
Bridging Current and Future Seismic Hazards Through Friction Damping Systems

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ABSTRACT

The recently released National Seismic Hazard Model represents a major step forward in understanding expected earthquake intensities across New Zealand. Its findings indicate an average increase in seismic shaking of around 50 percent nationwide, with some regions experiencing up to a twofold increase. The recent TS1170.5 aims to integrate these new hazard levels into the structural design framework, significantly increasing expected ground excitations and design base shear demands for most sites. This escalation challenges traditional seismic design approaches. Structures designed to remain elastic or nominally ductile are no longer feasible in many regions, and even conventional fully ductile designs, particularly for Importance Level 3 and 4 buildings, often lead to impractically large member forces, substantial foundation demands, and a proliferation of braces or walls to satisfy code requirements. This paper explores how innovative braced-frame systems, specifically friction-damped braces, can enable practical and economical designs that meet both current NZS1170.5 requirements and the higher demands implied by TS1170.5. When appropriately configured, friction damper braces enhance ductility and energy dissipation without excessively increasing member sizes or foundation capacity. A series of nonlinear static and dynamic analyses is presented for an IL3 steel frame building representative of a high-importance structure. The results demonstrate that a structure designed for current NZS1170.5 hazard levels at the ULS can also satisfy TS1170.5 ULS level demand without additional braces or capacity increases, highlighting the potential of friction damping systems to deliver resilient, efficient, and code-compliant seismic solutions.

1 INTRODUCTION

The seismic hazard defined in NZS 1170.5 for the design of buildings are based on a seismic hazard model that dates back to 2002. Subsequent research, particularly following the Canterbury and Kaikōura earthquakes, has demonstrated that this model underestimates the actual seismic hazard across New Zealand (Cubrinovski et al. 2021; Stirling et al. 2021). The recently developed and revised National Seismic Hazard Model (NSHM) (Gerstenberger et al. 2022) represents a substantial advancement in understanding expected earthquake intensities nationwide. A major contributor to the increased assessed hazard arises from updated treatment of the Hikurangi subduction zone, which is now understood to have the potential to affect a much wider area than large crustal fault systems (Clark et al. 2019; Williams et al. 2013). The newly released TS 1170.5 incorporates these updated hazard levels into the structural design framework, resulting in significantly higher expected ground motions and increased design base shear demands for most sites (Elwood et al. 2025). The document indicates an average increase in seismic shaking of approximately 50 percent nationwide, with some regions experiencing increases of up to twofold. This escalation challenges traditional seismic design approaches. Structures designed to remain elastic or nominally ductile are no longer feasible in many regions, and even conventional fully ductile designs, particularly for Importance Level (IL) 3 and 4 buildings, often lead to impractically large member forces, substantial foundation demands, and a proliferation of braces or walls to satisfy code requirements.

1.1 Objective

This paper explores how innovative braced-frame systems, specifically Friction Damped (FD) braces, can enable practical and economical designs that meet both current NZS1170.5 requirements and the higher demands implied by TS1170.5. The objective of the presented design approach is to achieve more resilient outcomes for new buildings in future seismic events. This approach helps avoid situations where newly completed buildings are rated significantly lower under the updated TS hazard levels, despite being designed under the earlier NZS hazard framework. “Whilst uncertainty in the ground shaking hazard is out of an engineer’s control, uncertainty in building performance can be controlled (although not eliminated) through design” (NZSEE 2022). An ideal design approach is one in which the structure is not designed solely to NZS, which would result in under-design for future hazard levels, nor solely to TS, which would lead to significantly higher demands and increased cost, but instead is designed to satisfy the requirements of both standards. When appropriately configured, friction damper braces enhance ductility and energy dissipation without excessively increasing member sizes or foundation capacity. An additional benefit is that structural designs can not only satisfy ULS-level hazard requirements but can also meet CALS, or potentially MCE, hazard levels with greater confidence and predictability. This improves confidence in seismic performance and resilience and ensures reliable and predictable structural behavior up to the CALS or MCE limit states.

