Exploring resiliency between different steel lateral systems for new construction

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ABSTRACT

It is known that while base level, code compliant structural designs produce buildings with a low likelihood of collapse, there is little guarantee that buildings will remain functional following moderate to severe earthquakes. Because of this reality, there is a rising focus on issues of resiliency in the design of buildings and encouraging owners and developers to think beyond code minimum designs. While this may be the ideal, the reality is that many buildings are still designed to the base code level. This paper explores how these types of code-compliant buildings stack up in terms of resiliency metrics. For this exploratory study, three building systems common in New Zealand are explored: Eccentric Braced Frames (EBFs), Steel Moment Frames (SMFs) and Steel Moment Frames with Fluid Viscous Dampers (the Taylor Damped Moment Frame, TDMF™).

The software package, Seismic Performance Prediction Platform (SP3), is used to explore the resiliency metrics for archetypical structures using these three lateral systems. IL2 and IL4 type structures are examined at two seismic hazard levels for a representative site in New Zealand. SP3 operationalizes the FEMA P-58 and ATC-138 frameworks to determine probabilistic distributions of metrics such as expected loss, reoccupancy time, and functional recovery time. Sources of damage and downtime are also explored, and recommendations for structural and non-structural adaptations that can help improve the resiliency of these types of steel structures are briefly discussed.

1 INTRODUCTION

An important shift is happening around how the structural engineering community communicates with the general public about buildings and earthquakes. We, as a professional community, are doing better about pushing against language like "earthquake proof" and are developing sophisticated tools to better predict what the anticipated performance of a code-compliant structure will be – beyond just "not collapsing." Looking more holistically at building performance is allowing the profession to communicate the expected performance of existing structures in ways that help inform public policy, business practices, real estate management and, inevitably, public safety.

The key frameworks being refined and utilized are the FEMA P-58 (FEMA 2018) and ATC-138 (FEMA 2023) frameworks to evaluate structural performance, losses and downtimes from earthquakes. The purpose of this paper is to use these cutting edge methodologies to evaluate the expected performance of new code-compliant steel structures in New Zealand.

2 ARCHETYPICAL BUILDINGS

2.1 Site and Seismic Hazard

A representative site located in Wellington is chosen for this investigation. The design response spectra for the 1/500 and 1/2500 year events (Figure 1) are developed per NZS 1170.5 using the location specific values shown in Table 1.

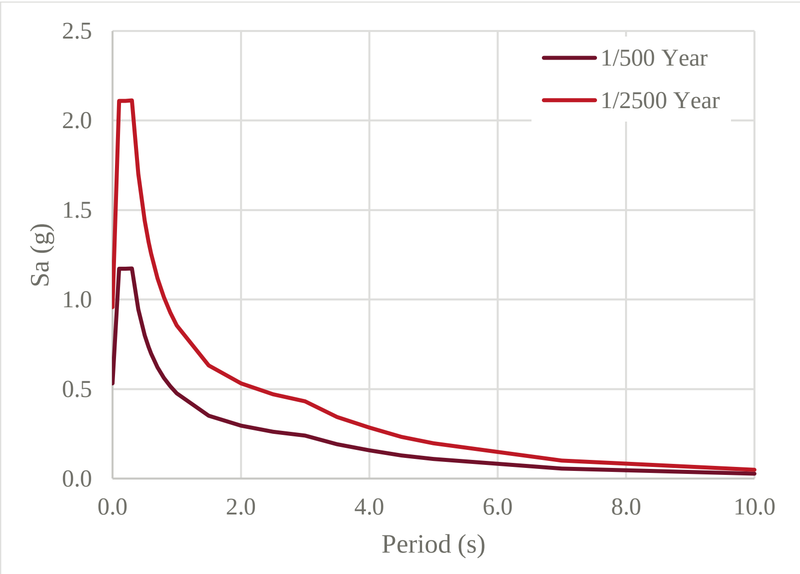


Figure 1: Design Response Spectra

Table 1: Key Seismic Design Parameters

Parameter	Value
Site Subsoil Class	C
Hazard Factor, Z	0.4
Return Period Factor, R	1.0 (IL2) 1.8 (IL4)
Near Fault Distance, D	2 km

