

Development of a Value-Based Design (VBD) Framework for Seismic Design of Buildings

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Abstract

Development of a Value-Based Design (VBD) Framework for Seismic Design of Buildings

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Seismic design in Canada, as prescribed by the National Building Code of Canada (NBCC 2020), primarily focuses on life safety. However, the code gives limited attention to post-earthquake functionality and the long-term economic consequences. This study presents a Value-Based Design (VBD) framework for buildings subjected to seismic hazard that integrates the Endurance Time Method (ETM) with Life-Cycle Cost Analysis (LCCA) and Multi-Criteria Decision-Making (MCDM) to enhance the seismic resilience of reinforced-concrete (RC) moment-resisting frames for buildings. To demonstrate the proposed framework, a six-storey RC building assumed to be located in Vancouver was designed in accordance with NBCC 2020 provisions. Multiple design alternatives, varying column dimensions and reinforcement ratios, were analyzed using spectrally matched Endurance Time Excitation Functions (ETEFs). The ETM reduced computational time from ~6.4 hours per alternative (conventional Incremental Dynamic Analysis) to ~7 minutes (98.2% reduction) while maintaining close agreement with IDA median trends.

VBD alternatives achieved measurable improvements in structural resilience and economic efficiency across eight seismic hazard levels. The entropy-weighted optimal design (DA_33) reduced structural LCC by 13.3% and Total Cost by 2.93% relative to the code-minimum design (DA_01), despite a modest 2.29% initial cost premium. DA_35 and DA_28 showed similar reductions in life-cycle losses but required higher initial investments, illustrating a family of mid-range solutions that trade construction cost for additional resilience gains. Lifecycle cost analysis, integrating losses across all hazard levels, revealed that the code-minimum design

(DA_01) incurred about 643 kCAD in expected fatality costs and the highest structural repair expenditures. Mid-range designs with Initial Costs modestly above the code minimum achieved near-optimal performance, indicating diminishing returns beyond moderate reinforcement.

The framework links structural performance to economic metrics, supporting adoption of drift-based verification and lifecycle cost pathways as alternative NBCC compliance routes.

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CHAPTER 1: INTRODUCTION

1.1 Background

Earthquakes are among the most destructive natural hazards, causing direct physical losses often exceeding billions of dollars and indirect impacts such as business interruptions, population displacement, and productivity losses that amplify societal and economic consequences (Porter, 2015). Traditional prescriptive seismic design prioritizes life safety, ensuring structures resist collapse under rare, high-intensity ground motions. However, meeting life-safety objectives alone does not guarantee favorable economic outcomes or rapid functional recovery for building owners, occupants, or communities.

Over the past two decades, Performance-Based Earthquake Engineering (PBEE) has reshaped seismic design by defining explicit performance objectives across multiple hazard levels, enabling transparent trade-offs among safety, damage, and functionality (Krawinkler & Miranda, 2004). Building on PBEE, seismic resilience emphasizes not only structural integrity but also the rapid restoration of functionality. The MCEER framework defines resilience across technical, organizational, social, and economic dimensions (Bruneau et al., 2003), while analytical approaches quantify resilience as the normalized area under a post-event functionality curve (Cimellaro et al., 2010). The Endurance Time Method (ETM), an innovative dynamic analysis approach, supports this paradigm by efficiently evaluating structural performance across a spectrum of seismic intensities, offering a practical tool for resilience-focused design (Estekanchi et al., 2007).

This focus aligns with global disaster risk reduction priorities, such as the Sendai Framework for Disaster Risk Reduction, which emphasizes minimizing direct and indirect losses (UNISDR, 2015). Canada's commitment to these goals highlights the need to align seismic design with

international best practices, as exemplified by resilience-focused codes in Japan and New Zealand, where performance-based approaches explicitly address post-earthquake recovery (Dhakal, 2024; Sullivan, 2020; The Building Center of Japan, 2016). In engineering practice, FEMA P-58-2 provides a PBEE-consistent, loss-based methodology, linking seismic demands to repair costs, casualties, and recovery times, and producing decision-oriented metrics like expected annual loss (EAL) and downtime (FEMA, 2018). Recent initiatives, such as NIST SP-1254/FEMA P-2090 and the ATC-138 extension of FEMA P-58, further advance functional-recovery-based objectives by estimating re-occupancy and recovery timelines (Applied Technology Council, 2022; Sattar et al., 2021).

In Canada, particularly in Vancouver, seismic risk is acute due to the region's exposure to both crustal and subduction earthquakes from the Cascadia Subduction Zone, one of the country's highest-risk seismic zones (Atkinson & Macias, 2009). A major Cascadia Subduction Zone earthquake could cause economic losses of approximately \$75 billion in Canada, with the majority concentrated in British Columbia (AIR Worldwide, 2013). Reinforced concrete (RC) moment frames, widely used in Canada's urban infrastructure due to their cost-effectiveness and design flexibility, face unique challenges in achieving seismic resilience, particularly under the complex loading imposed by subduction earthquakes (Tremblay & Atkinson, 2001). The National Building Code of Canada (NBCC, 2020) enforces stringent seismic provisions for collapse prevention and drift control but does not explicitly address life-cycle costs, recovery times, or resilience metrics. Moreover, Canada's seismic design policy lacks incentives for resilience-based approaches, lagging behind global leaders in integrating post-earthquake recovery into building codes.

As Canada's urban centers densify, prolonged downtime and recovery inefficiencies pose escalating economic and societal risks. This context underscores the need for frameworks like

Value-Based Design (VBD), which integrates stakeholder priorities such as initial costs, life-cycle performance, and post-earthquake recovery into a decision-making framework that balances safety and economic objectives (Mitropoulou et al., 2011). By leveraging tools like ETM, VBD can guide the design of RC moment frames to optimize resilience, aligning with both national priorities and global standards for disaster risk reduction.

1.2 Review of NBCC 2020 Seismic Provisions

The National Building Code of Canada (NBCC 2020) prescribes seismic design actions through a Uniform Hazard Spectrum (UHS) derived from Probabilistic Seismic Hazard Analysis (PSHA), with site-specific effects incorporated via site coefficients (e.g., F_a , F_v) and Site Class designations. The primary objective is life safety under rare earthquakes, typically defined as events with a 2% probability of exceedance in 50 years (2475-year return period), without explicit consideration of economic losses, downtime, or post-earthquake functionality.

The design approach remains predominantly force-based, employing ductility-related (R_d) and overstrength-related (R_o) factors to account for inelastic response and structural redundancy (Mitchell et al., 2010). Capacity-design principles, such as the strong-column–weak-beam requirement for reinforced concrete (RC) moment frames, promote ductile failure mechanisms to prevent collapse. Serviceability and damage-control checks, including interstorey drift limits (e.g., 2.5% for most buildings), mitigate excessive deformation but do not quantify life-cycle consequences or recovery timelines.

While this framework effectively reduces collapse risk, code-compliant designs may still incur significant economic losses and prolonged downtime, particularly for critical urban infrastructure like RC moment frames (Tremblay & Atkinson, 2001). Recent research advocates supplementing NBCC’s prescriptive provisions with resilience-based frameworks, such as Value-Based Design

(VBD), to integrate metrics like Expected Annual Loss (EAL), downtime, and functional recovery time (Tesfamariam & Goda, 2015). Such approaches, supported by advanced tools like the Endurance Time Method (ETM), enable more informed seismic design decisions that balance safety, cost, and post-earthquake functionality.

1.3 Problem Statement

Seismic design in Canada, as governed by the National Building Code of Canada (NBCC 2020), employs a prescriptive, force-based approach to ensure life safety by preventing collapse and controlling interstorey drift under rare earthquakes (2% probability of exceedance in 50 years). While effective in reducing catastrophic failures, this framework does not address economic losses, downtime, or post-earthquake functional recovery, resulting in designs particularly for reinforced concrete (RC) moment frames that may meet code requirements but incur significant repair costs and prolonged disruptions (Tesfamariam & Goda, 2015). This gap is critical for stakeholders, including building owners, policymakers, and communities, who prioritize not only safety but also rapid recovery and cost-effective infrastructure in high-risk regions like Vancouver, exposed to Cascadia Subduction Zone earthquakes (Atkinson & Macias, 2009).

Advanced analytical tools, such as the Endurance Time Method (ETM), enable efficient nonlinear dynamic evaluation across multiple seismic hazard levels, offering a means to assess resilience and life-cycle performance. Despite its adoption in international research, ETM remains underutilized in Canadian practice. Similarly, current design studies often focus narrowly on code compliance or reinforcement details for RC moment frames, neglecting holistic frameworks like Value-Based Design (VBD) that integrate initial costs, life-cycle performance, and resilience

metrics. This gap between minimum code compliance and comprehensive seismic performance creates a critical gap in achieving economically efficient and seismically robust designs.

Addressing this gap requires a novel framework that leverages ETM for time-history analysis, incorporates life-cycle cost assessment, and applies VBD to align seismic design of RC moment frames with stakeholder priorities balancing safety, cost, and resilience. Such an approach would empower building owners to optimize long-term value, enable policymakers to enhance urban resilience, and support communities in recovering swiftly from seismic events.

1.4 Research Objectives

This study aims to advance the seismic design of reinforced concrete (RC) moment frames in Canada by integrating Value-Based Design (VBD) and the Endurance Time Method (ETM) to enhance resilience, economic efficiency, and stakeholder outcomes. The specific objectives of the thesis are:

- a) Develop a reference framework for Canadian seismic design using a value-based design (VBD) approach, integrating stakeholder priorities such as initial costs, life-cycle performance, and post-earthquake recovery to optimize RC moment frame designs.
- b) Implement and validate the Endurance Time Method (ETM) for seismic analysis of RC moment-resisting frames, comparing its effectiveness with conventional approaches.
- c) Evaluate design alternatives for RC moment frames within a VBD framework, balancing life-cycle costs, seismic performance, and resilience metrics, such as Expected Annual Loss (EAL) and functional recovery.

- d) Quantify the performance gap between NBCC 2020 minimum-compliance design and VBD-optimized alternatives, establishing the lifecycle cost and resilience benefits of optimization within a value-based
- e) Demonstrate the potential implications of VBD and ETM in Canadian seismic practice, providing evidence to inform future updates to design guidelines and policy frameworks.

1.5 Thesis Organization

The thesis is divided into six chapters.

Chapter 1 – Introduction provides the background, problem statement, research objectives, and significance of the study. It establishes the motivation for moving beyond prescriptive seismic design in Canada by integrating cost, performance, and resilience metrics.

Chapter 2 – Literature Review synthesizes prior work in four key areas: (i) seismic hazard in Canada and performance-based design frameworks, (ii) the development and application of the Endurance Time Method (ETM), (iii) life-cycle cost analysis (LCCA) approaches including FEMA P-58 methodologies, and (iv) resilience metrics and multi-criteria decision-making (MCDM) in structural design. The review highlights existing gaps and positions this study within ongoing research on resilience-based seismic design.

Chapter 3 – Methodology presents the research framework in detail. It begins with the case study definition and baseline design following NBCC 2020, then outlines the generation of design alternatives through modifications of column dimensions and reinforcement. It describes the ETM modeling approach in OpenSees, damage state categorization, and loss modeling for LCCA. The chapter also introduces the Resiliency Index and explains the MCDM approach based on Shannon entropy weighting and weighted-sum ranking.

Chapter 4 – Endurance Time Excitation Functions (ETEFs) focuses on the generation and validation of the excitations used in the study. It describes the response-spectrum matching procedure adopted (via SeismoMatch), contrasts the combined ETEF (ETEF_C) with segmented excitations (ETEF_A), and evaluates the limitations of time-series length. Validation is performed against Incremental Dynamic Analysis (IDA) using real ground motions to assess accuracy and identify constraints of the simplified approach.

Chapter 5 – Results and Discussion present the findings across three dimensions. First, the life-cycle cost analysis (LCCA) is detailed, comparing Initial Cost, LCCA, and Total Cost across all design alternatives. Second, multi-criteria decision-making (MCDM) results are reported, including normalized scores, entropy-derived weights, and sensitivity analysis, leading to the identification of top-performing designs. Third, the Resiliency Index results are analyzed to establish the cost–resilience relationship. The discussion synthesizes these results to highlight trade-offs and trends in economic and seismic performance.

Chapter 6 – Conclusion summarizes the key findings, emphasizes the cost–resilience link, and reflects on the limitations of the study including the exclusion of non-structural losses and the restricted scope of design alternatives. It also outlines directions for future research, including more advanced ETEF generation strategies, broader sets of design parameters, and integration with socio-economic factors such as insurance and housing recovery.

CHAPTER 2: LITERATURE REVIEW

2.1 General

Seismic design has evolved from prioritizing life safety to embracing resilience-oriented approaches that ensure rapid functional recovery and long-term economic performance. As discussed in the introduction (Section 1.2), the National Building Code of Canada (NBCC 2020) sets minimum seismic requirements but overlooks post-earthquake functionality and economic resilience, necessitating advanced frameworks like Value-Based Design (VBD). Reinforced concrete (RC) moment frames, widely used in seismically active urban centers like Vancouver, face unique challenges due to complex subduction and crustal earthquake demands and site class variability. Stakeholders, building owners, policymakers, and communities, demand designs that balance safety, cost, and recovery efficiency in such high-risk regions.

VBD addresses these needs by integrating structural performance with life-cycle cost analysis (LCCA), optimizing trade-offs between initial investment, seismic resilience, and stakeholder priorities. Despite advances in nonlinear dynamic analysis, performance-based seismic design (PBSD), and resilience frameworks, VBD's application in Canada remains limited, particularly for RC moment frames. This review synthesizes developments in seismic analysis, focusing on the Endurance Time Method (ETM), VBD, and optimization tools, to inform cost-effective, resilient design strategies.

2.2 Resilience-Based Approaches In Seismic Design

Resilience-based seismic design (RBSD) extends **performance-based** practice by incorporating post-earthquake functionality, recovery time, and economic impacts into the design objectives. Early performance-based guidance most notably the **NEHRP Guidelines** (FEMA, 1997) and the subsequent **Prestandard** (FEMA, 2000) formalized target performance levels such as Immediate

Occupancy, Life Safety, and Collapse Prevention at specified hazard intensities, providing predictable structural performance under codified demands. These documents, developed originally for rehabilitation, established the performance-level vocabulary that underpins modern PBEE, but they did not explicitly quantify the duration or cost of recovery.

Building on this foundation, RBSD links **engineering demand parameters** (e.g., interstorey drift, floor accelerations, residual deformations) to **functional-recovery objectives** and **socio-economic consequences**. The MCEER framework by (Bruneau et al., 2003) conceptualizes resilience across technical, organizational, social, and economic dimensions, while analytical formulations by (Cimellaro et al., 2010) quantify a **resilience index** as the normalized area under a post-event functionality curve over a control period. These perspectives foreground both the depth of functional loss and the speed of restoration, not merely survival of life-safety limit states.

Quantitative resilience assessment has become tractable through PBEE-consistent, loss-based methods such as **FEMA P-58 (Second Edition)**, which integrate probabilistic hazard, component fragility, and consequence models to estimate **repair costs**, **repair/functional recovery time**, and **casualty risks** at multiple hazard levels. These outputs support resilience metrics and also enable **risk-informed** summaries (e.g., loss-exceedance curves and Expected/Average Annual Loss). Parallel national guidance on **functional recovery** (FEMA P-2090 / NIST SP-1254) recommends expressing goals explicitly in terms of **post-earthquake reoccupancy and functional-recovery time**.

A particularly relevant subset of RBSD for this thesis is **cost-based seismic resilience**, which evaluates **life-cycle cost (LCC)** over the building service life by combining **initial construction cost** with **expected future seismic consequences** (e.g., repair, rental/relocation, and casualty-related costs; downtime monetization). Within this stream, **Value-Based Design (VBD)** provides

a structured analytical approach, rather than a separate decision “framework”, that embeds value directly in the evaluation criterion and compares feasible designs on a common economic basis while retaining resilience measures where appropriate. In recent literature, **ETM-based** VBD and LCC studies demonstrate that efficiently sampling multiple hazard intensities with **Endurance Time Method (ETM)** enables practical optimization and screening of alternatives in terms of both **safety** and **long-term economic performance** (Charkhtab Basim et al., 2016; Mirfarhadi & Estekanchi, 2020). This combination is particularly salient for high-hazard, high-consequence urban contexts such as **Vancouver**, where resilience and cost considerations extend beyond minimum code compliance.

2.3 Value-Based Design in Seismic Engineering

Value-Based Design (VBD) is a design philosophy that integrates structural performance objectives with economic considerations over the entire service life of a building. Unlike prescriptive code-based design, which prioritizes minimum safety requirements, VBD evaluates alternatives using life-cycle cost analysis (LCCA) to account for initial construction costs, repair costs, downtime losses, and potential casualty costs. This approach enables the selection of designs that achieve an optimal trade-off between performance and total cost, making it particularly relevant in regions with high seismic hazard and dense urban development, such as Vancouver.

One of the foundational studies in this area is by (Charkhtab Basim et al., 2016), who proposed a VBD methodology combining LCCA with the Endurance Time Method (ETM) to evaluate seismic performance under multiple hazard levels. Their case study on a five-story steel moment-resisting frame demonstrated that while the VBD-optimized design had slightly higher initial costs, it resulted in the lowest total life-cycle cost compared to prescriptive and performance-based designs. The authors emphasized that integrating ETM into VBD significantly reduced computational time

compared to Incremental Dynamic Analysis (IDA) while providing detailed performance evaluation.

(Vitiello et al., 2017) extended the concept of cost-informed seismic design to retrofit scenarios. They developed a semi-probabilistic framework combining probabilistic seismic hazard analysis, nonlinear pushover analysis, and LCCA to assess different retrofit strategies for an existing RC building damaged in the 2009 L'Aquila earthquake. Their findings indicated that base isolation was the most cost-effective solution in high seismic zones, while fiber-reinforced polymer (FRP) strengthening was optimal for lower seismic demands. This study highlights the adaptability of cost-based resilience principles in both new design and retrofit contexts.

(Ciurlanti et al., 2023) provided a comparative cost-benefit analysis of alternative seismic design methodologies, including Force-Based Design (FBD), Direct Displacement-Based Design (DDBD), and low-damage technologies such as PRESSS systems. By evaluating 27 building configurations with varying ductility levels, they found that DDBD and low-damage systems reduced Expected Annual Losses (EAL) by 30–50% despite an 8% increase in initial costs. Their work reinforces that investing in enhanced performance often yields long-term economic benefits, a principle central to VBD.

From a practical implementation perspective, (Rad et al., 2021) proposed a Building Information Modeling (BIM)-integrated LCCA framework for conceptual-stage seismic design. By linking BIM with structural analysis (OpenSees) and energy simulation tools, their approach allowed real-time cost-performance feedback. A case study in Victoria, BC, showed that increasing initial construction costs by 4.6% reduced failure costs by 35.4% and total life-cycle costs by 10.4% over 45 years. This supports the argument that early-stage integration of cost-based resilience metrics can lead to substantial long-term savings.

In the domain of rapid evaluation, (Noureldin & Kim, 2020) proposed a parameterized LCC framework for low- to mid-rise buildings. Their method allowed for quick comparisons of design alternatives at the preliminary stage using simplified pushover analysis and single-degree-of-freedom approximations. Results demonstrated that increasing stiffness and strength significantly reduced life-cycle cost, while changes in ductility had a smaller effect. This framework's efficiency makes it suitable for early-stage screening before more detailed VBD assessments.