1.2 Friction Damper

Friction dampers (FD) provide an efficient passive seismic energy dissipation mechanism that utilizes the principle of solid surface contact-friction to provide ductility and control structural response. Initially proposed for structural systems by (Pall et al. 1980), FD dissipate energy through friction between clamped plates, with the inner steel plate incorporating a slot to facilitate free movement of the outer plates without damage while providing Isotropic loop hysteresis behavior. The slot in the middle plate can be designed to accommodate small or large displacements, generally ranging from 25 mm to 150 mm, depending on the structural drift requirements. Unlike conventional braces, which dissipate energy through yielding and plastic deformation, FD provide ductility without any yielding or damage. Recent developments of the symmetric-type FD, referred to as a ‘slip friction damper’, have demonstrated stable and symmetric Elasto-Plastic (EPP) hysteretic response under both static and dynamic loading, with low variability in force fluctuations and corresponding overstrength factors in the range of 1.1 to 1.2. (Loo et al. 2014).

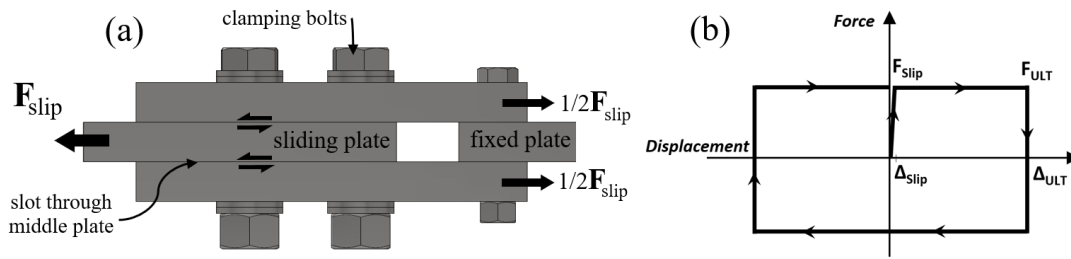


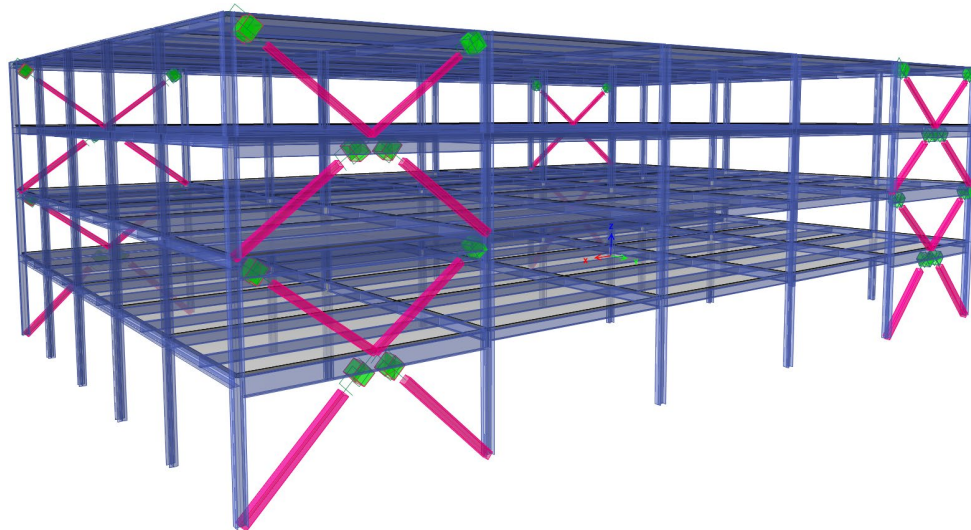
Figure 1: Typical slip friction joint: (a) symmetric joint assembly, (b) idealized joint isotropic hysteresis

2 METHODOLOGY

2.1 Building description, seismic parameters and numerical modeling

A four-storey steel structure adopted from the NIST technical document 1863-4 (Speicher and Harris 2020) is used to demonstrate the proposed design approach. This archetype structure generally represents the typical configuration and layout of high-importance buildings such as hospitals and schools. To illustrate the significant increase in hazard levels between NZS1170.5 and TS1170.5 and to demonstrate the proposed approach, the structure is assumed to be located in Wellington CBD, with IL3 for 50 years design, situated on soil class D (class IV in TS1170.5). Therefore, limit states considered for this study corresponds to SLS1 at 1/25, SLS2 at 1/250, ULS at 1/1000, and CALS at 1/2500 return period.