2.2 Building Parameters

Three different steel lateral force resisting systems are evaluated: Eccentric Braced Frames (EBFs), Steel Moment Frames (SMFs) and a prescriptively designed damped moment frame system called the Taylor Damped Moment Frame™ (TDMF™) which has ICC approval as an alternative lateral force resisting system (ICC ESR #4769, https://icc-es.org/report-listing/esr-4769/). Six and 12-story representative office (IL2) and medical (IL4) structures are investigated, each with a rectangular plan area of 30 by 60 meters, a first story height of 4.87 meters and other story heights of 3.96 meters.

The SP3 RiskModel design automation engine, with modifications to adapt it to New Zealand design codes, is used to determine modal and strength properties of the archetype buildings. First, effective values of the response modification factor, (R), and deflection amplification factor (C_d) are determined based on system parameters in alignment with standard practice in New Zealand for each building type; these are shown in Table 2. Additionally, the minimum base shear requirements per NZS 1170.05 EQ. 5.2(2), P-Delta considerations per NZS 1170.05 Section 6.5.4, using Method A, and stability coefficient limit per NZS 1170.05 Section 6.5 are enforced in the archetype structure design step. Building properties are determined based on the modal response spectrum method in NZS 1170.05, allowing for up to a 20% reduction in base shear per section 5.2.2.2(a). The fundamental period and ultimate strength values that are determined by SP3, based on NZS design requirements, are given in Table 3.

Table 2: Lateral Force Resisting System Design Parameters

System	μ	S_p	k_μ	Effective $R = k_\mu/S_p$	k_{dm}	Effective $C_d = \mu(k_{dm})$
EBF	4	0.7	4	5.71	1.5	6
SMF	4	0.7	4	5.71	1.5	6
TDMF	4	0.7	-	5.71/0.75=7.61	1.5	=7.61*(4.5/8) = 4.2*

*For the TDMF system, the ICC design procedure utilizes R=8 and C_d = 4.5 in alignment with ASCE 7-16 design procedures. This same ratio was used to adjust the effective R value to determine an effective C_d.

Table 3: Summary of archetype buildings.

Archetype ID	Structural System*	Number of Stories	Importance Level	Occupancy	T_{design} [s]**	T_{final} [s]**	$(V/W)_{Max}$***
SMF-6-IL2	SMF	6	2	Office	1.60	1.19	0.155
SMF-6-IL4	SMF	6	4	Hospital	1.20	0.95	0.275
SMF-12-IL2	SMF	12	2	Office	2.40	1.92	0.135
SMF-12-IL4	SMF	12	4	Hospital	1.90	1.58	0.215
TDMF-6-IL2	TDMF	6	2	Office	2.15	1.46	0.116
TDMF-6-IL4	TDMF	6	4	Hospital	1.75	1.27	0.181
TDMF-12-IL2	TDMF	12	2	Office	2.92	2.24	0.099
TDMF-12-IL4	TDMF	12	4	Hospital	2.57	2.03	0.148
EBF-6-IL2	EBF	6	2	Office	1.36	1.13	0.123
EBF-6-IL4	EBF	6	4	Hospital	1.06	0.92	0.215
EBF-12-IL2	EBF	12	2	Office	1.98	1.71	0.109
EBF-12-IL4	EBF	12	4	Hospital	1.61	1.43	0.174

*Steel Moment Frame (SMF); Taylor Damped Steel Moment Frame (TDMF); Eccentric Braced Frame (EBF) ** T_{design} is the period used for the design model; T_{final} includes overstrength and stiffness from gravity framing and non-structural components. *** $(V/W)_{Max}$ is the peak lateral strength to weight ratio of the building

