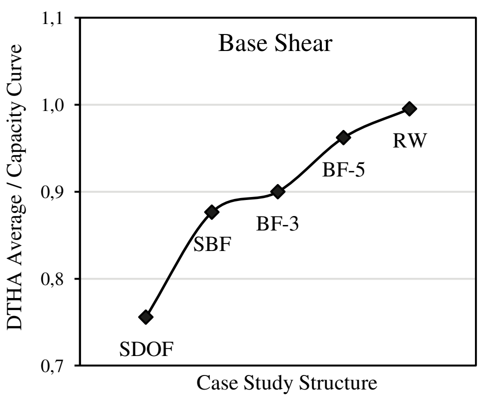

Figure 11: DTHA average to capacity curve peak ratio for case study structures

5 CONCLUSIONS

This paper investigates the feasibility of the capacity spectrum method for low damage structural systems with friction energy dissipators. Compared to dynamic time history analysis, the capacity spectrum method uses non-linear static pushover analysis that is simpler and more efficient. Therefore, it could be an ideal and reliable approach to new builds. Various structural types have been tested using both NPA and DTHA to evaluate the feasibility of the capacity spectrum method for low damage structural systems with friction energy dissipators. All case studies utilise flag-shaped friction energy dissipating devices, with non-linearities limited and localised to the friction devices. This study shows that the capacity spectrum method can be used efficiently and reliably for the analysis of regular structures utilising LDD friction connections with hysteretic damping ranges between %5 and %15, such as those described in this paper. It is concluded that to adopt a conservative yet safe design, an overstrength margin (e.g. extra lateral displacement available in the system) of %10 and %20 are suggested for braced frames and wall structures, respectively. In reality, if the demand for a record exceeds the capacity curve, it is likely that the connections are damaged or yielded; however, there should not be any damage to the structural members since they must be capacity designed with an overstrength factor that is appropriate for the particular friction connection.

This study is limited to LDD structures relying on friction connections to provide damping and ductility. The present study examines common structures in 2D. In future, Further investigation of similar structures in 3D will be conducted, in addition to investigating hybrid structures that include both bracing and wall components.

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