Seismic design guidelines for high-importance structures, such as those issued by the Ministry of Education (MoE) for schools and the Ministry of Health (MoH) for hospitals and healthcare facilities, specify performance targets that extend beyond the requirements of NZS1170.5 and TS1170.5. For an IL3 structure such as the one considered in this study, the SLS2 limit state corresponding to a 1/250 return period must be satisfied, with drift limits of 0.4% and ductility limits of $\mu \leq 2.0$ for hospitals and $\mu \leq 1.25$ for schools. These limits are established to align with the national resilience objectives. The structure is designed and analysed for the longitudinal direction for this study, with four braced bays located at the perimeter. The braced bays are arranged symmetrically on both sides and along the height of the structure and positioned near the corners to avoid inherent torsional behavior. Numerical modeling is carried out in ETABS. The capacity and slip force of the FD braces are adjusted through an iterative process to achieve synchronized interstorey drift and uniform sway response. In order to account for realistic nominally-pin behaviour of the column bases and collector beams, a rotational stiffness $0.1EL/L$ is applied to column bases and collector beams (Ainsworth et al. 2015). An overstrength factor ϕ_{os} of 1.15 is applied for the capacity design of collector beams and columns. FD joints are modeled via 'Plastic-Wen' link elements in ETABS.



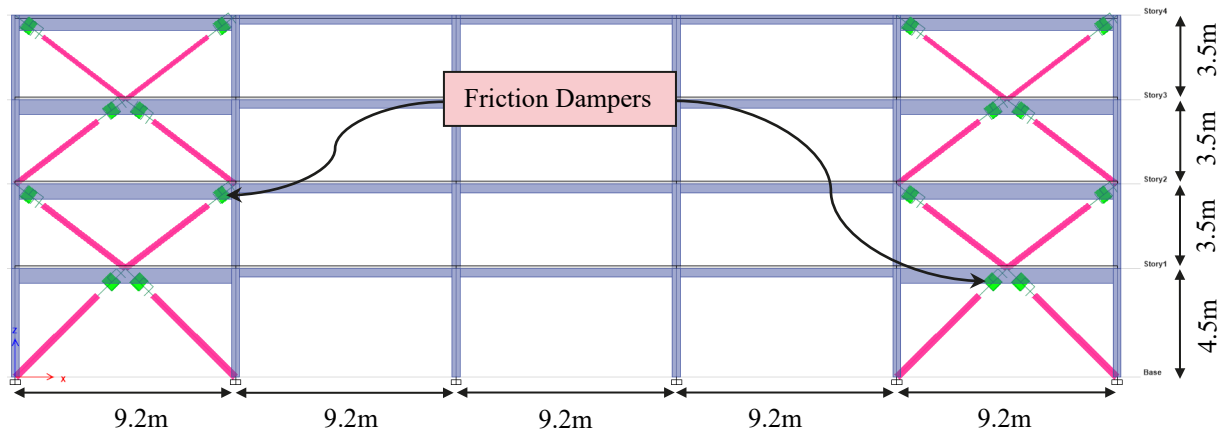


Figure 2: Four-storey steel structure with FD braces.

3 SEISMIC DESIGN

3.1 Capacity Spectrum Method

The design approach follows the Capacity Spectrum Method (CSM), also known as Acceleration-Displacement Spectrum (ADRS) method, using nonlinear static analyses, and is further verified through Dynamic Time History (DTH) analyses. To conduct the CSM, two pushover methods are carried out in accordance with Section C2.8.2.3 of the MBIE C2: Seismic Assessment and Analysis Techniques Guidelines (MBIE and NZSEE 2018):

- A uniform lateral load distribution consisting of lateral forces at each level proportional to the total mass at each level, referred to as Pushover Acceleration (Acc).
- A load pattern distribution proportional to the values of C_{vx} as defined in Equation C2.18, referred to as Pushover Load Pattern (LP).