2.3 Overview of Analysis Software

The seismic performance prediction platform (SP3) RiskModel is used to analyse the seismic performance of the archetype buildings. The SP3 RiskModel employs the FEMA P-58 loss prediction method (FEMA 2018) for determining probabilistic earthquake losses and the recently developed ATC-138 method (FEMA 2023) for computing reoccupancy and recovery time. The FEMA P-58 analysis method is a rigorous building specific risk assessment method based on the Pacific Earthquake performance-based earthquake engineering framework developed by the Pacific Earthquake Engineering Research Center (PEER, Moehle and Deierlein 2004). A schematic of the FEMA P-58 seismic risk assessment methodology is shown in Figure 2; probability distributions of structural responses, component fragilities, and damage consequences for each hazard level are combined with the site hazard via Monte Carlo simulation. The ATC-138 functional recovery methodology utilizes realistic repair sequences, including impedance factors (e.g. for inspection, engineering work, financing, etc.), to simulate the time required for a building to become safe to enter (reoccupancy) and regain function (functional recovery). A schematic of the ATC-138 methodology is shown in Figure 3.

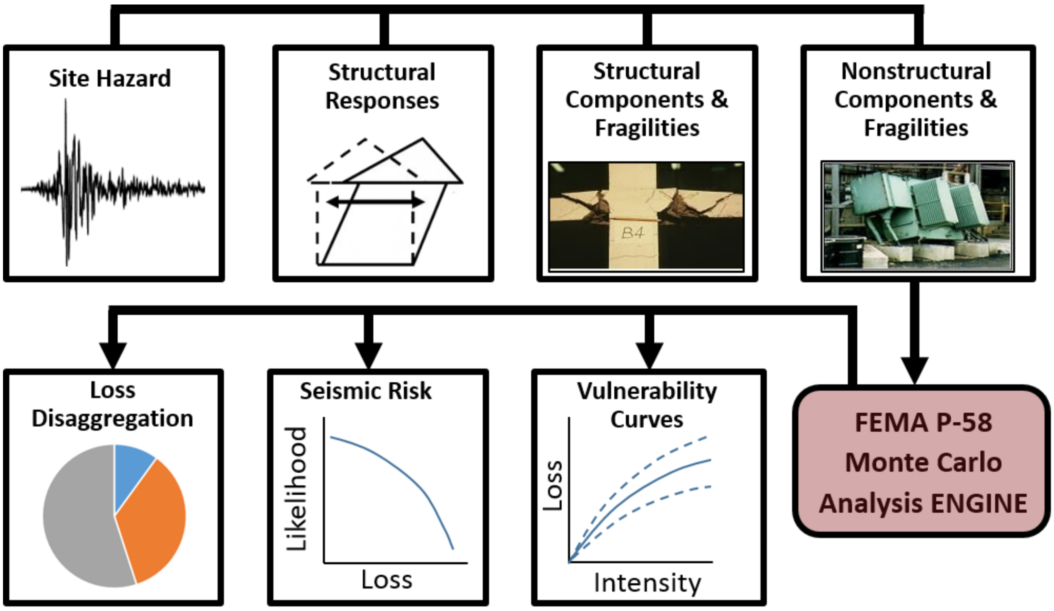
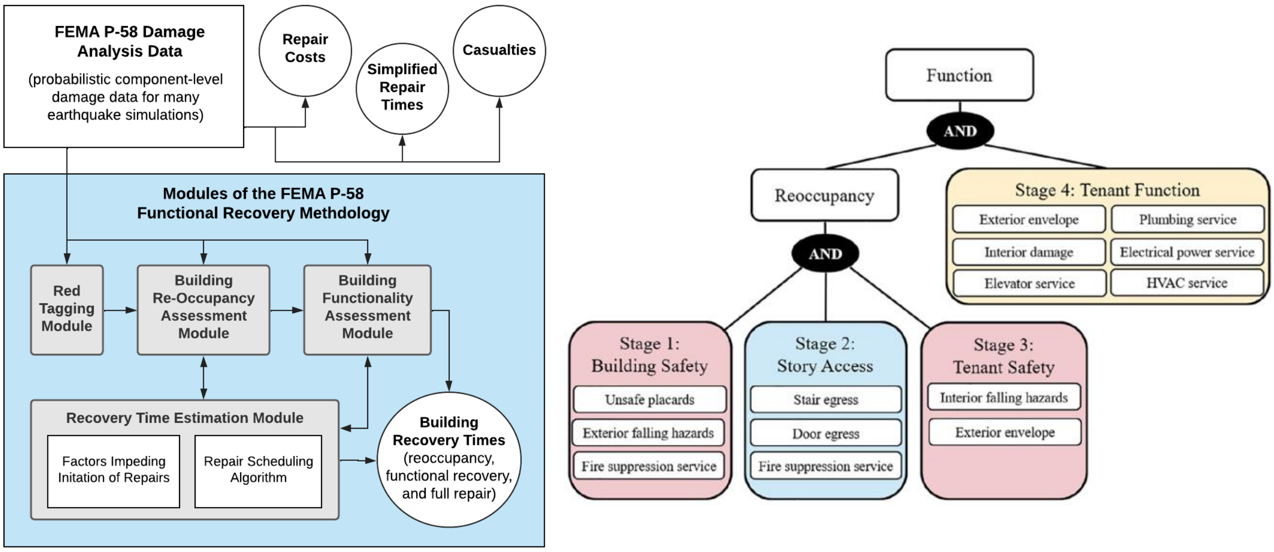