(Jebelli & Behnam, 2024) explored multi-objective optimization for VBD using the NSGA-II algorithm, focusing on both regular and setback irregular steel structures. By balancing initial construction cost and life-cycle cost under seismic loading, they found that irregular structures required up to 17% higher initial cost to achieve comparable reductions in life-cycle cost. Their findings highlight the importance of considering geometric irregularities in cost-based seismic resilience strategies.

Overall, the reviewed studies demonstrate that VBD offers a robust framework for optimizing seismic designs beyond minimum code compliance, balancing safety, functionality, and long-term economic performance. The methods range from detailed dynamic analysis using ETM to rapid parameterized evaluations, and from retrofit-focused applications to early-stage BIM integration. This diversity reflects VBD's flexibility and its alignment with resilience-based seismic design principles.

2.3.1 Optimization Approaches for Seismic Design

Optimization methods for seismic design enable stakeholders, building owners, policymakers, and communities, to select RC moment frame designs that balance resilience, cost, and recovery efficiency in high-risk regions like Vancouver. This study employs Multi-Criteria Decision-Making MCDM within a Value-Based Design VBD framework integrated with the Endurance

Time Method, using entropy-based data-driven weights combined with a Weighted Sum Model WSM ranking to prioritize designs based on seismic performance, life-cycle cost analysis LCCA, and stakeholder needs., such as owner cost savings and rapid community recovery. Studies demonstrate MCDM’s effectiveness, using methods like AHP and TOPSIS for sustainable and seismic design selection (Ogrodnik, 2019; Theilig et al., 2024), alongside other approaches like genetic algorithms (Charkhtab Basim et al., 2016) and non-dominated sorting genetic algorithm II (NSGA-II) (Jebelli & Behnam, 2024), which offer computational efficiency but less stakeholder interactivity. By leveraging AHP, this study optimizes RC moment frame designs under NBCC 2020, addressing Canada’s seismic and economic challenges.

In contrast to AHP- or TOPSIS-based studies in literature, the present work adopts an objective, entropy-weighted WSM scheme, where all criteria weights are inferred directly from the variability of the underlying data rather than from expert pairwise comparisons.

The table below summarizes optimization methods from reviewed studies, including MCDM applications relevant to this work:

Table 2.1: Optimization Methods in Seismic and Sustainable Design

Study	Structural Type	Performance Assessment Method	Optimization / Decision-Making Method	Key Cost Metric
(Charkhtab Basim et al., 2016)	Steel MRF	ETM	Genetic Algorithm (GA)	Total Life-Cycle Cost
(Vitiello et al., 2017)	RC retrofit	Nonlinear Pushover	Comparative Evaluation (LCCA-based)	Total Expected Cost
(Ciurlanti et al., 2023)	RC/Steel/Masonry	Parametric Study	Comparative Evaluation (EAL-based)	Expected Annual Loss (EAL)
(Rad et al., 2021)	RC	BIM + OpenSees	Integrated LCCA Feedback	Life-Cycle Cost

(Noureldin & Kim, 2020)	Low-/Mid-rise RC	Pushover + SDOF Approximation	Parameterized Evaluation	Life-Cycle Cost
(Jebelli & Behnam, 2024)	Steel (Regular/Irrregular)	Nonlinear Pushover	NSGA-II	Total Life-Cycle Cost
(Theilig et al., 2024)	RC Ceiling Structures	LCA + Seismic Analysis	AHP/ANP	Environmental/Cost Metrics
(Ogrodnik, 2019)	Residential Buildings	Energy + Cost Analysis	AHP/Fuzzy AHP	Cost and Environmental Impact

2.4 The Endurance Time Method (ETM) in Seismic Analysis

The Endurance Time Method (ETM) is a dynamic analysis technique in which a structure is subjected to an artificial, gradually intensifying ground motion so that a single time-history analysis can assess structural performance across multiple seismic hazard levels. A key advantage of ETM over Incremental Dynamic Analysis (IDA) is the substantial reduction in computational effort. While IDA requires running multiple nonlinear time-history analyses using a suite of ground motions scaled to different intensity levels, ETM condenses the process into a single run per excitation, with the seismic intensity increasing over time. For studies involving many ground motions and hazard levels, this can reduce the number of analyses from hundreds to only a few, resulting in significant time savings without sacrificing accuracy in median response trends (Estekanchi et al., 2007, 2020). These analyses produce continuous performance curves such as interstory drift, base shear, or member demands over an intensity “timeline,” enabling multi-hazard-level assessment in a single run.

Initial studies (Estekanchi et al., 2007) demonstrated that ETM could reproduce median structural response trends from IDA using a fraction of the computational effort. Comparative research has shown close agreement between ETM- and IDA-derived performance measures for elastic and moderately nonlinear systems, provided that ETEFs are well-calibrated to the hazard spectrum and the period range of interest.

In performance-based seismic design (PBSD), ETM enables efficient checking of multiple limit states (e.g., immediate occupancy, life safety, collapse prevention) in one analysis. In value-based design (VBD), ETM's continuous intensity sweep can be mapped to damage states, repair actions, downtime, and costs, allowing rapid integration with life-cycle cost analysis (LCCA). This has been shown to reduce computational time dramatically compared to running large-scale IDA suites while preserving decision-making accuracy (Charkhtab Basim et al., 2016).

Advantages over traditional methods:

- Computational efficiency: Captures a wide range of hazard levels in one run.
- Direct hazard mapping: Intensity progression can be linked directly to probability-based hazard levels.
- Versatility: Applicable to steel, RC, and hybrid systems, as well as retrofit evaluations.

Implementation considerations and limitations:

- Calibration: ETEFs must be tuned to the target hazard spectrum (NBCC 2020 or site-specific) and relevant structural period range.
- Record-to-record variability: Because ETM uses few synthetic records, additional ETEFs or uncertainty modeling may be needed to capture variability comparable to real ground motion suites.

- High-nonlinearity behavior: For collapse-level or highly nonlinear phenomena, targeted IDA checks remain important for validation.

In summary, ETM is a proven, time-efficient alternative to large IDA suites for generating seismic performance curves, especially when integrated into PBSD and VBD workflows. Its success depends on the careful calibration of ETEFs and validation against real ground motion response trends where high accuracy is critical.

2.4.1 The Endurance Time Excitation Function

Endurance Time Excitation Functions (ETEFs) are synthetic, nonstationary acceleration records specifically developed for the Endurance Time Method (ETM), enabling a single nonlinear dynamic analysis to evaluate structural performance across multiple seismic hazard levels. Their defining characteristic is a monotonic increase in intensity with time, allowing the creation of performance curves (e.g., drift vs. time) analogous to Incremental Dynamic Analysis (IDA) curves, but at a fraction of the computational cost.

From their inception, ETEF development has undergone five major generational advancements, each improving realism, spectral compatibility, and suitability for performance-based and value-based seismic design:

First generation – Random Vibration Theory (RVT): Constructed by summing harmonic components with random phases to form a stationary process, then modulating with a time-dependent envelope to achieve gradual intensity growth. While conceptually important, these signals often lacked accurate spectrum matching and realistic duration properties.

Second generation – Linear Spectral Matching: Optimization algorithms adjusted the synthetic record to match a target elastic response spectrum over a selected period range, improving code-spectrum compatibility for linear analysis but still limited in nonlinear fidelity.

Third generation – Nonlinear Spectral Matching: Targeted inelastic SDOF response spectra to ensure accurate representation of nonlinear structural behavior, extending ETEF reliability to collapse-sensitive and high-ductility systems.

Fourth generation – Duration and Energy Constraints: Incorporated cumulative absolute velocity (CAV), Arias intensity, and other duration/energy indicators into the objective function, producing records that more closely matched the energy content and shaking duration of real earthquakes.

Fifth generation – Damage Consistency: Introduced damage-based metrics such as maximum drift and Park–Ang damage index into the optimization, generating ETEFs capable of reproducing observed structural damage patterns from real ground motion suites.

Across these generations, multi-objective optimization has been central, with metaheuristic methods (e.g., genetic algorithms, particle swarm optimization) balancing spectral accuracy, duration measures, and damage indicators. Wavelet-domain and frequency-domain spectrum-matching techniques are often used in post-processing to further refine existing ETEFs so they meet code-based (e.g., NBCC 2020) or site-specific hazard spectra with high accuracy.

In practical applications, ETEFs often undergo response spectrum scaling to ensure compliance with regulatory target spectra, such as those defined in NBCC 2020 or site-specific seismic hazard curves. This involves scaling the synthetic record so that its elastic response spectrum closely matches the target spectrum over the structure’s fundamental period range. Such scaling can be applied either during the initial optimization or as a post-processing step using wavelet-based or frequency-domain spectrum matching algorithms (Al Atik & Abrahamson, 2010). In the present study, this approach was employed to align the generated ETEFs with the NBCC 2020 Vancouver site-specific hazard spectrum, thereby improving the accuracy of subsequent ETM-based performance and cost evaluations.

This evolving framework allows engineers to select the generation and calibration approach most appropriate to their objectives, from rapid preliminary screening in PBSO to detailed Value-Based Design workflows where accurate cost and damage prediction across multiple hazard levels is critical.

2.5 Research Gaps and Optimization

The literature review reveals several gaps that directly inform the focus of this research:

- a) *VBD–ETM integration for RC frame*: VBD is usually paired with pushover or SDOF models, while ETM is used mainly for response prediction; systematic coupling of VBD, ETM, and loss-based evaluation for RC moment frames across multiple hazard levels is still rare.
- b) *Lack of NBCC-calibrated Canadian applications*: Most VBD–LCC or ETM studies are international and not calibrated to NBCC 2020 spectra or Vancouver-type seismic hazard, so there is little evidence on how value-based optimization compares with NBCC minimum criteria for RC designs in Canada.
- c) *Limited use of explicit resilience metrics*: Performance-based studies commonly report drifts and losses, but quantitative resilience indices linking dynamic response to recovery and functionality are seldom computed and compared across alternatives.
- d) *Underdeveloped life-cycle, cost-based decision frameworks*: LCCA has been applied in retrofit and generic seismic studies, yet frameworks that link ETM-based demands, PBEE-type loss models, and LCCA to compare code-minimum and strengthened RC designs over the building life in Canada remain scarce.

This thesis directly targets these four gaps through an integrated ETM–VBD–LCCA–MCDM framework for an NBCC 2020 RC frame in Vancouver.

2.6 Conclusion

This review highlights the need for a multidisciplinary approach to seismic design that combines Value-Based Design principles, advanced analysis methods such as the Endurance Time Method, and robust cost-based performance evaluation through Life-Cycle Cost Analysis. Addressing the identified gaps will contribute to the development of cost-effective, resilient, and sustainable structural solutions. Future research should focus on implementing and validating these integrated methodologies in real-world projects, particularly within high seismic risk regions governed by NBCC 2020 provisions, to ensure both technical performance and economic efficiency.

CHAPTER 3: METHODOLOGY

3.1 Introduction

This chapter presents the methodology adopted to evaluate the seismic resilience of a six-story reinforced concrete (RC) building located in Vancouver, Canada. The process begins with the structural design of the building and its alternatives using the prescriptive provisions of the National Building Code of Canada (NBCC 2020). These baseline designs are then evaluated and optimized within a Value-Based Design (VBD) framework to identify cost-effective and resilient alternatives. This combined approach ensures full compliance with code requirements while enabling the exploration of designs that balance safety, performance, and economic efficiency.

3.2 Prescriptive Design Following NBCC 2020

The first stage of the study involves designing the six-story RC building in accordance with the prescriptive provisions of NBCC 2020. The design process incorporates seismic performance requirements and establishes a code-compliant baseline for subsequent optimization and comparative evaluation.

3.2.1 Description Of the Building

The case study building is a six-story reinforced concrete (RC) residential structure located in Vancouver City Centre, British Columbia (49°15'00" N, 123°06'00" W) a region of high seismicity, making it a critical location for resilience assessment. Each floor has a uniform story height of 3.65 m.

The lateral load-resisting system consists of concrete moment-resisting frames (CMRFs), positioned along the building's shorter direction to resist both seismic and gravity loads. The structural layout (Figure 3.1) is symmetrical, with the interior frame located near the center of rigidity to minimize accidental torsion effects.

For analytical modelling, the building is idealized as a series of two-dimensional frames in two perpendicular directions. This research focuses on the CMRFs in the weaker direction of the building, where the structural response is expected to be most critical.

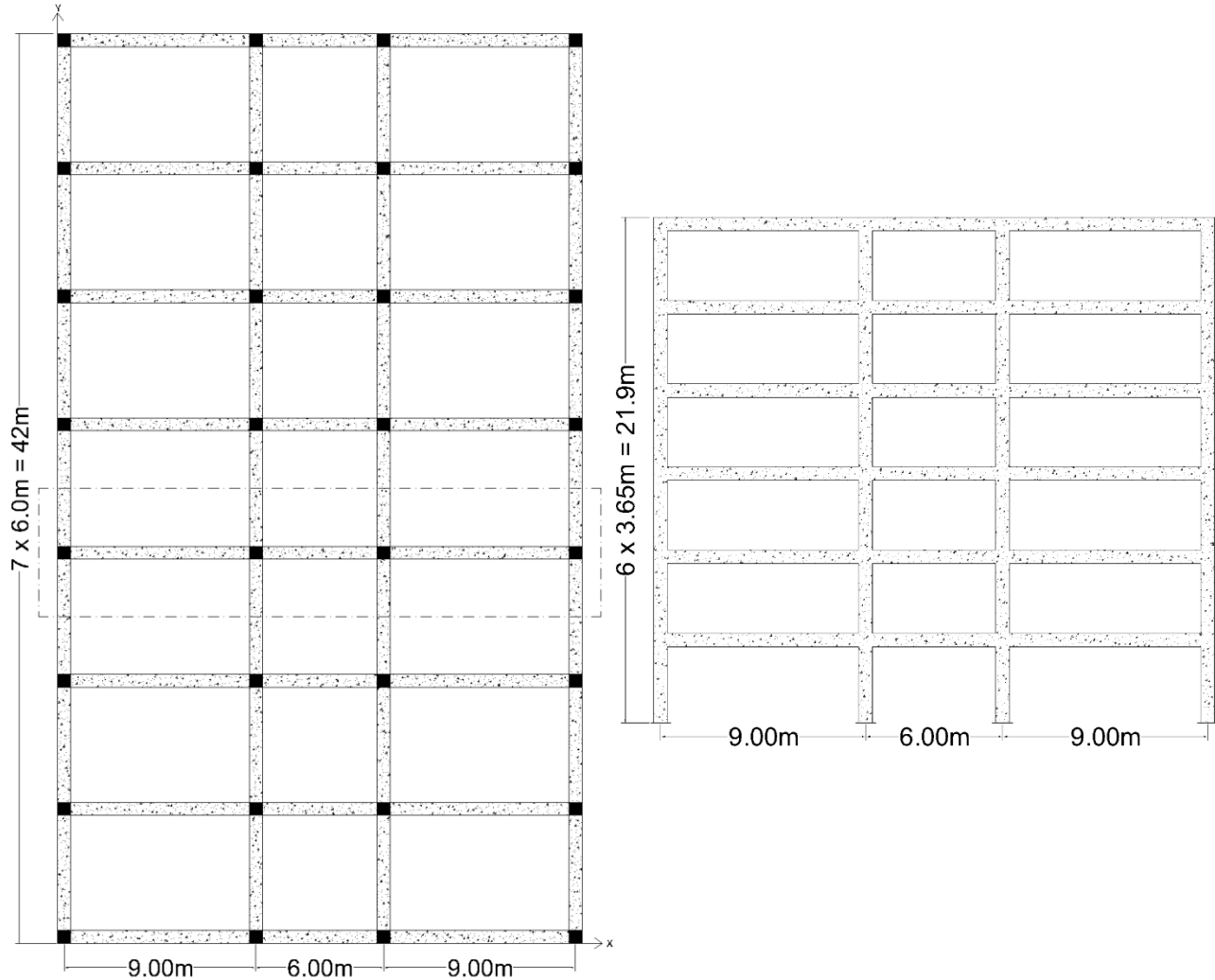


Figure 3.1: Typical floor layout(left) and elevation(right) of the 6 storied building (**Badal & Tesfamariam, 2023**)

3.2.2 Design Parameters and Assumptions

For simplicity, the beam width is fixed at 400 mm for all frame beam sections. Based on the design of the longest span, the beam depth is set to 550 mm in the weaker (E–W) direction and 450 mm

in the stronger (N–S) direction. The slab thickness is designed as 200 mm to meet strength and serviceability requirements; however, its in-plane stiffness contribution is not considered in the structural analysis. All columns are assumed to be rectangular with uniform dimensions throughout the building for architectural consistency and ease of construction.

The unconfined compressive strength f'_c of the reinforced concrete is taken as **30MPa** with an initial elastic modulus E_c of **24649 MPa**. The unit weight of reinforced concrete is **24 kN/m³** and the reinforcing steel yield strength f_y is **400 MPa**. Masonry infill walls are excluded from the analysis, although partition wall loads are considered for gravity load calculation.

Design live loads are taken as **2.4 kN/m²** for all the floor and **2.2 kN/m²** for the roof including the snow load. Additional superimposed loads include **1.0 kN/m²** for partition walls, **0.5 kN/m²** for mechanical services, and **0.5 kN/m²** for floor finishes. To account for the low probability of full live load occupancy, live load reduction factors (LLRF) are applied using:

$$LLRF = 0.3 + \sqrt{\frac{9.8}{A_t}}$$

Where A_t is the tributary area in square meters.

In accordance with NBCC 2020 seismic provisions, the structure is classified as regular, with a total height under 60 m and a fundamental period T_a in both orthogonal directions below 2 seconds. The Equivalent Static Force Procedure (ESFP) is therefore deemed appropriate for the prescriptive design phase. The base shear is calculated and the frames are designed for full ductility, with a ductility-related force modification factor $R_d = 4$, and an overstrength-related factor $R_o = 1.7$. The building site is located on soft rock with a shear wave velocity V_{s30} of 450 m/s. Given its classification as a residential apartment, the importance factor I is 1 and a 5% damping ratio is assumed. Two primary load combinations are used: the gravity load combination

($1.25D+1.5L$) and the lateral load combination ($D+0.5L+E$), where D, L, and E represent dead, live, and lateral earthquake-induced loads, respectively.

NBCC 2020 specifies the fundamental design period. However, the period obtained from modal analysis often exceeds the empirical value from the code. To address this, NBCC permits increasing the empirical period by up to 50%, or adopting the period from modal analysis, whichever is smaller. This adjustment reduces the design base shear, resulting in a more economical design without compromising safety.

3.2.3 Design Process Following NBCC 2020

The initial design of the six-story reinforced concrete (RC) building was carried out in accordance with the **NBCC 2020**. The process began with the **gravity load combination** $1.25D+1.5L$, using preliminary member dimensions. Static analysis results were used to size elements and determine reinforcement, with adjustments made iteratively until the sections met strength and detailing requirements.

A **modal analysis** was then performed to determine the fundamental period of vibration, which was compared with the NBCC empirical period. The final period was selected in compliance with the code's adjustment limits.

Equivalent static lateral forces were calculated per NBCC 2020 and applied to the model. The design was verified under the **lateral load combination** $D+0.5L+E$, with interstory drifts adjusted for inelastic effects using the force modification factors R_d and R_o . Drifts were checked against the code-specified limit of **2.5% of the story height**, and member sizes were revised if necessary.