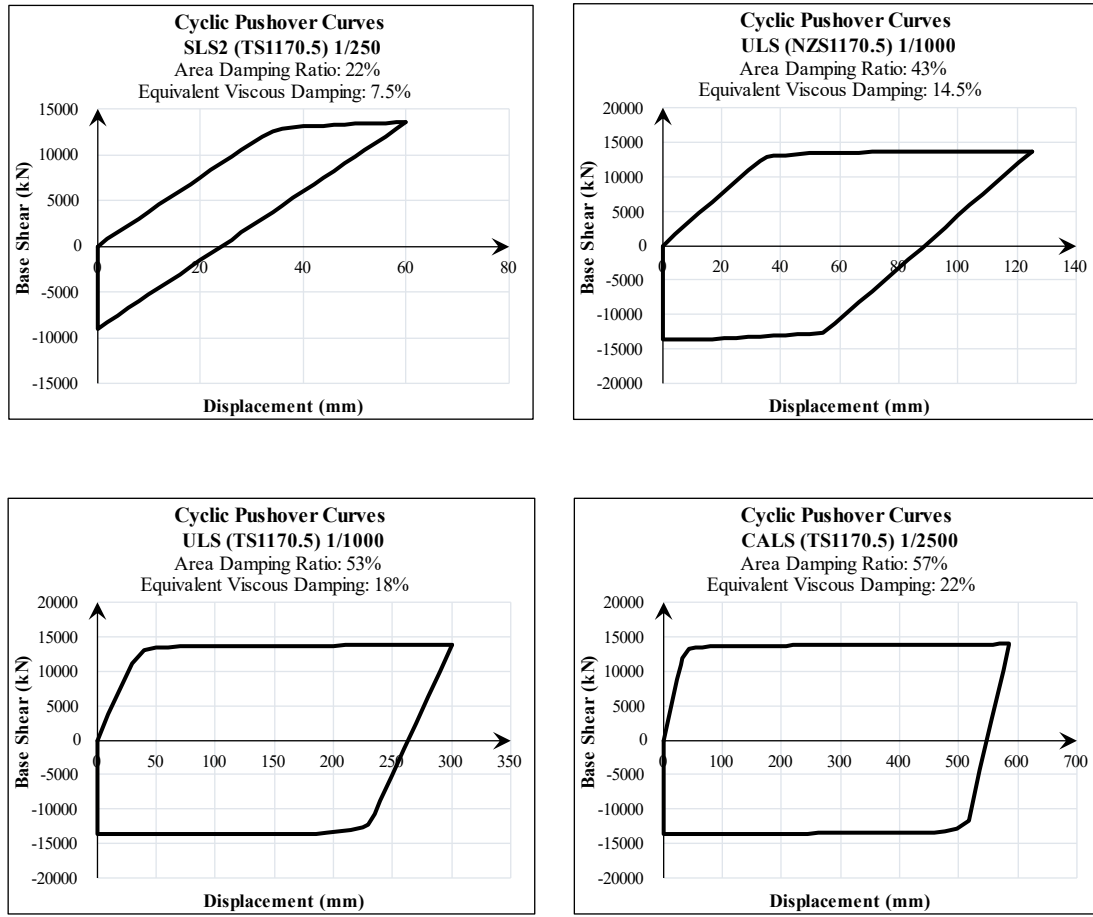
$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^n w_i h_i^k} \quad (1)$$

Where:

C_{vx} is the load distribution factor for each floor, w_i portion of total building weight W on floor level i , w_x is portion of total building weight W on floor level x , h_i is height (in m) from base to floor level i , h_x is height (in m) from base to floor level x , and k is 2.0 for $T_1 \geq 2.5$ seconds and 1.0 for $T_1 \leq 0.5$ seconds (linear interpolation is to be used for intermediate values of T_1).

Hysteresis damping ratios (area-based) are determined using Jacobsen's simplified equation (Chan et al. 2021) by conduction cyclic pushover analyses, and are then converted to equivalent viscous damping for use in the ADRS method following Priestley's approach (Priestley et al. 2007). As shown in the ADRS graph (Figure 4), the seismic demand in terms of base shear is governed by the acceleration pushover, while the drift demands are governed by the load-pattern pushover. This method allows the envelope of seismic demand in terms of force and drift to be adequately determined. Table 1 presents the seismic design output for each limit state based on performance points (defined as the intersections between the hazard spectra and the pushover curves) derived from ADRS graph. The hazard spectra plotted on ADRS graph are damped spectra based on obtained equivalent viscous damping values presented on Figure 3. For this design, a conservative structural performance factor S_p of 0.85 for ULS and CALS; and 0.7 for SLS2 and SLS1 is adopted for the application

of ADRS (Marriott 2018). The structure is subjected to displacement levels beyond the ULS to demonstrate its ability to meet CALS demands to ensure a robust seismic design, facilitated by the overstrength and reserve displacement capacity of the FD joints, this is illustrated on the ADRS graph.