Figure 2: Schematic of FEMA P-58 seismic risk assessment methodology (Wade et al 2018)



a) b)

Figure 3: a) Overview of extending FEMA P-58 to include the functional recovery methodology (FEMA, 2023); b) Illustration of different requirements for reoccupancy and building function (Cook et al., 2022)

3 RESULTS

Each of the archetype buildings are analysed for site hazards equivalent to the 500-year and 2500-year design hazard spectra. Their median reoccupancy and functional recovery times, probability of an unsafe placard, and repair cost ratio (i.e. repair cost divided by the estimated building value) are summarized in Table 4. Recovery times for the 500- and 2500-year hazard levels are also depicted graphically in Figures 4 and 5, respectively.

Both of the steel moment frame systems (SMF and TDMF) outperform the EBF, primarily because of structural damage in the EBF links that causes high red tag probabilities and structural repair consequences. Note that the SP3 repair time estimates for EBF links are based on USA construction practices, which do not use an easily replaceable EBF link. Since it is common to use easily replaceable EBF links in New Zealand, the EBF recovery times in this study should be considered somewhat conservative. However, downtime estimates for structural repairs are typically dominated by impedance factors (e.g. permitting and lining up a contractor), not the actual time to perform the structural repair, so the effects of accounting for the differences in NZ vs USA construction practices for EBFs would be minor.

Both moment frame systems, damped and not damped, are expected to be occupiable but not functional immediately after the 500-year event. In terms of functional recovery and repair cost, the damped moment frames (TDMFs) significantly outperform the undamped moment frames (SMFs) for the 500-year hazard, mostly because the supplemental damping significantly reduces floor accelerations (~45% reduction of floor acceleration at the roof level), which thereby reduces damage to non-structural components. Notably, designing the TDMFs for IL4 results in a functional recovery time of only three days for the 500-year hazard, compared to ~120 days for the SMF and 179-263 days for the EBF systems. For the 2500-year hazard, structural damage for both moment frame systems is likely, leading to similar performance. Furthermore, the non-structural damage associated with the high drifts and accelerations at the 2500-year hazard is significant for all three systems.

Table 4: Building Seismic Performance Summary.