The final design was also reviewed for compliance with the **strong column–weak beam** principle to promote ductile performance and ensure plastic hinges form in beams rather than columns during seismic events.

3.2.4 Design Base Shear

The design base shear for the six-story building is calculated as 393.83 kN, corresponding to a base shear coefficient V/W of 6.04%. Here, V is the design base shear and W is the total seismic weight of the structure, defined as the sum of the dead load plus 25% of the snow load, in accordance with NBCC 2020 provisions.

Lateral forces were distributed across the building height following the NBCC equivalent static procedure. The resulting force distribution is shown in Figure 3.2, indicating that the maximum story-level lateral force of 138.54 kN occurs at the roof level (sixth story), with forces decreasing progressively toward the base.

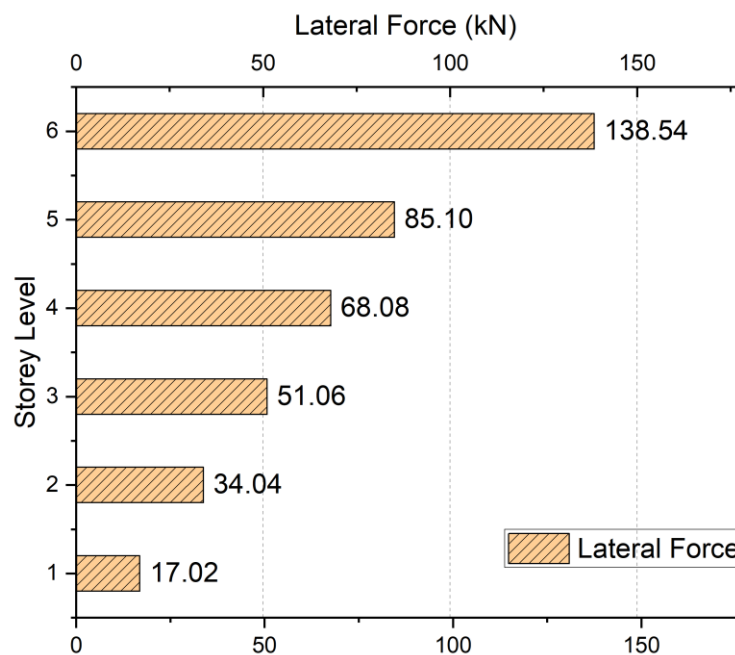


Figure 3.2: Design Base Shear Distribution

3.2.5 Inter-Storey Drifts

Interstory drift ratios were evaluated using **static analysis**, with the NBCC 2020 limit set at **2.5% of story height**. For the six-story building, the **maximum drift** occurs at the **second story**, with a value of **1.89%**, while values decrease progressively toward the roof and base (Figure 3.3).

All computed drifts remain within the NBCC 2020 serviceability limit, confirming that the building's lateral deformation is acceptable under design-level seismic loading. The concentration of higher drifts in the lower stories is consistent with the expected behavior of mid-rise RC frame structures subjected to lateral forces.

The application of the **Equivalent Static Force Procedure (ESFP)** during the prescriptive design phase provided an adequate representation of the building's drift performance, ensuring compliance with serviceability and safety requirements without the need for additional dynamic analysis at this stage.

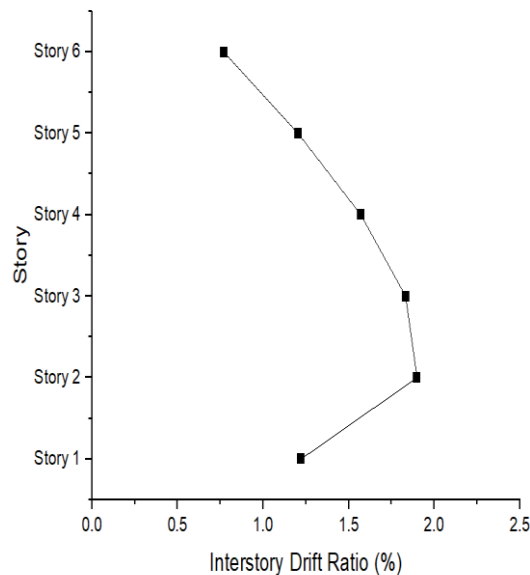


Figure 3.3: Estimated inter-storey drifts of the frame

3.2.6 Strong Column-Weak Beam Requirement

In accordance with NBCC 2020 seismic provisions, the structural design ensures that plastic hinges form in beams rather than columns, thereby preventing soft-story mechanisms that can cause sudden structural failure. To achieve this, the combined nominal flexural resistance of columns framing into a joint must exceed that of the beams at the same joint:

$$\sum M_{nc} \geq \sum M_{pb}$$

where:

$\sum M_{nc}$ = sum of the nominal flexural resistances of the columns meeting at a specific joint.

$\sum M_{pb}$ = sum of the probable flexural resistances of the beams meeting at the same joint.

This check is conducted at all joints after completion of the gravity load design and application of lateral loads. If any joint fails to meet the criterion, the design is revised iteratively by adjusting member dimensions or reinforcement details until compliance is achieved.

This requirement is essential for ensuring ductile seismic performance, allowing the structure to dissipate energy effectively and reducing the risk of collapse during strong earthquakes.

3.2.7 Final Design Details

After completing several iterations, the final cross-sectional dimensions and reinforcement details were determined for the prescriptive design. These details are summarized in Tables 3.1 and 3.2, with a sample representation of the cross sections illustrated in Figure 3.4.

Table 3.1: Final Cross Sections of Beams

Storey	External Beams		Internal Beams	
	Top	Bottom	Top	Bottom
1	10#20M	7#20M	8#20M	3#20M
2	10#20M	7#20M	8#20M	3#20M
3	10#20M	7#20M	8#20M	3#20M
4	10#20M	7#20M	8#20M	3#20M
5	10#20M	7#20M	8#20M	3#20M
6	10#20M	7#20M	8#20M	3#20M

Table 3.2: Final Cross Sections of Columns

	External		Internal	
Storey	Dimensions (mm)	Reinforcement	Dimensions (mm)	Reinforcement
1	500x500	8#25M	500x500	8#25M
2	500x500	8#25M	500x500	8#25M
3	500x500	8#25M	500x500	8#25M
4	500x500	8#25M	500x500	8#25M
5	500x500	8#25M	500x500	8#25M
6	500x500	8#25M	500x500	8#25M+4#20M

3.3 Value Based Design

Value-Based Design (VBD) represents a paradigm shift in structural engineering, moving beyond prescriptive code compliance toward a holistic evaluation of structural alternatives based on a comprehensive assessment of value. Unlike traditional approaches, which often prioritize safety and minimum cost as isolated objectives, value-based design seeks to maximize overall project

value by considering not only initial construction costs and regulatory compliance, but also long-term factors such as seismic resilience, life-cycle performance, usability, and broader socioeconomic impacts (Estekanchi et al., 2023; Haselton et al., 2012).

Central to the VBD philosophy is the recognition that the “best” design is not necessarily the one with the lowest initial cost, but rather the one that delivers the most favorable balance of performance, sustainability, and user requirements over the structure’s lifespan. By integrating advanced seismic analysis methods such as the Endurance Time Method (ETM) and robust life cycle cost assessment, value-based design enables stakeholders to identify and select structural solutions that optimize both engineering performance and long-term investment.

In this study, a value-based design framework is implemented to systematically generate, evaluate, and optimize a population of structural design alternatives for a 6-story reinforced concrete frame building, subject to the seismic hazard conditions of Vancouver, Canada. The framework combines ETM-based dynamic analysis, PBEE-consistent loss and life-cycle cost calculations, a Resiliency Index, and an entropy-based Weighted Sum Model (WSM) for multi-criteria decision-making, rather than relying on evolutionary optimization algorithms such as genetic algorithms.

3.3.1 Value-Based Design Framework

The workflow adopted in this research follows a structured value-based design framework (see Figure 3.5), adapted from the pioneering work of (Mirfarhadi & Estekanchi, 2020) and further tailored to meet the requirements of Canadian seismic design standards (NBCC 2020). In this adaptation, ETM-derived multi-hazard drift demands replace single-hazard or pushover-based estimates. Here, a Resiliency Index is evaluated alongside life-cycle cost, and an entropy-based Weighted Sum Model (WSM) is used instead of subjective AHP/TOPSIS-type weighting to rank design alternatives.

The value-based optimization loop developed here enables a direct, performance-driven selection of the most efficient and resilient design, PBEE-consistent loss and life-cycle cost calculations; and the entropy-weighted MCDM ensures that both structural performance and long-term value are systematically maximized in the final design.

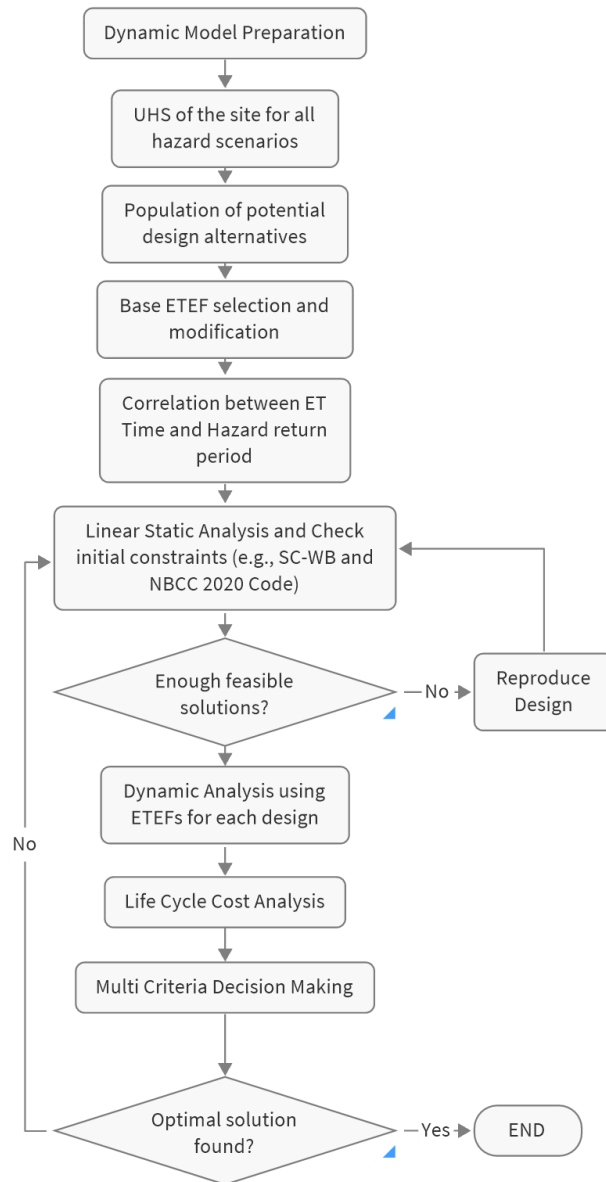


Figure 3.5. Value-Based Design Framework employed in this study.

3.3.2 Design Alternatives

To enable a comprehensive value-based optimization, a broad set of structural design alternatives was systematically developed for the target building. The alternatives were defined by varying key design parameters, primarily column cross-sectional dimensions and longitudinal reinforcement ratios, while ensuring compliance with NBCC 2020 requirements for strength, ductility, and serviceability.

The range of column sizes and reinforcement ratios was selected based on practical constructability limits and standard engineering practice in Canadian high-seismic regions. For each alternative, beams were maintained with consistent dimensions and reinforcement to isolate the impact of column variation on seismic performance and cost. Additional parameters, such as concrete compressive strength and reinforcement grade, were held constant across all alternatives for consistency.

A total of 42 design alternatives were generated and subjected to initial screening for code compliance, including strong column–weak beam (SCWB) checks. Each alternative was then evaluated through nonlinear time history analysis using Endurance Time Excitation Functions (ETEFs), with performance indicators including peak interstory drift ratios, maximum base shear, and predicted damage states recorded for comparative assessment.

Summary tables listing the geometric and reinforcement properties of all alternatives are provided in Table 3.3.

Table 3.3: Design Alternatives Details (Column Dimension and Reinforcement)

DA	Width, mm	Depth,mm	ReBar Qty #25M	Reinforcement Ratio
DA 01	500	500	8	1.60%
DA 02	500	500	10	2.00%
DA 03	500	500	12	2.40%
DA 04	500	500	14	2.80%
DA 05	500	500	16	3.20%
DA 06	500	500	18	3.60%
DA 07	500	500	20	4.00%
DA 08	500	550	8	1.45%
DA 09	500	550	10	1.82%
DA 10	500	550	12	2.18%
DA 11	500	550	14	2.55%
DA 12	500	550	16	2.91%
DA 13	500	550	18	3.27%
DA 14	500	550	20	3.64%
DA 15	500	550	22	4.00%
DA 16	550	550	10	1.65%
DA 17	550	550	12	1.98%
DA 18	550	550	14	2.31%
DA 19	550	550	16	2.64%
DA 20	550	550	18	2.98%
DA 21	550	550	20	3.31%
DA 22	550	550	22	3.64%
DA 23	550	550	24	3.97%
DA 24	550	600	10	1.52%
DA 25	550	600	12	1.82%
DA 26	550	600	14	2.12%
DA 27	550	600	16	2.42%
DA 28	550	600	18	2.73%
DA 29	550	600	20	3.03%
DA 30	550	600	22	3.33%
DA 31	550	600	24	3.64%
DA 32	550	600	26	3.94%
DA 33	600	600	10	1.39%
DA 34	600	600	12	1.67%
DA 35	600	600	14	1.94%
DA 36	600	600	16	2.22%
DA 37	600	600	18	2.50%
DA 38	600	600	20	2.78%
DA 39	600	600	22	3.06%
DA 40	600	600	24	3.33%
DA 41	600	600	26	3.61%
DA 42	600	600	28	3.89%

3.3.3 Life Cycle Cost Assessment

To evaluate the long-term economic performance of each design alternative, a life-cycle cost assessment (LCCA) was conducted in constant Canadian dollars (CAD) and discounted to a common base year. For a design defined by the vector $\mathbf{s}$ and a planning horizon $\mathbf{t}$ (years), the total cost $\mathbf{C}_{TOT}$ is written as:

$$\mathbf{C}_{TOT}(\mathbf{t}, \mathbf{s}) = \mathbf{C}_{IN}(\mathbf{s}) + \mathbf{C}_{LC}(\mathbf{t}, \mathbf{s})$$

Where, $\mathbf{C}_{IN}(\mathbf{s})$: initial construction cost, and $\mathbf{C}_{LC}(\mathbf{t}, \mathbf{s})$: present value of the life cycle cost over lifetime (Mitropoulou et al., 2011).

3.3.3.1 Initial Cost

The initial cost establishes the baseline of the LCCA and includes only direct structural construction costs. To maintain comparability across alternatives, land and non-structural systems (architectural finishes; mechanical, electrical, and plumbing) are excluded. Quantity take-off follows standard practice: column costs are obtained from concrete volume and reinforcing steel weight computed from member sizes and reinforcement ratios ($\approx 1\text{--}4\%$ in this study), multiplied by unit rates for concrete and steel. Beam and slab costs are estimated from unit area rates consistent with the selected framing and slab system. Foundation costs are computed using a standardized footprint and depth assumption to isolate design-dependent effects; this assumption is held constant across all alternatives.

All unit rates were sourced from RSMeans data (Gordian) for the selected price year. For columns, quantities were computed from concrete volume and reinforcing steel weight (reinforcement ratios $\approx 1\text{--}4\%$), then priced using RSMeans line items for cast-in-place concrete and reinforcing steel. Beam and slab costs used RSMeans assemblies/square-foot items consistent with the chosen framing system; foundation costs used standardized footprint/depth assumptions applied

uniformly to all designs. Rates were adjusted using the RSMeans City Cost Index (CCI) for the project location and converted to CAD at the prevailing exchange rate for the study's base (price) year. All figures are reported in constant-CAD real terms (i.e., no general inflation), ensuring consistency with the real discount rate used later in the present-value calculations.

The calculated initial costs form a critical component of the life-cycle cost analysis, providing a baseline for comparing the economic performance of all design alternatives. These costs are integrated with operational, damage, and maintenance costs in subsequent steps to offer a comprehensive evaluation of each alternative's long-term viability.

3.3.3.2 Life-Cycle Cost

Life-cycle seismic losses are evaluated within the PEER performance-based earthquake engineering (PBEE) formulation, which probabilistically links intensity measures (***im***), engineering demand parameters (***edp***), damage measures (***dm***), and decision variables (***dv***) (e.g., repair cost, repair time). In exceedance form, the mean annual frequency that a DV exceeds a level ***dv*** is:

$$\lambda(dv) = \iiint_{dm\ edp\ im} G(dv|dm) |dG(dm|edp)| |dG(edp|im)| |d\lambda(im)$$

Where: G() are conditional (exceedance) distributions.

The PEER framework divides the LCCA process into four key stages; seismic hazard analysis, response analysis, damage analysis, and loss analysis, each interconnected through probabilistic models. These stages enable an integrated assessment that bridges seismic intensity, structural response, damage outcomes, and economic consequences.

Seismic Hazard: Seismic hazard quantifies the likelihood of ground motion intensities at the site of the structure. The hazard data for the site was obtained in terms of annual probabilities of

exceedance for ground motion intensities, which align with established seismic hazard curves. Probabilities stated over a 50-year period (e.g., 40%, 30%, 20%, 14%, 10%, 7%, 5%, 2% in 50 years) are converted to annual rates via the Poisson model $APE = 1 - \exp[-\lambda \cdot 1yr]$ with $\lambda = -\ln(1 - P_{50})/50$. These hazard levels were used to calibrate the endurance time excitation function (ETEF) and to anchor response evaluation at multiple shaking intensities.

Table 3.4: Description of damage state (FEMA 227, 1992)

Damage State	Description
None	No visible or measurable damage
Slight	Limited, localized minor damage that does not impair structural integrity and does not require repairs.
Light	Significant localized damage of non-critical components, generally not impairing functionality or requiring urgent repairs.
Moderate	Localized damage of structural components requiring targeted repairs to maintain structural safety and serviceability.
Heavy	Extensive damage compromising multiple structural elements, necessitating major structural repairs.
Major	Widespread structural damage that could lead to partial collapse or extensive reconstruction needs.
Destroyed	Total collapse or failure of most structural and non-structural components, rendering the facility unusable.

Table 3.5: Limit state drift ratio and floor acceleration limits for bare Moment resisting Frames

Performance level	Damage States	Interstorey Drift (%) ATC-13 (1985), Mitropoulou 2011
I	None	$\Delta \leq 0.1$
II	Slight	$0.1 < \Delta \leq 0.2$
III	Light	$0.2 < \Delta \leq 0.4$
IV	Moderate	$0.4 < \Delta \leq 1.0$
V	Heavy	$1.0 < \Delta \leq 1.8$
VI	Major	$1.8 < \Delta \leq 3.0$
VII	Destroyed	$\Delta > 3.0$

For each design alternative and hazard level (ETEF_A run), the interstorey drift ratio (IDR) was evaluated over the full 40-s record, and the peak IDR was used to quantify damage. Drift-based damage thresholds (augmented, where relevant, by floor-acceleration limits) mapped the EDPs to damage measures (DMs). To avoid plateau effects when IDRs fall near state boundaries, damage was computed using within-band interpolation between adjacent states; the resulting damage shares were then used to form expected repair consequences (repair cost and time), casualty rates, and any derived indices (e.g., MDI, resilience).