$$\xi_{system_{SLS2_TS}} = \xi_{el} + \xi_{hys} = 5\% + 7.5\% \quad \xi_{system_{ULS_NZS}} = \xi_{el} + \xi_{hys} = 3\% + 14.5\%$$

$$\xi_{system_{ULS_TS}} = \xi_{el} + \xi_{hys} = 3\% + 18\% \quad \xi_{system_{CALS_TS}} = \xi_{el} + \xi_{hys} = 3\% + 22\%$$

Figure 3: Cyclic non-linear pushover analyses and damping determination.

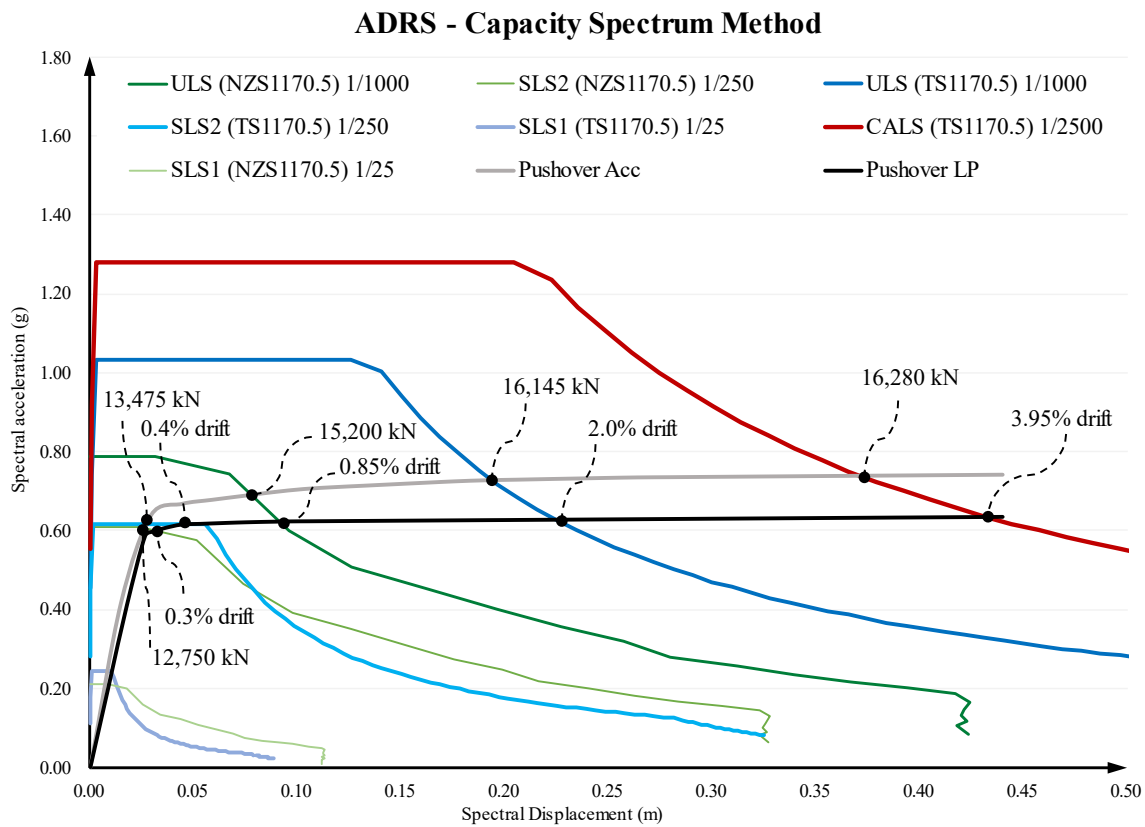


Figure 4: ADRS along with pushover curves and the performance points for considered limit states.

As illustrated in the ADRS graph, a single design can satisfy SLS2, ULS, and CALS demands under both NZS1170.5 and TS1170.5. This demonstrates an efficient solution in which the structure can comply with both standards simultaneously without increasing the number of braces, upgrading to higher-capacity braces, or introducing supplementary lateral load-resisting systems to meet the higher TS 1170.5 demands. Accordingly, the braced-bay frame members and foundation can be designed once, without subsequent strengthening or overdesign to accommodate TS1170.5 requirements. Avoiding post-construction foundation strengthening, which is technically challenging and highly costly, results in a substantially more economical and sustainable design. The damage-free performance of the friction damper braces and the ability to satisfy CALS demands without yielding or disruption further ensure a resilient and robust structural solution.