Archetype	500-year Hazard				2500-year Hazard			
	Median Reoccupancy [days]	Median Functional Recovery [days]	Probability of Unsafe Placard	Repair Cost Ratio	Median Reoccupancy [days]	Median Functional Recovery [days]	Probability of Unsafe Placard	Repair Cost Ratio
SMF-6-IL2	0	221	0.074	0.049	199	368	0.439	0.163
SMF-6-IL4	0	122	0.024	0.025	122	332	0.184	0.065
SMF-12-IL2	0	205	0.097	0.027	249	400	0.536	0.147
SMF-12-IL4	0	126	0.033	0.016	145	372	0.276	0.049
TDMF-6-IL2	0	141	0.039	0.026	183	347	0.394	0.128
TDMF-6-IL4	0	3	0.030	0.014	103	326	0.241	0.068
TDMF-12-IL2	0	148	0.061	0.015	228	390	0.501	0.134
TDMF-12-IL4	0	3	0.023	0.009	164	359	0.366	0.051
EBF-6-IL2	158	249	0.576	0.092	343	440	0.950	0.286
EBF-6-IL4	0	179	0.346	0.049	331	423	0.884	0.154
EBF-12-IL2	281	356	0.666	0.100	544	625	0.953	0.273
EBF-12-IL4	29	263	0.437	0.058	535	610	0.916	0.151

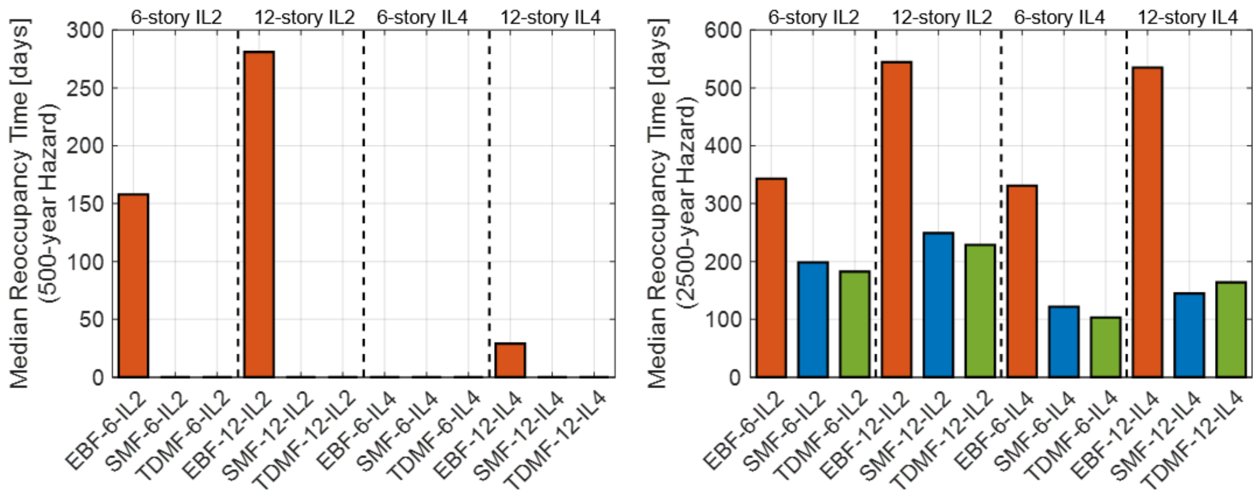


Figure 4: Median Reoccupancy and Functional Recovery times for the 500-year hazard event