Decision Variables and loss curves: The primary decision variable (DV) in this study is life-cycle cost, formed from initial cost and the present value of expected seismic losses. For each cost component LLL (e.g., damage repair, contents, rental interruption, relocation, injury, fatality), we construct a loss-exceedance curve (a sample loss curve presented figure 3.6) by pairing the component value at each hazard level with its annual probability (or mean annual frequency) of exceedance. Let $\Delta_L(l)$ denote the mean annual frequency of exceeding loss level l for component L. The expected annual loss (EAL) for that component is the area under the loss-exceedance curve:

$$EAL[L] = \int_0^{\infty} \Delta_L(l) dl$$

Which is the standard PBEE identity $E[L] = \int_0^{\infty} P > l \, dl$ in annualized form. In discrete form (eight hazard levels) with losses sorted in ascending order $l_1 \leq \dots \leq l_n$ and corresponding annual exceedance values $\Delta_1 \dots \Delta_n$ we evaluate EAL by the trapezoidal rule:

$$EAL[L] \approx \sum_{i=1}^{n-1} \frac{1}{2} (\Delta_i + \Delta_{i+1}) (l_{i+1} - l_i)$$

Axes and interpretation. The horizontal axis is the cost level l (e.g., repair cost), and the vertical axis is the annual exceedance (probability or frequency) of exceeding l . Each curve captures the stochastic link from hazard $\rightarrow$ response $\rightarrow$ damage $\rightarrow$ cost for a given design; its area yields the component's EAL. Summing EALs across components provides the seismic EAL for the building, which is then converted to present value in the LCCA

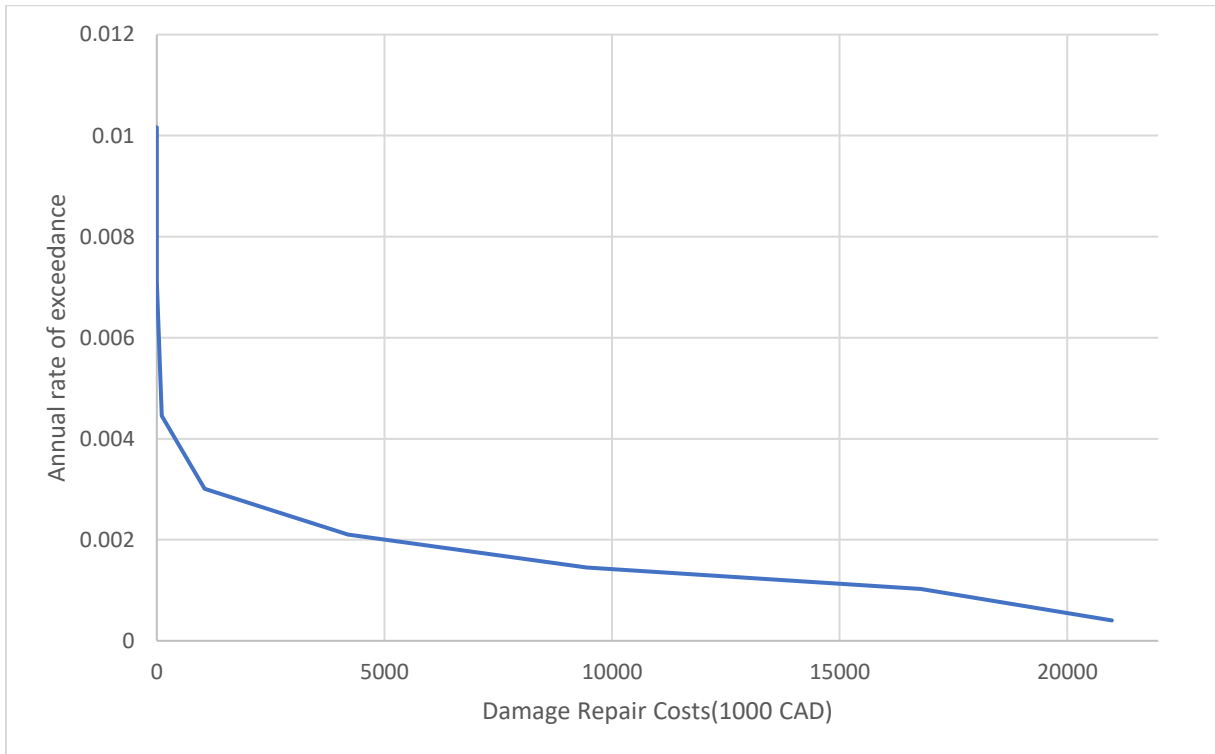


Figure 3.6: A sample loss curve

The loss curve provides valuable insights into the performance of each design alternative, offering a probabilistic basis for decision-making in seismic design. The results of the loss curve analysis directly feed into life-cycle cost estimations, enabling comparison and optimization of design alternatives.

The life-cycle cost consisting of several key components which can be formulated as follows:

$$C_{LC} = C_{dam} + C_{con} + C_{rent} + C_{reloc} + C_{inj.m} + C_{inj.s} + C_{fat}$$

Life Cycle cost Components

- i. Damage Repair (C_{dam}) : This component captures the cost required to repair structural and non-structural damages sustained during a seismic event. The replacement cost includes materials, labor, and construction equipment costs necessary for repair. The mean damage index represents the average damage level for the structure, typically derived from damage state models (e.g., FEMA 227). It is multiplied by the floor area to scale the repair cost to the building's size. This component provides insight into the direct financial burden of restoring the building to its pre-disaster condition.
- ii. Loss of Contents (C_{con}) : This represents the financial loss associated with the damage or destruction of building contents, such as furniture, equipment, and stored goods. The unit cost of contents is based on social function classification. The mean damage index is used here to scale content losses relative to the structural damage experienced.
- iii. Loss of Rental (C_{rent}) : This component reflects the opportunity cost of losing rental income due to building downtime. The rental rate is typically determined based on market conditions in the building's location, while the gross leasable area accounts for the rentable space. Downtime is the estimated time the building is non-operational and is usually derived from repair schedules or industry averages.

- iv. Relocation Cost (C_{reloc}) : Relocation costs arise when occupants must vacate the premises during repair or reconstruction. This includes expenses for temporary housing, moving logistics, and alternative workspace rentals. The relocation cost per unit area is multiplied by the gross leasable area and downtime to provide an estimate.
- v. Minor injury ($C_{inj,m}$) : This component quantifies the economic impact of minor injuries sustained during a seismic event. Minor injuries are defined as those requiring minimal medical attention, such as first aid or outpatient care. The cost per person is multiplied by the occupancy rate (the number of people typically present in the building), the floor area, and the expected rate of minor injuries derived from seismic vulnerability models.

The number of people exposed to risk, N , was estimated using the NBCC 2020 Structural Commentary L formulation for maximum occupancy:

$$N = A_{occ} \times \rho_{occ} \times \phi_{dur}$$

where A_{occ} is the floor area exposed to risk in m^2 , ρ_{occ} is the occupant density in persons/ m^2 , and ϕ_{dur} is the duration factor, defined as the average number of hours of human occupancy per week divided by 100. For residential use, $\rho_{occ} = 0.05$ persons/ m^2 and $\phi_{dur} = 1.0$ (100 hours/week), consistent with Table L-6 of the NBCC Commentary. The resulting N value was then used as the occupancy rate term in the minor injury, serious injury, and fatality cost components summarized in Table 3.6

- vi. Serious injury ($C_{inj,s}$) : Similar to minor injuries, this component calculates the financial impact of severe injuries requiring significant medical care, such as hospitalization or surgeries. The calculation methodology mirrors that of minor injury costs but uses rates for serious injuries and their associated medical costs.

- vii. Human fatality (C_{fat}) : This component reflects the societal and economic cost of fatalities caused by seismic events. The fatality cost per person is derived from statistical life value studies and includes direct (e.g., lost income) and indirect costs (e.g., societal impact). It is scaled using the floor area, occupancy rate, and expected fatality rate.

Table 3.6: Summary of Cost component calculation formulas of LCCA

Cost Component	Formula	Basic Cost
Damage Repair (C_{dam})	Replacement cost $\times$ floor area $\times$ mean damage index	3,469 CAD/m²
Loss of Contents (C_{con})	Unit contents cost $\times$ floor area $\times$ mean damage index	1,306 CAD/m²
Loss of Rental (C_{rent})	Rental rate $\times$ gross leasable area $\times$ loss of function time	25 CAD/month/m²
Relocation (C_{reloc})	Relocation cost $\times$ gross leasable area $\times$ down time	677.24 CAD/year/m²
Minor injury ($C_{inj.m}$)	Minor injury cost per person $\times$ floor area $\times$ occupancy rate $\times$ expected minor injury rate	1,000 CAD/person
Serious injury ($C_{inj.s}$)	Serious injury cost per person $\times$ floor area $\times$ occupancy rate $\times$ expected serious injury rate	10,000 CAD/person
Human fatality (C_{fat})	Human fatality cost per person $\times$ floor area $\times$ occupancy rate $\times$ expected death rate	6,637,120 CAD/person

Table 3.7: Damage State Parameters for Cost Calculations (FEMA 227, 1992)

Damage States	Mean Damage Index (%)	Expected minor injury rate	Expected serious injury rate	Expected death rate	Loss of function time (days)	Down time (days)
(I)-None	0	0	0	0	0	0
(II)-Slight	0.5	0.00003	0.000004	0.000001	0.8	0.8
(III)-Light	5	0.0003	0.00004	0.00001	3.3	3.3
(IV)-Moderate	20	0.003	0.0004	0.0001	10.5	10.5

(V)-Heavy	45	0.03	0.004	0.001	71.9	71.9
(VI)-Major	80	0.3	0.04	0.01	146.6	146.6
(VII)-Destroyed	100	0.4	0.4	0.2	211.9	211.9

To accurately reflect the time value of money, a discount rate of 3% was applied in the life-cycle cost analysis (LCCA). The 3% discount rate reflects a standard practice in infrastructure-related life-cycle cost analyses. It is considered a conservative rate, representing long-term financial trends and the relatively stable risk profile of structural investments. This rate accounts for inflation, opportunity costs, and other financial considerations, making it a suitable choice for evaluating the economic implications of seismic resilience.

To account for 50years expected loss at a 3% discount rate, following formula was used

$$PV_t = EAL * \frac{1 - (1 + r)^{-t}}{r}$$

Where, EAL= Expected Annual Loss, r: Discount rate (3% or 0.03 in this case) and t: Life Span in years.

3.4 Development of a Dynamic Model

A nonlinear finite element model of the six-story reinforced concrete (RC) moment frame was developed using OpenSeesPy (Figure 3.7) (Zhu et al., 2018), a Python-based interface to the OpenSees simulation framework, widely used for advanced earthquake engineering research and nonlinear structural analysis (Mckenna, 2011). The model was configured to facilitate both the Endurance Time Method (ETM) and Incremental Dynamic Analysis (IDA), supporting direct benchmarking and validation with real ground motions. A two-dimensional frame idealization was adopted in the plane of the moment-resisting frame; diaphragm behavior was represented only

through lumped floor masses and in-plane rigidity, and no explicit diaphragm flexibility or torsional response was modeled.

Columns were modeled as nonlinear force-based beam-column elements using detailed fiber-section discretization (**Scott & Fenves, 2006**). Each section comprised 164 concrete fibers (64 core and 100 cover), with core concrete fibers employing the Concrete02 (Kent–Scott–Park) material model for confined regions, and cover fibers modeled as unconfined concrete. Material parameters were based on CSA A23.3 and the (**Mander et al., 1988**) confinement model. Longitudinal reinforcement was modeled with the Steel02 bilinear cyclic material.

Beams were modeled as elastic elements, each with nonlinear rotational end springs at both ends using the IMKPeakOriented hysteretic material to capture inelastic hinge behavior (**Dimitrios Lignos and Krawinkler, 2011**).

Beam–column joints were modeled with the OpenSees Joint2D element (Alath & Kunnath, 1995; Lowes & Mitra, 2004). Panel shear behavior was represented by an elastic-perfectly-plastic shear spring whose initial stiffness was estimated from $k = GA_j/L_j$ and whose nominal strength was limited by ACI 352R-02 type $\sqrt{f'_c}$ joint shear stresses (Alcocer, 2012). The model thus captures joint shear flexibility in an approximate but code-consistent manner rather than a full test-based calibration.

Effective flexural stiffness modifiers of $0.35I_g$ were applied to the elastic beam elements to represent cracking, while column flexural behavior was captured directly by the nonlinear fiber sections without applying an explicit I_g reduction factor..

Rayleigh damping was implemented to provide 5% critical damping in the first and third vibration modes (**Chopra, 2017**), using mass and tangent-stiffness proportional coefficients, as recommended for nonlinear time history analysis.

Soil-structure interaction and foundation flexibility were included through rotational springs at the base of columns, with stiffnesses approximated using ASCE 41-17 provisions for shallow foundations. Global P- Δ effects were captured by applying gravity loads directly to the frame columns and using a P- Δ geometric transformation for the column elements, so second-order effects were included explicitly in the nonlinear analysis of the moment-resisting frames.

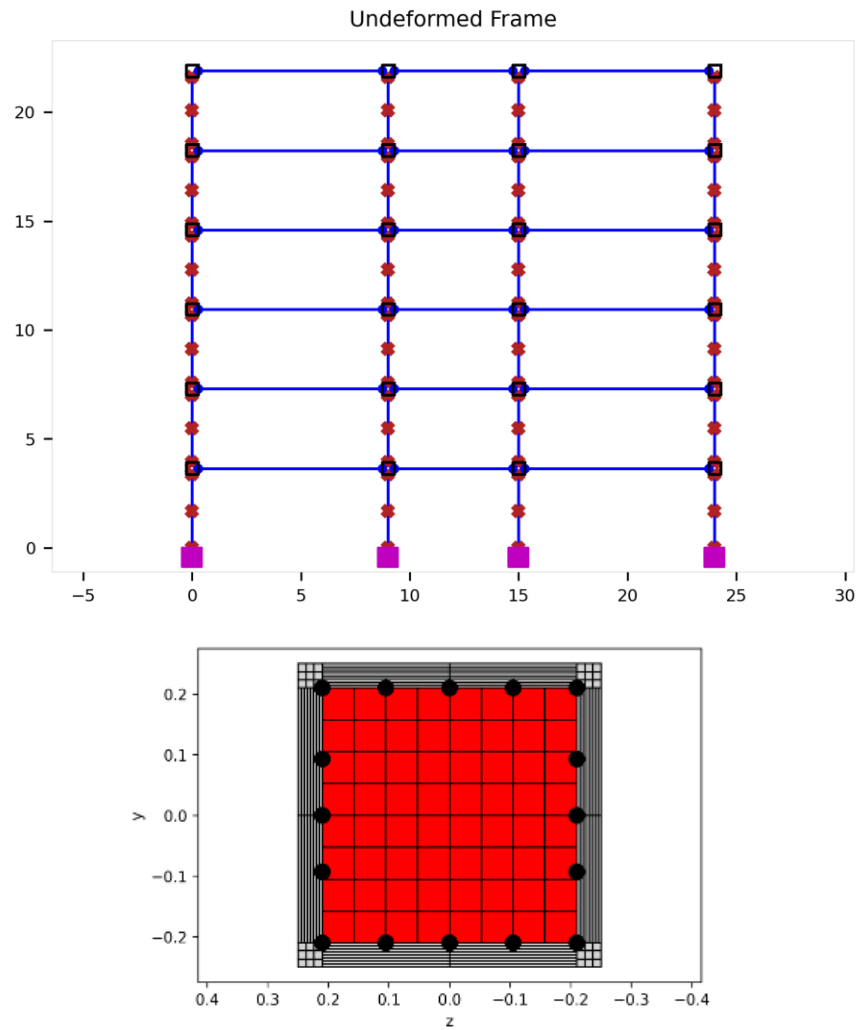


Figure 3.7: View of Dynamic Model from Openseespy (top) and fiber section detailing of column (bottom)

Floor seismic mass was lumped at beam-column joint nodes and distributed according to tributary area to interior and exterior columns, ensuring mass participation accurately reflected the design dead and live loads (including 0.5L, per NBCC).

Gravity loads were applied first to establish equilibrium prior to dynamic analysis. Nonlinear time history analysis used the Newmark integration method ($\Delta t = 0.01$ s), with strict convergence criteria for displacement and energy. Seismic excitation was introduced using Endurance Time Excitation Functions (ETEFs), spectrally matched to the site-specific response spectrum for Vancouver per NBCC 2020, to represent progressive increases in seismic intensity over a single analysis. Multiple ETEF records were analyzed to capture record-to-record variability.

3.5 Endurance Time Method (ETM)

The Endurance Time Method (ETM) was selected as the primary dynamic analysis approach in this study for its ability to evaluate structural performance across multiple seismic hazard levels within a single nonlinear time-history analysis. This efficiency is achieved through Endurance Time Excitation Functions (ETEFs) synthetic, nonstationary acceleration records that increase in intensity over time and are calibrated to match the NBCC 2020 Vancouver site-specific hazard spectrum. In this work, baseline ETM accelerograms from the ETM open-source library were adjusted using the Al Atik and Abrahamson (2010) spectral-matching algorithm implemented in SeismoMatch so that their response spectra closely matched the NBCC 2020 target spectra at the selected hazard levels.

ETM produces continuous performance curves, such as peak interstory drift ratios and base shear, which are directly linked to damage states for subsequent life-cycle cost (LCC) and multi-criteria decision-making (MCDM) assessments. Compared to Incremental Dynamic Analysis (IDA), which requires numerous scaled analyses using real or spectrally matched records, ETM achieves

comparable median response trends with only a few well-calibrated excitations, resulting in significant computational time savings. In this thesis, IDA is therefore used as a benchmark method to validate the calibrated ETEFs, after which ETM serves as the primary analysis method for evaluating the design alternatives.

The detailed procedures for ETEF generation, spectrum matching, calibration, and validation as well as computational time comparisons with IDA are presented in Chapter 4.

3.6 Multi Criteria Decision Making (MCDM)

Decision set and objectives: The decision space comprises the generated design alternatives (DA_i).

Four project-level criteria are considered: Peak Mean IDR, Initial Cost, LCCA, and Total Cost. All criteria are framed as to be minimized at the raw level; the multi-criteria procedure converts them to a common benefit scale, derives data-driven weights, and aggregates to a single score for ranking. (Belton & Stewart, 2002; Smith, 1977).

Normalization and Orientation: All criteria were transformed to a common benefit scale so that larger normalized values indicate better performance. Let (x_{ij}) be the raw value of alternative i on criterion j . Because the four raw criteria used here (Peak Mean IDR, Initial Cost, LCCA, Total Cost) are “smaller-is-better”, each was mapped to $[0, 1]$ using an inverted min–max transform over the full set of alternatives (Hwang & Yoon, 1981; Smith, 1977):

$$x'_{ij} = \frac{x_j^{max} - x_{ij}}{x_j^{max} - x_j^{min}}, \quad 0 \leq x'_{ij} \leq 1$$

Under this convention, $x'_{ij} = 1$ denotes the best observed performance and $x'_{ij} = 0$ the worst. The same global bounds (x_j^{max}, x_j^{min}) are used throughout weighting, scoring, and sensitivity to avoid rank reversal. If a future criterion is naturally “larger-is-better”, the standard (non-inverted) min–

max $\frac{x_{ij} - x_j^{min}}{x_j^{max} - x_j^{min}}$ will be used so that the higher-is-better convention remains universal.