Table 1: Seismic demand output derived from ADRS.

Limit State	Base Shear (kN)	Structural Drift (%)	Equivalent Ductility (μ)
SLS2 (NZS1170.5) 1/250	12,750	0.28	1.2
SLS2 (TS1170.5) 1/250	13,475	0.40	1.85
ULS (NZS1170.5) 1/1000	15,200	0.85	2.1
ULS (TS1170.5) 1/1000	16,145	2.0	3.0
CALS (TS1170.5) 1/2500	16,280	3.95	4.25

3.2 Complimentary verification method

The structure can be designed using the ADRS method since the modal participation in both orthogonal directions is approximately 90% and the structure is regular in shape. Although not necessary as part of the design procedure, an additional layer of verification is carried out to provide an independent check on the seismic demands. For this purpose, 3 suites, each with 15 ground motion records from PEER NGA-West2 database have been selected and scaled to TS 1170.5 requirements for three limit states of ULS (NZS1170.5) 1/1000, ULS (TS1170.5) 1/1000, and CALS (TS1170.5) 1/2500. The seismic response obtained from DTH closely matches the results derived from ADRS, demonstrating that the structure can be adequately designed using the CSM methodology.

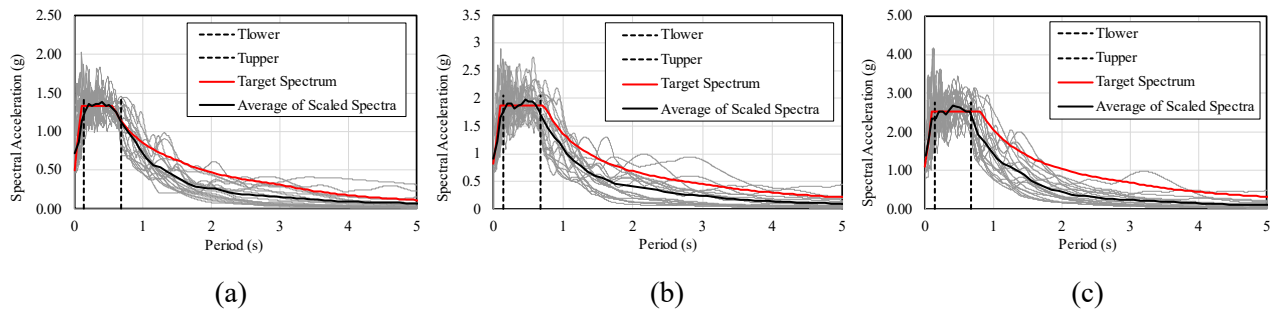


Figure 5: Scaled ground motion records for three hazard levels : (a) NZS 1170.5 ULS [1/1000]; (b) TS 1170.5 ULS [1/1000]; (c) TS 1170.5 CALS [1/2500].

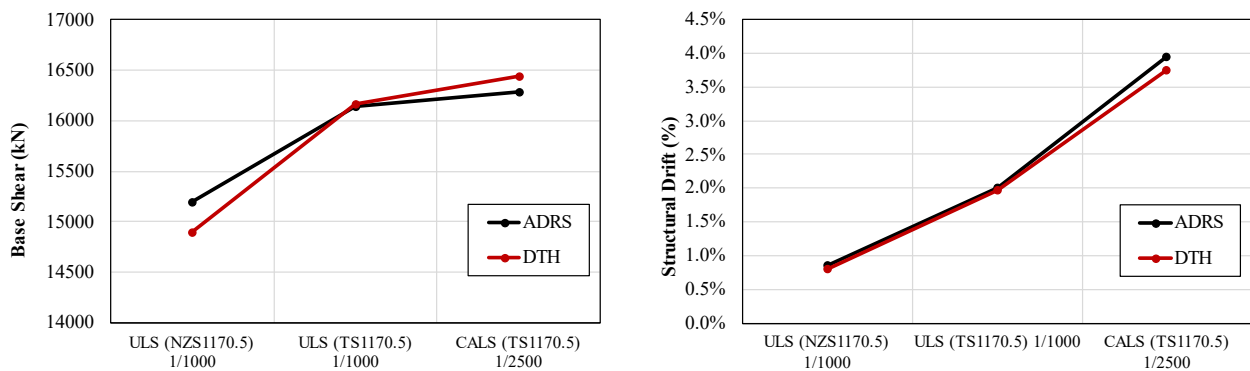


Figure 6: Seismic output of ADRS in comparison to DTH.