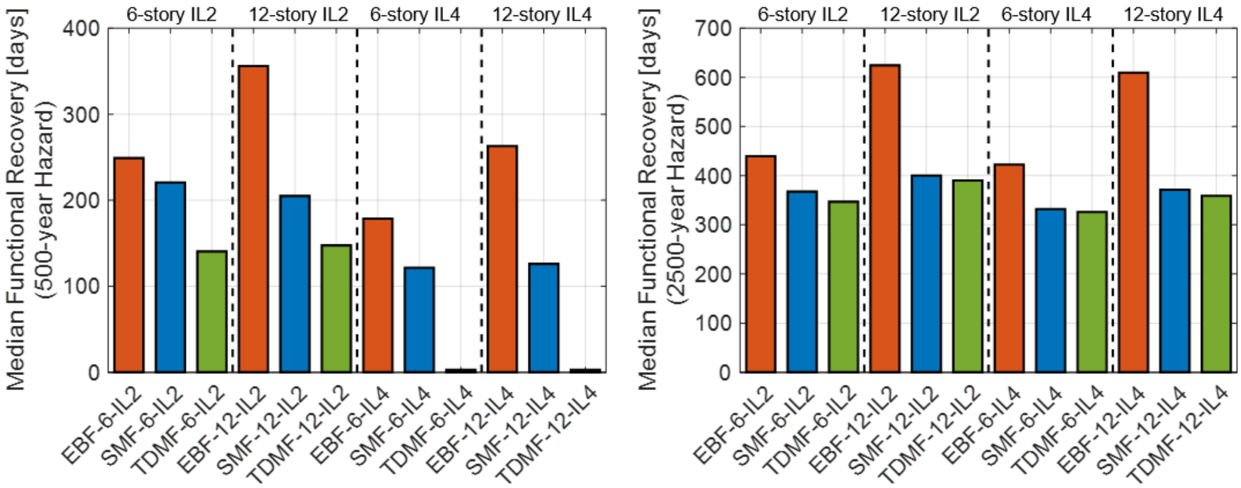


Figure 5: Median Reoccupancy and Functional Recovery times for the 2500-year hazard event

4 CONCLUSIONS

This evaluation of three different steel lateral force resisting systems shows that moment frame structures (damped and undamped) performed better than eccentric braced frame structures in metrics of loss and downtime, for IL2 and IL4 buildings and at the 1/500-year and 1/2500-year hazards. Damage associated with the yield link contributed largely to these differences, which is based on standard practice for the United States. The practice of using replaceable yield links in New Zealand may help improve the EBF performance over what was analysed here and would be an avenue for future exploration.

Comparing the damped and undamped moment framed buildings shows that both systems performed very well in terms of reoccupancy at the 1/500-year hazard. Adding dampers also improved the functional recovery times, nearly eliminating downtime to functional recovery for IL4 structures. The addition of dampers also halved the expected financial losses at the 1/500-year hazard. For the 1/2500-year hazard, however, the expected structural damage to both systems made their overall performance similar.

The increased design hazard used as the basis of design for the IL4 buildings was impactful at reducing losses and downtimes at the 1/500-year hazard. This was true across all three systems. At the 1/2500-year hazard, however, the IL4 buildings did have reduced losses, but comparable downtime estimations across all three systems. Said in a different way, if immediate occupancy is the goal of IL4 buildings in the NZS, this objective is not being met based on this evaluation with the shortest duration to reoccupancy being 103 days for the 6-story TDMF system.

Though outside of the scope of this study, there are many design and planning decisions that can be adjusted to help improve structural resilience and significantly reduce the anticipated downtimes. This evaluation assumed a base-level code compliance with respect to non-structural anchorage and design decisions, but additional design considerations, such as strengthened bracing for distributed building systems, enhanced equipment anchorage, and the use of seismically pre-qualified equipment that is essential for functionality of buildings systems or subsystems, could be specified. Further, more stringent quality assurance and quality control measures could be taken to ensure that non-structural building systems are properly installed as designed. Factors to mitigate delays in starting work after the earthquake such as having an inspector and engineer on retainer and accessible funding can also reduce downtime estimates for all systems. Finally, structural engineering that go beyond a code-minimum design, like performance-based engineering, can be used to make targeted changes to the structural system to improve behaviour and seismic performance.

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