Weighting via Shannon entropy: Weights reflect the discriminatory power of each criterion in the normalized matrix $\mathbf{X}' = [\mathbf{x}'_{ij}]$. For criterion j , define proportions $p_{ij} = x'_{ij} / \sum_i x'_{ij}$ and entropy (Hwang & Yoon, 1981; Shannon, 1948; Zeleny, 1982)

$$e_j = -k \sum_i p_{ij} \ln(p_{ij}) \quad k = \frac{1}{\ln m}$$

With m alternatives. The diversification $d_j = 1 - e_j$ measures information content; entropy weights are

$$w_j = \frac{d_j}{\sum_i d_i}, \quad w_j \geq 0, \quad \sum_j w_j = 1$$

This assigns larger weight to criteria that vary more across alternatives (i.e. higher information content), consistent with standard entropy-weighted MCDM practice.

Aggregation model and ranking: Alternatives are scored with a Weighted-Sum Model (WSM) (Hwang & Yoon, 1981; Triantaphyllou, 2000):

$$S_i = \sum_j w_j x'_{ij}$$

And ranked by S_i (descending). This is the most common linear value-model aggregator in the MADM family and is fully compatible with entropy-derived objective weights. In the rare event of ties, precedence is given to lower Total Cost and then to lower Peak Mean IDR. The code-based design (DA_01) is tracked as a baseline in figures and tables but is not privileged in weighting or scoring.

Sensitivity and robustness analysis: Robustness to weight specification is examined using one-at-a-time (OAAT) perturbations. For each criterion j and perturbation level $\delta \in \{5\%, 10\%\}$, the weight is scaled by $(1 \pm \delta)$, all weights are renormalized to sum to one, S_i is recomputed, and the ranking is updated. Two diagnostics are presented: (i) the share of scenarios in which the reported

top-tier set remains unchanged and (ii) the rank trajectories of selected designs across all scenarios **(Belton & Stewart, 2002; Saltelli et al., 2008)**.

Data handling and reproducibility: Computations use the same decision matrix and bounds as in the Results. Missing entries are treated by listwise deletion after normalization; no imputation is applied. Axis truncations used in cost figures are not used in normalization, weighting, or scoring. Scripts for normalization, entropy weighting, WSM scoring, and sensitivity are archived with the dataset to ensure repeatability.

Assumptions and scope: The WSM assumes monotonic preferences and compensability (a deficit in one criterion can be offset by gains in others in proportion to w_j) **(Belton & Stewart, 2002; Smith, 1977)**. Criteria are treated as distinct decision dimensions; while Total Cost is algebraically related to Initial and LCCA, its inclusion is justified as a project-level objective used by stakeholders. The chosen approach is standard within MCDA practice; alternative weighting (e.g., value-based or stakeholder-elicited) can be evaluated within the same framework **(Greco et al., 2016)**.

These criteria were chosen for their direct relevance to value-based design objectives and the availability of robust data. While additional factors such as carbon footprint, construction time, and insurance premium can be highly influential in real-world decision-making, they were not included here due to the absence of reliable project-specific data. Their importance is nevertheless acknowledged, and the integration of such criteria is recommended for future research and practice.

3.7 Resiliency Index

Seismic resilience is defined as the normalized area under the post-event functionality curve $Q(t)$ over a control period T_c (dimensionless $0 \leq R \leq 1$):

$$R = \frac{1}{T_c} \int_0^{T_c} Q(t) dt$$

Here $Q(t)$ is the fraction of usable functionality at time t (1= fully functional, 0= no function).

This “functionality-integral” definition is widely used in the Bruneau-Reinhorn framework and subsequent analytical formulations (**Bruneau et al., 2003; Cimellaro et al., 2010**).

For each hazard level h (e.g., 40%, 30% upto 2% in the site specific ladder), and quantified: (i) an immediate functionality loss $\Delta_h \in [0, 1]$ (mapped from analysis-based damage metrics such as MDI), so $Q(0) = 1 - \Delta_h$; and (ii) a functional recovery time τ_h (days) from performance-based assessment (FEMA P-58), which produces repair time/functional-recovery estimates.

Recovery from $Q(0) = 1 - \Delta_h$ to 1 over τ_h is represented with a smooth half-cosine (trigonometric) ramp – one of several standard parametric forms (linear, exponential, trigonometric) used in analytical resilience studies (**Cimellaro et al., 2010**). For $0 \leq t \leq \tau_h$:

$$Q(t) = (1 - \Delta_h) + \frac{\Delta_h}{2} \left(1 - \cos\left(\frac{\pi t}{\tau_h}\right) \right), \quad Q(t) = 1 \text{ for } t > \tau_h$$

Integrating the above $Q(t)$ (with $\tau_h \leq T_c$) gives the simple and widely used closed form

$$R_h = 1 - \frac{\Delta_h \tau_h}{2T_c}$$

i.e. the average functionality deficit during recovery is $\Delta_h/2$. If recovery extends beyond T_c , the integral is truncated at T_c . This expression follows directly from the functionality-integral definition and the standard recovery function cited above.

CHAPTER 4: ENDURANCE TIME EXCITATION FUNCTION

4.1 Introduction

The accurate simulation of seismic demands is fundamental to modern earthquake engineering and performance-based design. The Endurance Time Method (ETM) has emerged as an efficient technique for evaluating structural response across a spectrum of earthquake intensities within a single analysis. The core of this method is the Endurance Time Excitation Function (ETEF) a synthetic, nonstationary acceleration record with systematically increasing amplitude, enabling a single time history analysis to simulate structural performance at multiple hazard levels.

Compared to traditional incremental dynamic analysis (IDA) with suites of real ground motions, the use of ETEFs significantly reduces computational effort, streamlines parametric studies, and is particularly suited for value-based and resilience-based design. However, the accuracy and reliability of ETEF-based analyses depend on the generation technique and the extent to which the ETEF captures the essential intensity and spectral characteristics of real earthquakes. Recent literature has established several approaches for ETEF generation, including amplitude envelope scaling, stochastic simulation, and spectral matching with the latter increasingly recognized as the standard for producing code-compliant records suitable for advanced nonlinear dynamic analysis (Estekanchi et al., 2023; NBCC 2020).

This chapter details the methodology used for generating and validating ETEFs in this research. Two practical strategies (i) segmented scaling of a combined ETEF and (ii) multiple individually scaled ETEFs are implemented and compared. Their performance is benchmarked against IDA using real ground motions, with discussion of their respective strengths, limitations, and implications for value-based seismic design in Canada.

4.2 Generation of ETEF

In this study, response spectrum matching was adopted as the primary technique for generating Endurance Time Excitation Functions (ETEFs), consistent with best practices in contemporary earthquake engineering. The process began with the selection of base acceleration records, which serve as the seed motions for spectral matching.

The ETA40lc series was selected as the base ETEFs due to their proven accuracy, reliability, and computational efficiency. These records were sourced from the official ETM open-source library provided by Estekanchi et al. (2023). The ETA40lc series is generated using an advanced methodology that minimizes residuals in acceleration spectra, displacement spectra, and cumulative absolute velocity (CAV), thereby ensuring consistency with both the intensity and duration characteristics of real ground motions. Prior research has demonstrated that the ETA40lc records provide structural response predictions closely matching those from incremental dynamic analysis (IDA), with average errors below 10% and at a fraction of the computational cost (Mashayekhi et al., 2018, Tables 5 and 7). Their performance has been validated for both global and local engineering demand parameters, making them particularly well suited for value-based and performance-based seismic assessment. Accordingly, ETA40lc records are widely recommended and adopted in endurance time analysis as robust, CAV-consistent ETEFs.

Spectral matching was then conducted using the SeismoMatch software, utilizing the **(Al Atik & Abrahamson, 2010)** frequency-domain matching algorithm to iteratively modify the seed records so that their response spectra closely matched the prescribed NBCC 2020 target spectra. The target spectra were obtained from the NBCC 2020 seismic hazard tool for the project location (Vancouver, Site Class C), and the matching was performed within a period range of 0.05–4.0 seconds using a mismatch tolerance of 0.3 and up to 30 iterations. These parameters were selected to capture the fundamental and higher-mode responses of the structure, over the period range of

interest. Although many performance-based guidelines emphasize intensity-based scaling of real ground motions to preserve record characteristics, spectrum-compatible motions generated by frequency-domain matching remain widely used in research for efficiently producing code-consistent input motions, particularly when developing and testing analysis frameworks such as ETM-based VBD. In this study, the matched records were visually and quantitatively checked against the target spectra and were used to generate consistent multi-hazard demand histories for comparing design alternatives, recognizing that alternative selection and scaling procedures could slightly shift absolute demand estimates.

This workflow is fully consistent with the spectrum matching methodologies recommended by Estekanchi et al. (2023), who highlight frequency-domain and wavelet-based spectral matching as robust approaches for producing code-compliant ETEFs suitable for advanced nonlinear analysis. The final matched records were visually and quantitatively validated against the target spectra, ensuring their suitability for use in subsequent dynamic analyses.

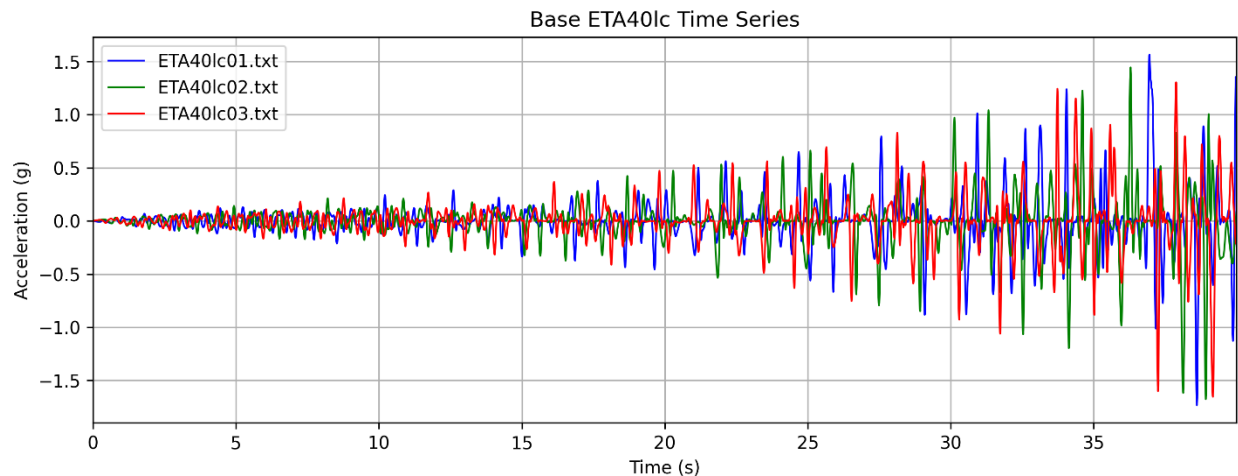


Figure 4.1. Base ETEF acceleration time series (ETA40lc01.txt, ETA40lc02.txt, ETA40lc03.txt)

(Source: Estekanchi et al., 2023, ETM open-source library)

Table 4.1: Site Specific Hazard Spectrum values (Appendix)

Probability (% in 50 years)	Sa(0.05) [g]	Sa(0.1) [g]	Sa(0.2) [g]	Sa(0.3) [g]	Sa(0.5) [g]	Sa(1.0) [g]	Sa(2.0) [g]	Sa(5.0) [g]	Sa(10.0) [g]	PGA [g]	PGV [m/s]
2	0.658	0.954	1.06	1.01	0.758	0.434	0.269	0.074	0.0324	0.452	0.462
5	0.45	0.659	0.736	0.699	0.521	0.29	0.165	0.0411	0.0162	0.316	0.303
7	0.386	0.565	0.636	0.601	0.445	0.245	0.134	0.0326	0.0126	0.273	0.254
10	0.323	0.474	0.535	0.506	0.371	0.201	0.105	0.0251	0.00968	0.23	0.209
14	0.269	0.396	0.449	0.423	0.308	0.164	0.0817	0.0194	0.00739	0.193	0.17
20	0.217	0.321	0.365	0.342	0.247	0.128	0.0613	0.0145	0.00538	0.157	0.134
30	0.164	0.242	0.279	0.259	0.185	0.0937	0.043	0.01	0.00358	0.12	0.0981
40	0.129	0.19	0.222	0.205	0.145	0.0724	0.0324	0.0074 5	0.00258	0.095	0.0761

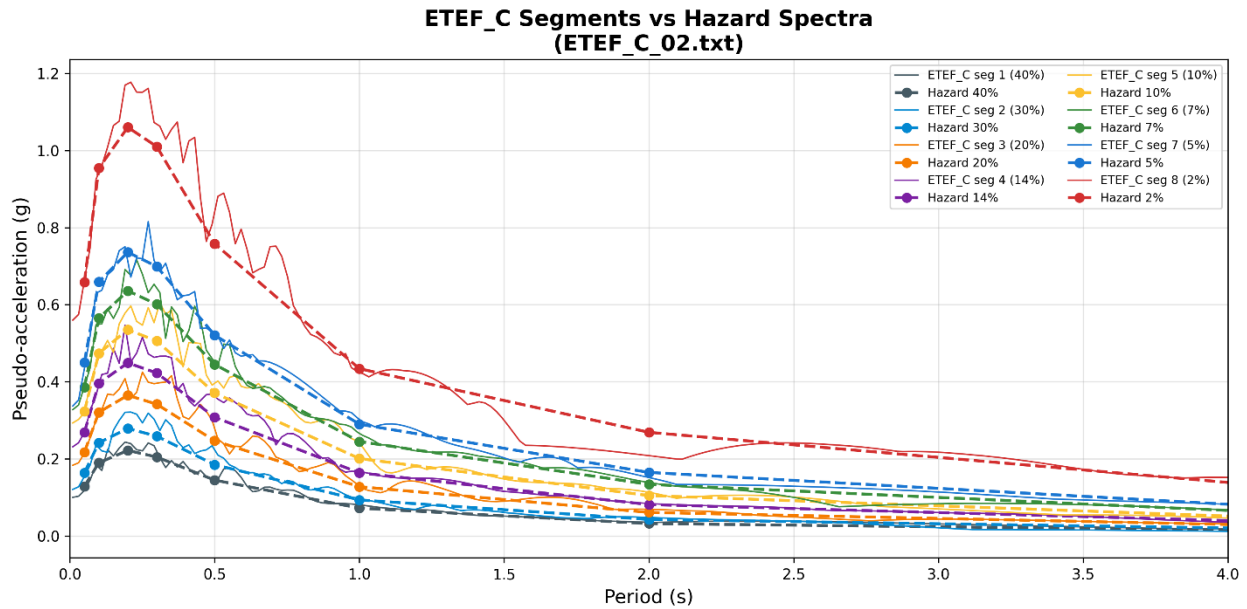
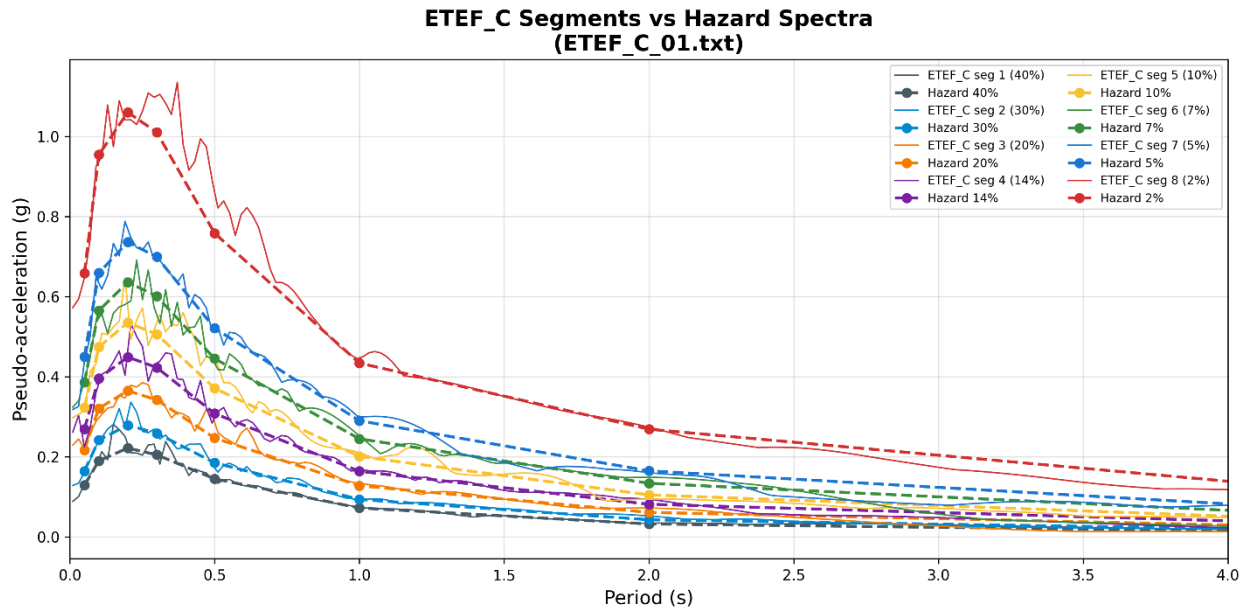
4.3 Spectral Matching Approaches

In order to comprehensively assess the structural response across multiple seismic hazard levels, two distinct spectral matching approaches were implemented in this study. These approaches, while inspired by the flexibility of the endurance time methodology, were adapted specifically for the performance-based and value-based assessment objectives of this research. The first approach, ETEF_C, applies spectral matching in segmented intervals, producing a stepwise accelerogram that captures all hazard levels within a single analysis. The second approach, ETEF_A, applies spectral matching separately to independent records, allowing each hazard level to be assessed without cumulative effects. The following subsections describe these two approaches in detail.

4.3.1 ETEF_C Approach

In the first approach, the base 40-second ETEF was divided into eight consecutive 5-second segments. Each segment (0–5s, 5–10s, 10–15s, 15–20s, 20–25s, 25–30s, 30–35s, and 35–40s) was individually scaled to match the response spectrum corresponding to a specific seismic hazard level (ranging from 40% in 50 years up to 2% in 50 years). The scaled segments were then concatenated to form a single, continuous 40-second accelerogram ETEF_C (Figure 4.3), whose

intensity increases stepwise with time, each step representing a higher hazard level. By applying this combined ETEF to the structure, a single nonlinear dynamic analysis could efficiently produce structural response data for all eight hazard scenarios in sequence, with each 5-second segment reflecting the demands of a specific hazard level.