4 COMPARISON WITH EQUIVALENT EBF SYSTEM

To provide a comparison, an equivalent new-build design using Eccentrically Braced Frames (EBF) is developed for the same study structure to accommodate TS 1170.5 demands, and the EBF system is designed in accordance with the HERA P4001 design guideline (Clifton and Cowie 2013), targeting a similar ductility level of 3.0 at ULS (TS1/1000) as achieved with the FD bracing system. The EBF system is designed using active shear link configuration and design entails for active links of size 400WC361 for the first two storeys and 350WC230 for the top two stories. The collector beams and columns are designed with an overstrength factor of $\phi_{os_EBF} = 1.4$ in accordance with Table 12.2.8 (2) of NZS3404 (NZS 2007).

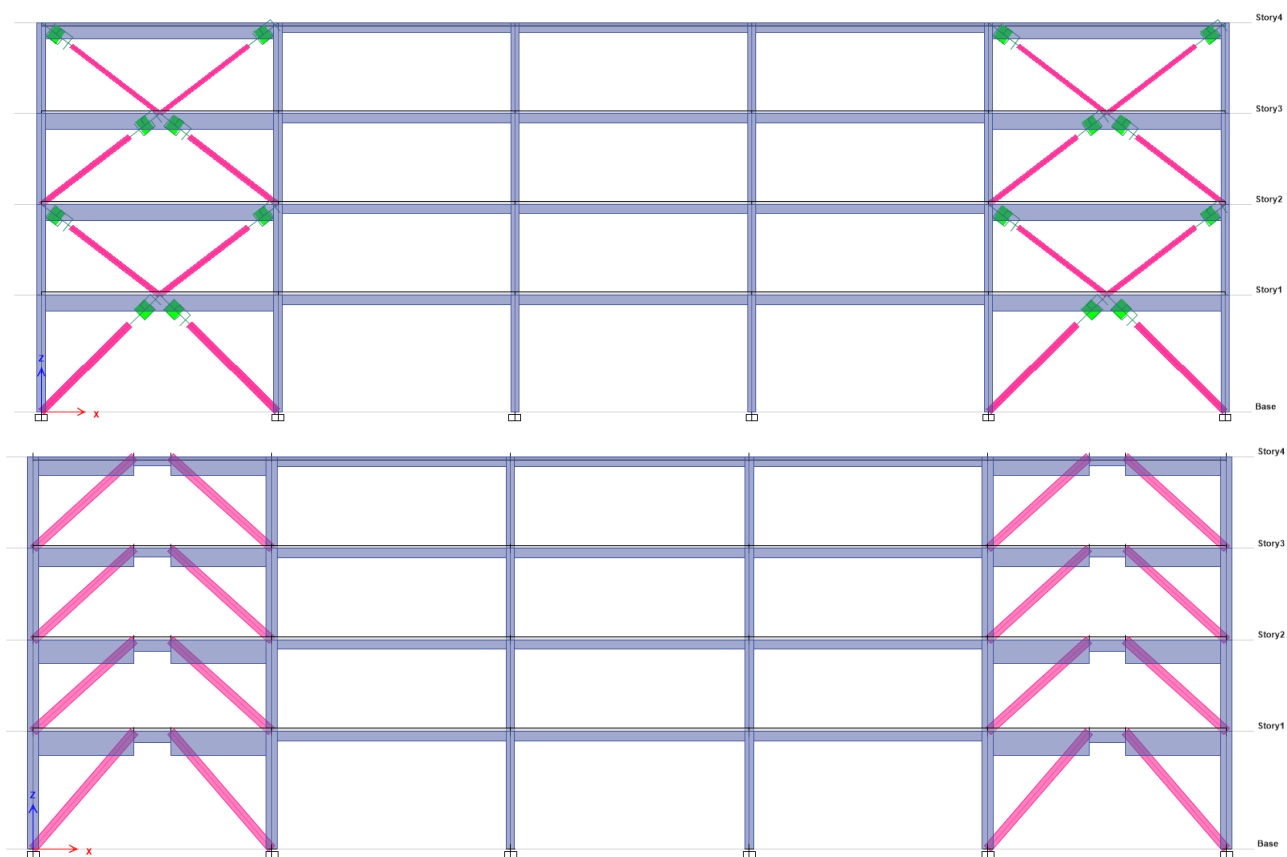


Figure 7: FD brace layout (top) and EBF brace layout (bottom).