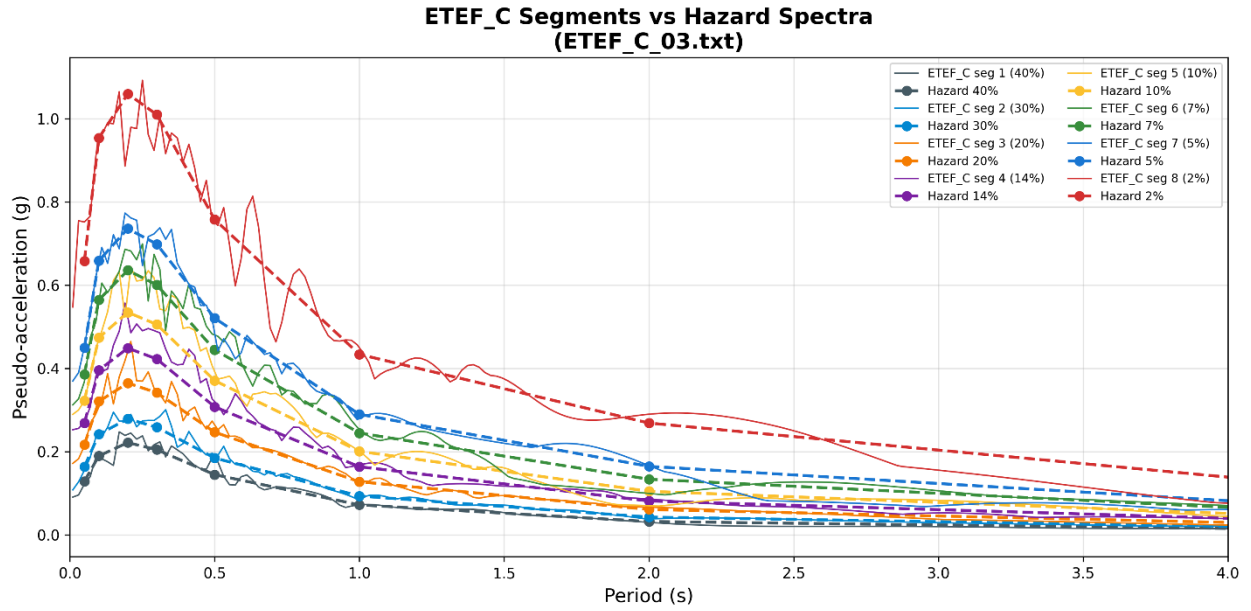


Figure 4.2: Representation of matched response spectrum of ETEF_C (ETEF_C_01.txt, ETEF_C_02.txt, ETEF_C_03.txt)

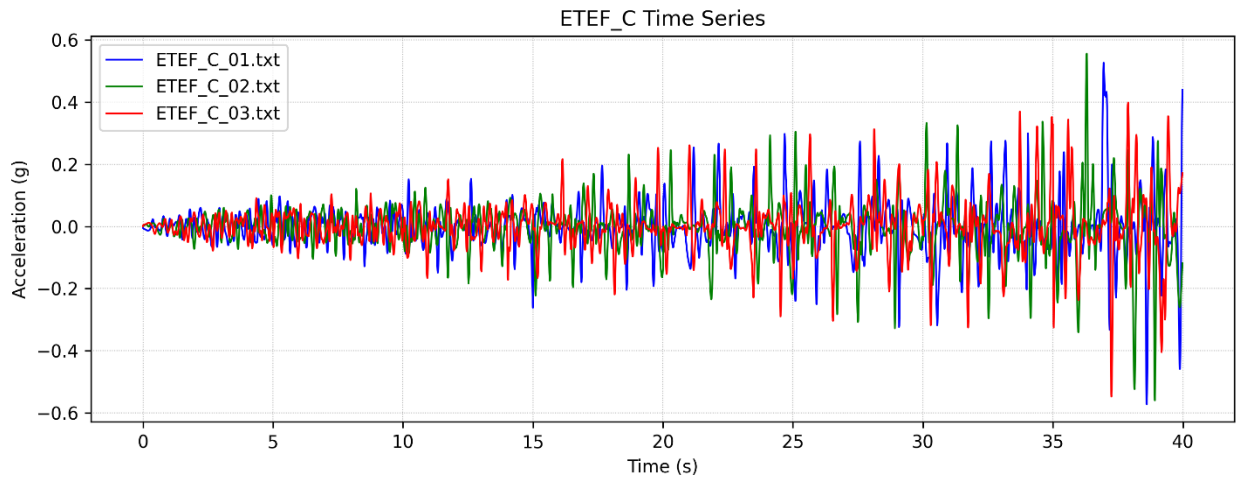


Figure 4.3: ETEF_C acceleration time series (ETEF_C_01.txt, ETEF_C_02.txt, ETEF_C_03.txt)

4.3.2 ETEF_A Approach

The second approach utilized eight separate ETEFs, each representing a distinct seismic hazard level. In this method, the base ETEF profile was used in its full 40-second length for each case but scaled so that its maximum amplitude at $t = 40$ seconds corresponded to the spectral demand of the assigned hazard level. Each scaled ETEF was then applied independently to the structural model, with the structure assumed to be undamaged at the start of each run. This allowed for an independent assessment of the structural response for each hazard level, without cumulative damage effects from lower hazard intensities.

Both approaches were adopted to compare their effectiveness, efficiency, and fidelity in predicting structural performance under the full spectrum of seismic hazard considered in Canadian design practice. The first method (ETEF_C) leverages the efficiency of the endurance time method by simulating multiple hazard scenarios in a single analysis, while the second (ETEF_A) follows a more traditional probabilistic approach by treating each hazard level separately. The subsequent sections present a detailed evaluation of these methods and a comparison with results from incremental dynamic analysis using real ground motions.

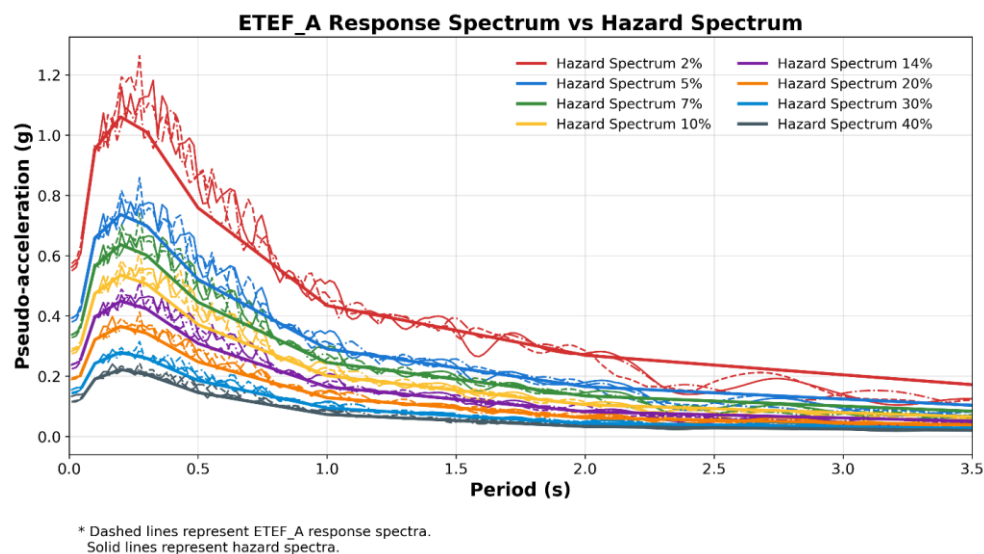
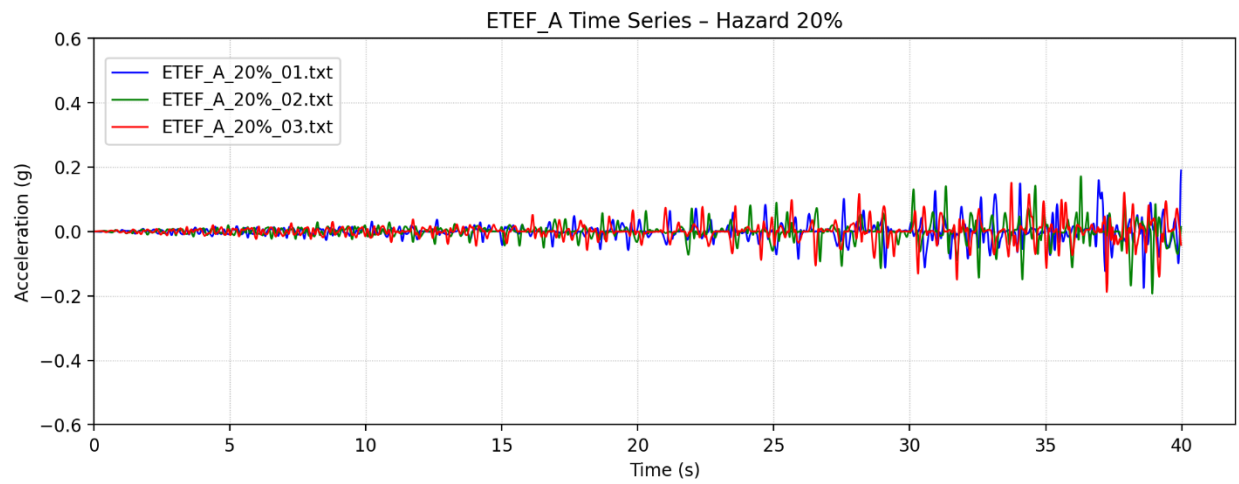
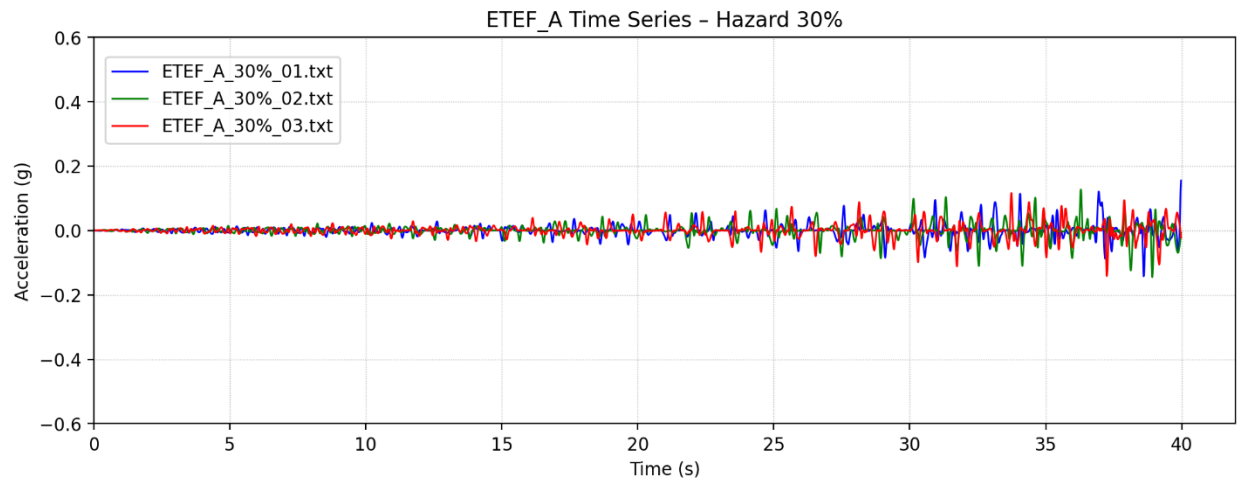
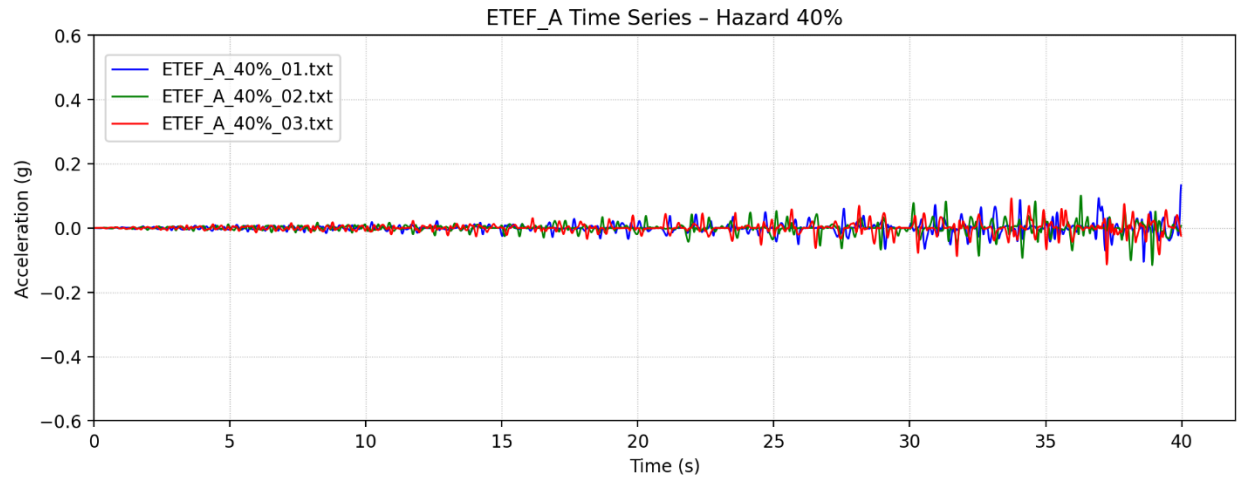
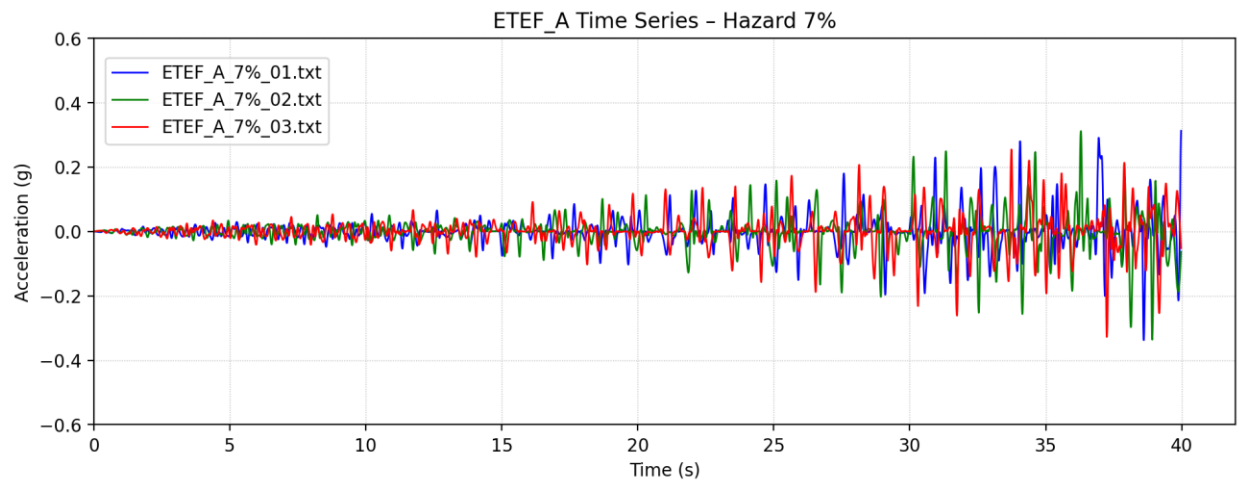
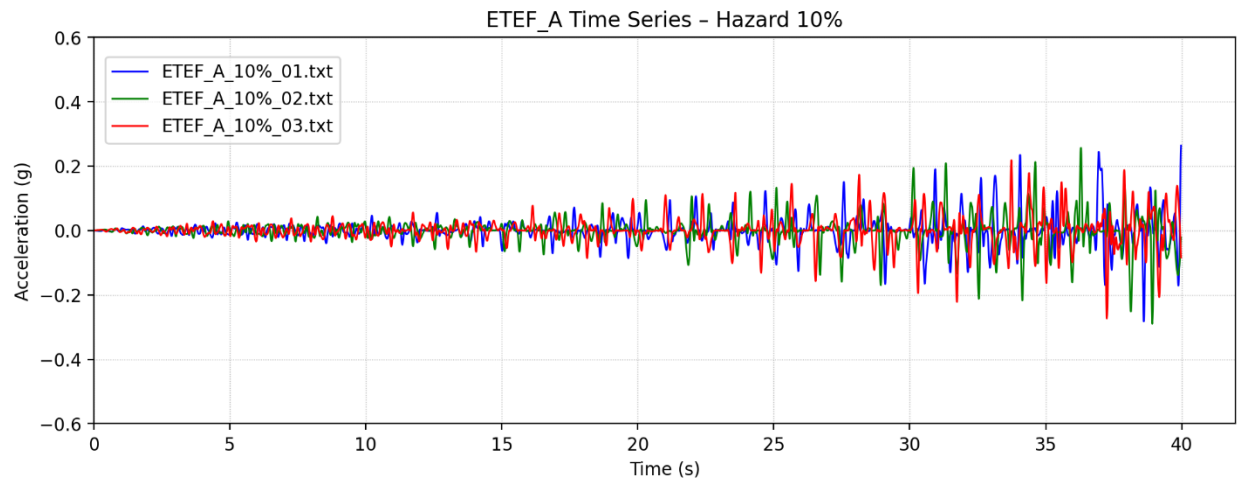
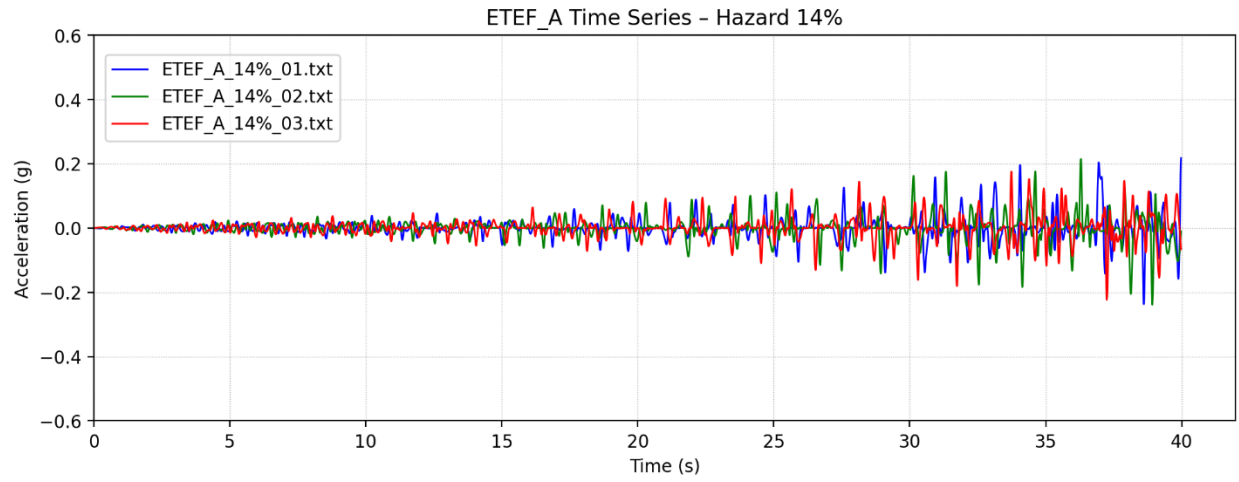


Figure 4.4: Representation of matched response spectrum of ETEF_A (8 hazard scenarios)





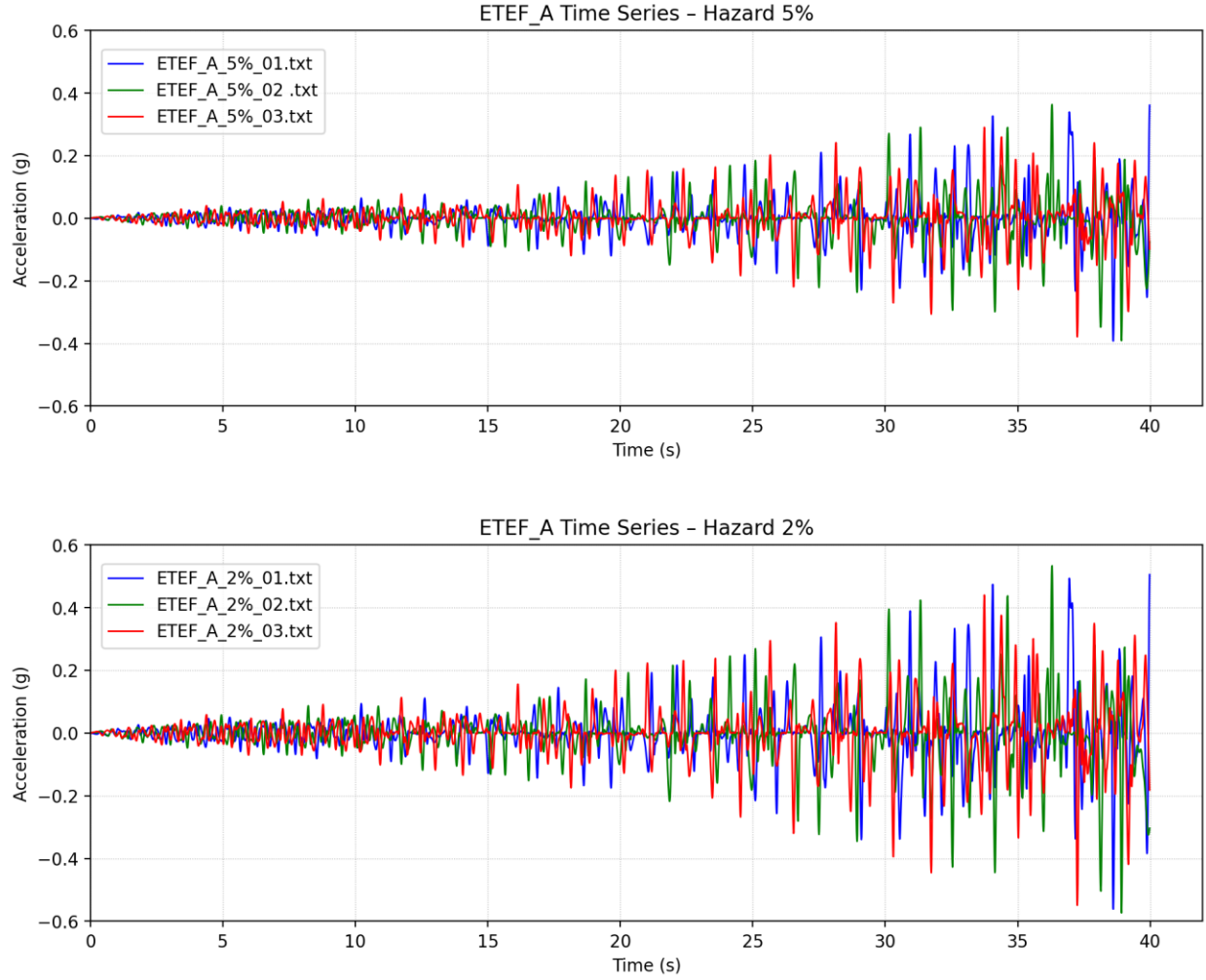


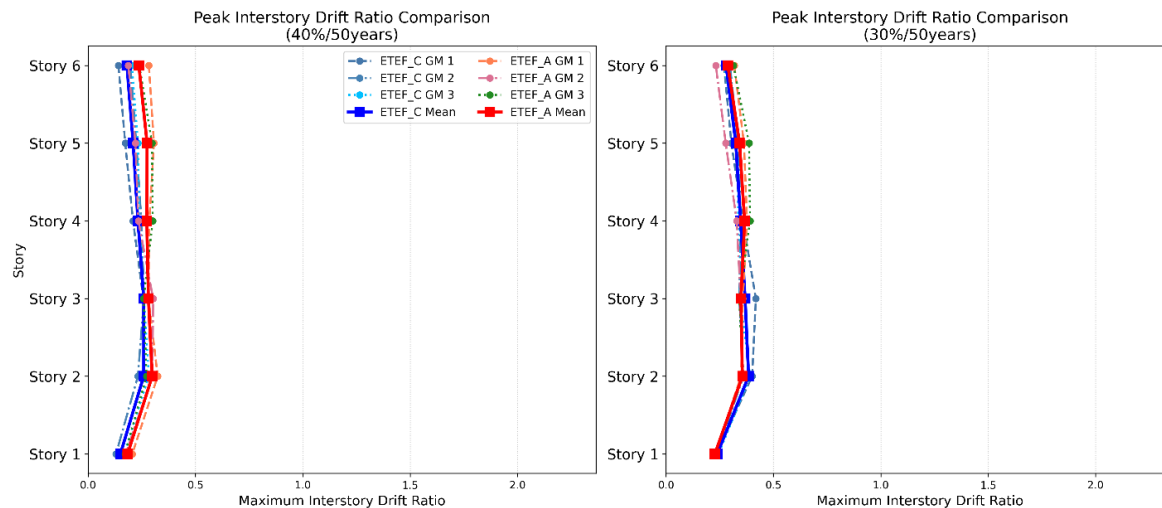
Figure 4.5: ETEF_A acceleration time series (8 ETEF_A each representing individual hazard)

4.3.3 Comparative Response of ETEF_C and ETEF_A

To assess the sensitivity of the Endurance Time Method (ETM) to different implementation strategies, a detailed comparison was conducted between the interstory drift ratio (IDR) responses obtained using ETEF_C (segmented, single-record approach) and ETEF_A (multiple, individually scaled records) across all hazard scenarios. For each hazard level considered (ranging from 40% to 2% probability of exceedance in 50 years), nonlinear time history analyses were performed using three series of each ETEF variant, and the resulting mean IDR values were extracted at each story.

As shown in Figure 4.6, both ETEF_C and ETEF_A yield closely aligned IDR profiles across most hazard scenarios, reflecting the practical equivalence of the two methods for capturing overall structural response trends. However, at higher hazard levels, a nuanced divergence is observed: for the 5% probability scenario, ETEF_C tends to produce slightly lower IDR estimates, suggesting a conservative bias; conversely, at the most extreme (2% in 50 years) hazard level, ETEF_C sometimes predicts marginally higher IDRs than ETEF_A. These shifts may be attributable to the segmentation and scaling methodology inherent to ETEF_C, which can introduce minor discontinuities or variations not present in the individually matched ETEF_A records.

Overall, the comparison demonstrates that both approaches are robust for engineering assessment, with ETEF_A providing marginally greater fidelity in matching hazard-specific response trends, especially at the highest intensities. This finding supports the use of ETEF_A for subsequent value-based design analyses in this study, while also validating the efficiency and reliability of ETEF_C for rapid multi-level hazard assessment.



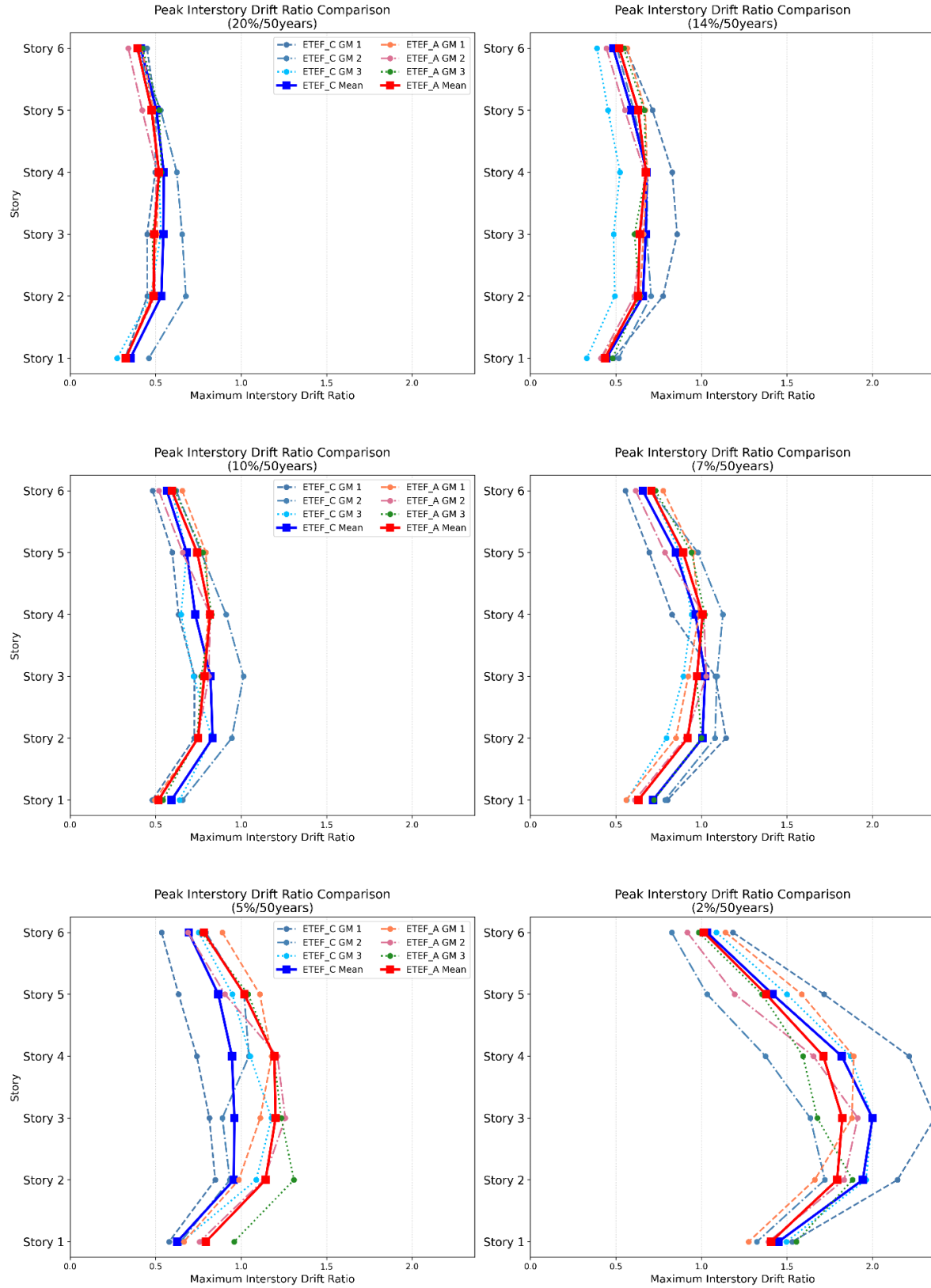


Figure 4.6 Maximum IDR comparisons between ETEF_A and ETEF_C for all hazard scenarios

4.4 Validation of ETEF with Real Ground Motion

The validity of the Endurance Time Method (ETM) for advanced seismic assessment relies on its ability to accurately reproduce structural demands observed under real earthquake ground motions. In this section, a detailed comparison is presented between ETEF-based analyses and traditional Incremental Dynamic Analysis (IDA) performed with a suite of statistically representative real ground motions. The aim is to benchmark the performance, accuracy, and practical utility of the generated ETEFs across a range of seismic hazard levels and engineering demand measures. The following subsections outline the criteria for ground motion selection and scaling, describe the IDA methodology, and present the comparative results and key findings.

4.4.1 Ground Motion Selection and Scaling

To provide a rigorous benchmark for validating the performance of Endurance Time Excitation Functions (ETEFs), a suite of real earthquake ground motions was carefully selected from the PEER NGA-West2 database. The selection process followed established best practices for nonlinear dynamic analysis (e.g., ASCE 7-10, FEMA P695), ensuring statistical representativeness and direct relevance to the seismic hazard at the study site.

The chosen suite emphasizes strong ground-shaking events, with moment magnitudes ranging from 6.5 to 8.5, thereby capturing the range of earthquake scenarios most pertinent to performance-based seismic design. To represent both near-field and far-field effects, records were limited to source-to-site Joyner-Boore distances (R_{jb}) of up to 100 km. Site conditions were addressed by restricting V_{s30} (the average shear-wave velocity in the upper 30 meters) to between 300 and 550 m/s, corresponding to NEHRP site classes C and D thus aligning with the reference conditions for the design spectra.

Special consideration was also given to diversity in record duration (as measured by the 5–95% significant duration) and Arias intensity, ensuring the suite captured the variability in strong-

motion duration and energy content observed in real earthquakes. Fault mechanisms were balanced among reverse, normal, and strike-slip events, providing a comprehensive and realistic basis for nonlinear validation.

Following selection, both horizontal components of each record were extracted and amplitude-scaled in accordance with the ASCE 7-10 protocol. Each record was linearly scaled such that its average spectral acceleration matched the 2% in 50 years uniform hazard target spectrum over the period range of interest (typically $0.2T_1$ – $1.5T_1$). The resulting suite, as illustrated in Figure 4.7, closely matches the target spectrum across the relevant period range and provides a robust, unbiased basis for benchmarking the performance of ETEF-based and traditional IDA analyses.

Table 4.2: Selected Ground Motions Characteristics

Record Sequence Number	Earthquake Name	Year	Magnitude	Mechanism	Rjb (km)	Vs30 (m/sec)	5-95% Duration (sec)	Arias Intensity (m/sec)
15	"Kern County"	1952	7.36	Reverse	38.42	385.43	30.3	0.6
79	"San Fernando"	1971	6.61	Reverse	25.47	415.13	14.5	0.2
164	"Imperial Valley-06"	1979	6.53	strike slip	15.19	471.53	36.4	1.3
284	"Irpinia_ Italy-01"	1980	6.9	Normal	9.52	476.62	19.3	0.1
292	"Irpinia_ Italy-01"	1980	6.9	Normal	6.78	382	15.2	1.4
313	"Corinth_ Greece"	1981	6.6	Normal Oblique	10.27	361.4	15.4	0.9
746	"Loma Prieta"	1989	6.93	Reverse Oblique	53.46	391.01	17.8	0.1
749	"Loma Prieta"	1989	6.93	Reverse Oblique	78.32	512.27	14.6	0
832	"Landers"	1992	7.28	strike slip	69.21	382.93	28.5	0.8
983	"Northridge-01"	1994	6.69	Reverse	0	525.79	6.9	6.5
1027	"Northridge-01"	1994	6.69	Reverse	36.86	499.31	12.4	0.1
1148	"Kocaeli_ Turkey"	1999	7.51	strike slip	10.56	523	11.1	0.3
1184	"Chi-Chi_ Taiwan"	1999	7.62	Reverse Oblique	19.93	538.69	29.8	0.7
1190	"Chi-Chi_ Taiwan"	1999	7.62	Reverse Oblique	49.98	497.53	39.1	0.1

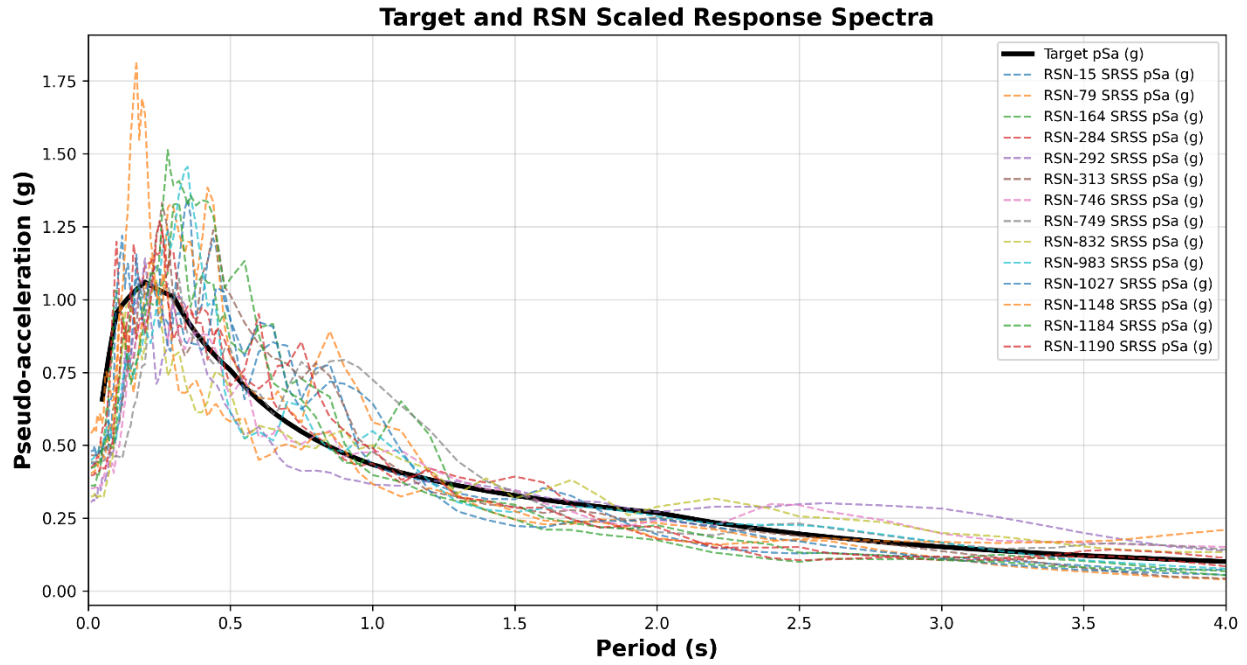


Figure 4.7 Response spectrum comparisons between scaled GMs and Target Spectrum

4.4.2 Incremental Dynamic Analysis (IDA)

Incremental Dynamic Analysis (IDA) was performed to establish a robust benchmark for validating the performance of Endurance Time Excitation Functions (ETEFs). For each selected ground motion, both horizontal components were applied in turn, and the structural model was subjected to a series of nonlinear time history analyses with systematically increasing intensity. The ground motions were amplitude-scaled in increments of 0.1g, starting from the original record and progressing up to a maximum scale factor of 4.0g, or until global structural collapse was observed. This IDA procedure provided the reference median drift trends used to benchmark the accuracy of the ETEF-based analyses.

At each intensity level, peak engineering demand parameters, such as maximum interstory drift ratio (IDR) and spectral acceleration at the fundamental period ($S_a(T_1)$), were recorded. This

process generated a suite of IDA curves for each ground motion, enabling the quantification of structural response across a wide range of seismic demand levels and providing a direct basis for comparison with the ETEF-based analyses. The use of fine scaling increments and a high upper bound on intensity ensured that both elastic and inelastic (collapse) behavior could be thoroughly assessed for all ground motions in the suite.

4.5 Result & Discussion

The comparative validation of the two ETEF analysis strategies, ETEF_C (single combined record with segmented scaling) and ETEF_A (multiple individually scaled ETEFs) highlights both the flexibility and practical value of the Endurance Time Method (ETM) for advanced seismic performance assessment.

Figure 4.8 presents Incremental Dynamic Analysis IDA curves derived from the real ground motions alongside the mean curves obtained using ETEF_C and ETEF_A. Across the intensity range, the ETEF_A mean curve tracks the median IDA response with good agreement in both elastic and moderately inelastic regimes, while some divergence appears at the highest drift levels where record-to-record variability and numerical nonlinearity are most pronounced. ETEF_C shows a similar overall trend but with slightly larger deviations from the IDA median, particularly near the upper end of the drift range, reflecting the additional uncertainty introduced by the segmented compilation strategy.

In this study, ETEF_C required only ~1 minute per series, ETEF_A about 7 minutes, and IDA approximately 385 minutes for a comparable suite of analyses demonstrating the exceptional efficiency of the ETM framework.

Although the original philosophy of ETM is to use a single intensifying record for multi-level assessment (as in ETEF_C), the multi-run ETEF_A approach was adopted here to facilitate direct

benchmarking against IDA and to robustly evaluate each hazard level independently. Three series for each ETEF variant (consistent with ETA40lc family practice) were used to capture record-to-record variability, further strengthening the statistical reliability of the results.