For the same seismic force corresponding to a ductility of 3.0 at ULS (TS 1/1000), the EBF design results in larger and heavier member sections for the braces, collector beams, and columns. This increase is further exacerbated by the higher overstrength factor required for the EBF system compared to the FD bracing system. By comparing the total weight of the bracing group members, including brace bay members, FD joints or EBF links, collector beams, and collector columns, the overall tonnage for each system is calculated. The results show that the FD bracing system achieves approximately 20% steel tonnage reduction due to the smaller required section sizes. Table 2 presents the comparison of brace demands for the FD and EBF systems. The listed design demands represent the forces for which the brace members, and subsequently the collector beams and columns in accordance with capacity design principles, must be designed.

Table 2: Brace design capacity comparison between FD and EBF system (with respective overstrength factors).

Storey	FD Brace with $\phi_{os} = 1.15$	EBF Brace with $\phi_{os} = 1.4$
4	805	1631
3	1553	2423
2	2214	3556
1	2760	4298

Considering the reaction at a collector column base, the FD bracing system results in a vertical reaction of 4445 kN, whereas the EBF system produces a vertical reaction of 5450 kN that must be resisted by the foundation. When accounting for the respective system overstrength and applying capacity design principles to the foundation, this corresponds to approximately 35% reduction in vertical demand at each column base location for the FD bracing system. Although the nominal horizontal reactions at the column bases are similar for both systems, the higher overstrength factor associated with the EBF system increases the horizontal design demand on the foundation by approximately 20%. These reductions in foundation demand achieved by the FD bracing system can enable smaller foundation caps and fewer piles, leading to significant overall reductions in foundation and piling requirements and substantial cost savings across the structure.

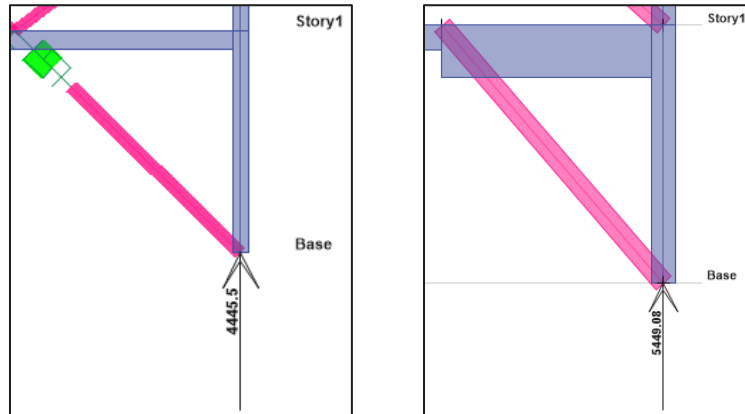


Figure 8: Collector column base reaction: (left) FD bracing, (right) EBF bracing

5 CONCLUSION

A proposed design approach incorporating a friction damper bracing system is presented, in which a single design for the friction damper braces, collector beams, columns, and foundation enables the structure to satisfy SLS2, ULS, and CALS demands under both NZS1170.5 and TS1170.5. A series of nonlinear static and dynamic analyses is conducted for an IL3 frame building located in Wellington CBD, representative of a high-importance level structure, to support the proposed design. The results demonstrate that the objectives are achieved without the need for additional braces or increased member capacities, highlighting the potential of friction damping systems to provide resilient, efficient, sustainable, and code-compliant seismic solutions. Additionally, it is demonstrated that the proposed design, when compared with a conventional EBF ductile design at the same TS 1170.5 hazard level, can achieve substantial savings and improved cost-effectiveness through reductions in steel tonnage and significant decreases in foundation demands. The study presented in this paper is material independent and is therefore applicable to steel, concrete, and mass timber structures, as well as hybrid systems such as timber-steel frames.

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