A quantitative assessment of accuracy further supports these findings. The standard deviation of ETEF_A Mean relative to the real ground motion median curve (at 2%/50-year hazard) is 0.0401, compared to 0.0655 for ETEF_C Mean (see Table 4.3). This smaller standard deviation demonstrates that ETEF_A not only reproduces the central tendency (median) of real ground motion response, but does so with much less scatter, indicating higher consistency and reliability across the analyzed stories and hazard levels.

The high level of accuracy achieved by the ETEF_C approach is also consistent with previous research. For example, Estekanchi et al. (2023) demonstrated that spectral matching-based ETEFs achieve nearly the same accuracy within 3% as those generated with computationally intensive nonlinear optimization methods, while requiring only a fraction of the analysis time. This finding, also supported by Mashayekhi et al. (2018), underscores the robustness and practicality of spectral matching for generating code-compliant endurance time records.

Importantly, the minor conservative bias observed in ETEF_C results may not be inherent to the spectral matching algorithm itself but could instead be attributed to the methodology used for compiling multiple hazard spectra into a single segmented ETEF. The stepwise segmentation and concatenation where each 5-second segment is scaled to a different target spectrum may introduce artificial transitions or discontinuities that are not present in natural ground motions or in the ETEF_A approach. Such effects could subtly influence the nonlinear structural response, partially accounting for the observed differences. Future research should further investigate alternative

segmentation techniques or explore smoother transitions between hazard levels to improve the fidelity of multi-level ETEF records.

In summary, both ETEF_A and ETEF_C provide highly efficient and credible alternatives to IDA for value-based and performance-based seismic design. ETEF_A delivers near-equivalent accuracy to real ground motion ensembles for median structural demand, with notably lower standard deviation (i.e., greater statistical reliability), while ETEF_C maximizes analysis efficiency with only minor conservatism which is most likely attributable to the segmentation and compilation strategy rather than any fundamental limitation of spectral matching. These findings reinforce the robustness, adaptability, and practical utility of the ETM for advanced dynamic structural analysis.

On the basis of these findings and given the satisfactory validation of ETEF_A against real ground motion responses for the 2% in 50 years scenario, the remainder of this study adopts ETEF_A as the preferred approach for seismic design analyses. This selection is motivated by ETEF_A's demonstrated accuracy, statistical consistency, and efficiency in representing code-level seismic demands.

In this study, therefore, IDA serves as a benchmark method for validating the calibrated ETEFs, while ETM is used as a computationally efficient method for evaluating multiple design alternatives rather than as a universal replacement for IDA in seismic design practice.

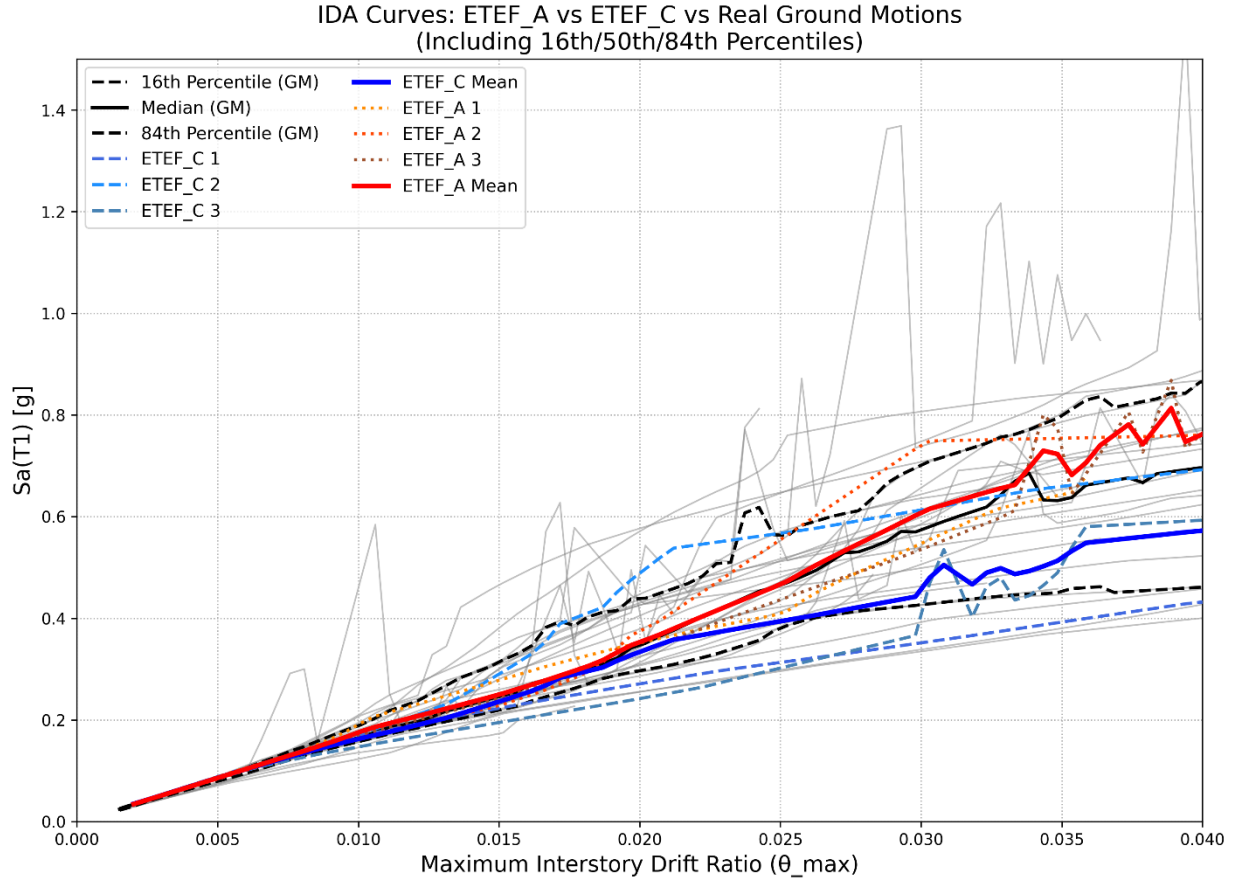


Figure 4.8 Incremental Dynamic Analysis (IDA) curves for real ground motions, ETEF_C, and ETEF_A.

Table 4.3: Standard deviation of ETEF mean vs. real ground motion median IDA curves, 2%/50-year hazard.

ETEF	Standard Deviation
ETEF_A_Mean	0.0401
ETEF_C_Mean	0.0655

Limitations

Unlike the canonical endurance time method (ETM) introduced by Estekanchi et al. (2007, 2011, 2023), which develops a distinct endurance time excitation function (ETEF) for each target hazard spectrum, this study implemented a simplified merging strategy. ETEF_C was generated by

sequentially concatenating 5-second segments matched to different hazard levels, thereby combining the entire hazard curve into a single 40-second record.

While this approach offered efficiency and a unified representation of the hazard levels, it also introduced important limitations. First, the short 5-second segments are insufficient to represent the cumulative duration and energy release associated with real earthquakes at each hazard intensity. Second, the abrupt transitions between hazard levels may create artificial discontinuities not present in natural ground motions. These factors likely contributed to the conservative bias and higher scatter observed in ETEF_C relative to IDA benchmarks.

Future work should therefore investigate hazard-consistent durations (longer strong-motion windows for higher hazards) and adopt smoother blending or wavelet-domain transitions when combining multiple intensities. Such refinements would enhance the physical realism of merged ETEFs while retaining their efficiency advantages.

CHAPTER 5: RESULTS & DISCUSSION

5.1 Introduction

This chapter presents the results of nonlinear time-history analyses for the 6-story RC frame designed to NBCC 2020. All analyses use the validated ETEF_A input set; validation and modeling details are provided in Chapter 4. Results are reported for the full set of design alternatives and contrasted against the code-based baseline (DA_01). The main response metrics are peak interstorey drift ratio (IDR) and derived damage state, followed by cost metrics (Initial, LCCA, Total). We then synthesize outcomes using entropy-weighted MCDM (WSM) and examine weight sensitivity ($\pm 5/\pm 10\%$). Finally, we evaluate functionality and resilience using $Q(t)$ recovery curves and the normalized area under $Q(t)$.

5.2 Validation of ETEF

The Endurance Time Excitation Functions (ETEFs) used here were validated against real ground-motion IDA and against an alternative compiled ETEF strategy. The ETEF_A approach reproduced median GM response with much lower scatter than the combined, segmented ETEF_C (e.g., std. dev. vs GM-median ≈ 0.0401 for ETEF_A vs 0.0655 for ETEF_C), and analyses showed a large runtime advantage over full IDA. Accordingly, ETEF_A is adopted for all results, while ETEF_C is referenced only for sensitivity. See Chapter 4 for full validation, spectra matching details, and comparative plots.

5.3 Dynamic Response

The first three modal periods and cumulative mass participation factors of each design alternative are summarized in Table 5.1, showing that all frames have similar dynamic characteristics and that more heavily reinforced, larger-column designs produce modest reductions in fundamental period while maintaining high mass participation in the first mode. The dynamic response of the 42 design

alternatives is then evaluated in terms of Peak Mean interstorey drift, base-shear ratio V/W , and residual drift demands derived from the ETEF_A analyses across the eight NBCC-consistent hazard levels. These response measures provide the link between structural behavior, damage states, and the loss and resilience metrics discussed in subsequent sections.

Figure 5.1 shows the Peak Mean IDR for all 42 design alternatives, grouped by column cross-section (500×500 mm, 500×550 mm, 550×550 mm, 550×600 mm, and 600×600 mm), with the code-based baseline DA_01 highlighted in black. Peak Mean IDR is defined as the maximum, over all stories and hazard levels, of the mean interstorey drift ratio obtained from the ETEF_A analyses and is evaluated consistently with the drift-based damage-state thresholds in Table 3.5. The dashed horizontal line at 1.8 % represents the Heavy–Major damage threshold; the code-based design DA_01 lies slightly above this limit at approximately 1.81 %, and most alternatives in the 500×500 mm and 500×550 mm groups cluster close to this threshold, whereas almost all 550×600 mm and 600×600 mm designs remain comfortably below about 1.7 %. Within each column-size group, increasing the reinforcement ratio generally reduces Peak Mean IDR but not in a strictly monotonic manner; small variations reflect differences in yielding patterns and record-to-record variability rather than a reversal of the overall trend that larger, better-reinforced columns lead to lower drift demands. Because each hazard level is represented by only three ETEF_A records whose responses are averaged to obtain mean IDR, some scatter between neighbouring reinforcement levels is expected and should be interpreted as variability around the underlying size-driven drift reduction, not as distinct design regimes.

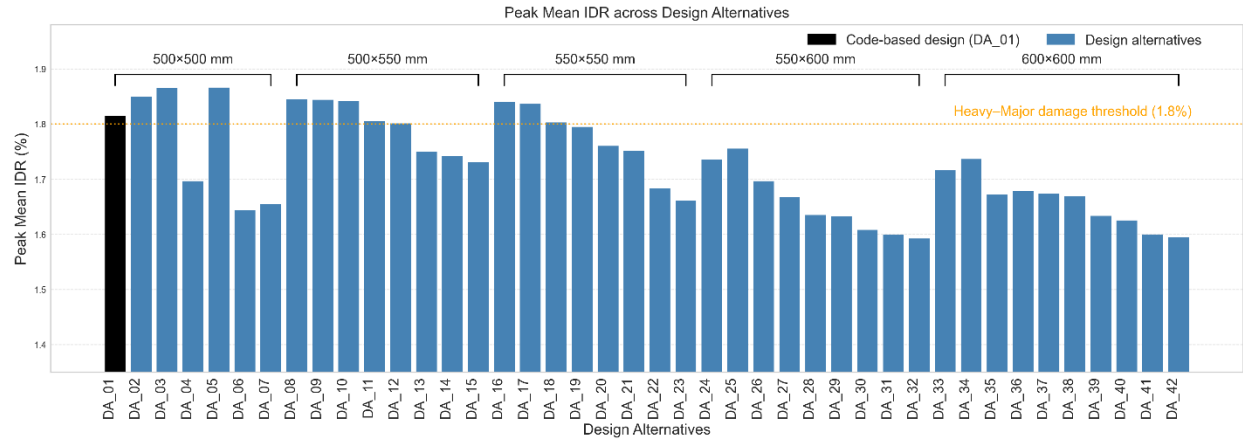


Figure 5.1: Peak Mean IDR of all design alternatives

Figure 5.2 presents the maximum base-shear ratio V/W for all design alternatives at the 2 % in 50-year hazard level, where V is the peak total base shear from the ETEF_A analyses and W is the NBCC-defined seismic weight of the building. Across the set, V/W ranges from roughly 15–17 % for the weaker, small-column designs to about 22–23 % for the largest and most heavily reinforced sections, showing a clear increase in lateral strength as column dimensions and reinforcement grow. The code-based design DA_01 lies near the lower bound of this range, consistent with its comparatively high Peak Mean IDR, whereas mid-range value-based designs achieve modestly higher V/W without pushing strength to the maximum, indicating that only a portion of the available overstrength is actually needed to bring drifts below the Heavy–Major threshold.

Table 5.1: Dynamic Properties (First 3 modes period and Cumulative Mass) of all DAs

DA	T1(s)	CM1(%)	T2(s)	CM2(%)	T3(s)	CM3(%)
DA_01	1.719	78.838	0.491	90.530	0.233	95.611
DA_02	1.706	78.640	0.485	90.441	0.228	95.572
DA_03	1.703	78.604	0.484	90.424	0.228	95.564
DA_04	1.691	78.418	0.478	90.342	0.224	95.530
DA_05	1.688	78.376	0.477	90.324	0.223	95.521
DA_06	1.676	78.201	0.471	90.250	0.219	95.491
DA_07	1.673	78.159	0.470	90.232	0.218	95.483
DA_08	1.668	77.925	0.465	90.150	0.214	95.458
DA_09	1.655	77.730	0.459	90.074	0.210	95.428
DA_10	1.652	77.694	0.458	90.059	0.209	95.421
DA_11	1.640	77.510	0.453	89.990	0.205	95.395
DA_12	1.637	77.468	0.451	89.974	0.204	95.388
DA_13	1.626	77.297	0.446	89.912	0.201	95.365
DA_14	1.623	77.254	0.445	89.897	0.200	95.359
DA_15	1.620	77.212	0.443	89.881	0.199	95.353
DA_16	1.636	77.523	0.452	89.999	0.205	95.400
DA_17	1.634	77.490	0.451	89.985	0.204	95.395
DA_18	1.623	77.321	0.445	89.924	0.201	95.372
DA_19	1.620	77.283	0.444	89.910	0.200	95.366
DA_20	1.610	77.122	0.439	89.854	0.197	95.346
DA_21	1.607	77.083	0.438	89.841	0.196	95.341
DA_22	1.597	76.930	0.434	89.790	0.193	95.323
DA_23	1.595	76.891	0.432	89.777	0.192	95.318
DA_24	1.590	76.658	0.428	89.714	0.188	95.304
DA_25	1.588	76.631	0.427	89.705	0.188	95.301
DA_26	1.577	76.465	0.422	89.654	0.185	95.284
DA_27	1.575	76.427	0.421	89.642	0.184	95.280
DA_28	1.565	76.270	0.416	89.597	0.181	95.265
DA_29	1.562	76.231	0.415	89.585	0.180	95.261
DA_30	1.552	76.083	0.410	89.544	0.177	95.249
DA_31	1.550	76.045	0.409	89.533	0.177	95.245
DA_32	1.547	76.007	0.408	89.522	0.176	95.242
DA_33	1.573	76.467	0.421	89.659	0.184	95.288
DA_34	1.572	76.442	0.420	89.651	0.184	95.285
DA_35	1.562	76.288	0.415	89.606	0.181	95.271
DA_36	1.559	76.252	0.414	89.595	0.180	95.267
DA_37	1.550	76.106	0.410	89.554	0.177	95.255
DA_38	1.548	76.070	0.409	89.544	0.177	95.251
DA_39	1.538	75.931	0.405	89.506	0.174	95.241
DA_40	1.536	75.896	0.404	89.497	0.173	95.238
DA_41	1.527	75.764	0.400	89.463	0.171	95.228
DA_42	1.525	75.730	0.398	89.454	0.170	95.226

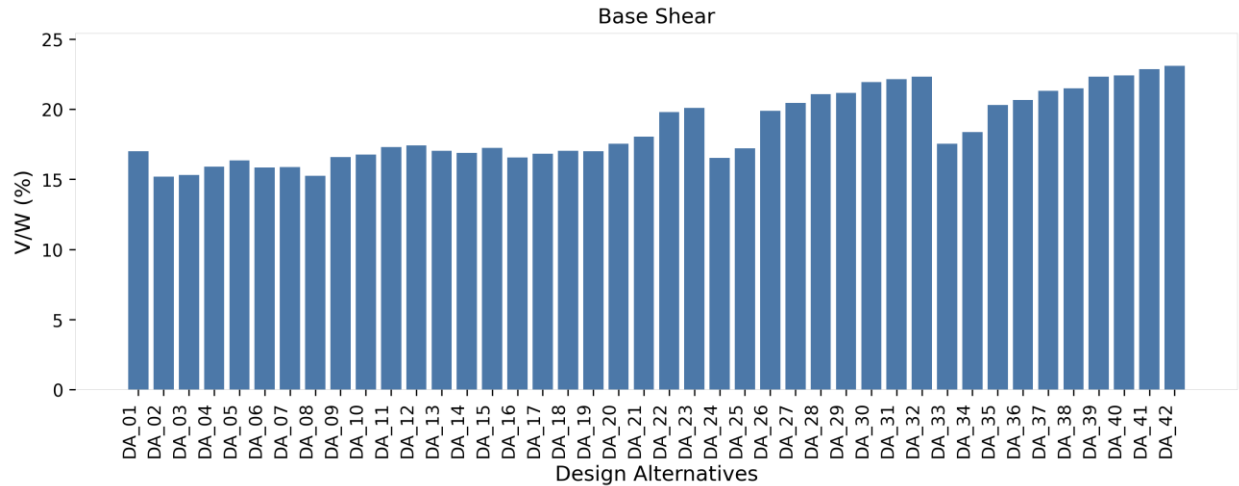


Figure 5.2: Base-shear ratio V/W for all design alternatives

Residual drift demands for all design alternatives are summarized in Figure 5.3 using the Peak Mean residual interstorey drift ratio is enveloped over the eight hazard levels. Peak Mean residual drift is defined, consistent with Tesfamariam and Goda (2015) and FEMA P-58, as the story drift ratio remaining at the end of the ETM time history, averaged over the three ETEF_A records per hazard and then enveloped over the six stories and all hazards for each design. The resulting values range between approximately 0.40 % and 0.68 %, with the code-based baseline DA_01 and several small-section designs in the 500×500 mm and 500×550 mm groups clustering near the upper end of this band, and designs with 550×550 mm and larger columns tending toward the lower end around 0.40–0.45 %.

When compared to the residual-drift performance limits compiled by Tesfamariam and Goda (2015) 0.2 % for Immediate Occupancy, 0.5 % for Damage Control, and 1.0 % for Life Safety these results indicate that all alternatives remain below the 1.0 % Life Safety threshold, but several small-section designs, including DA_01, approach or slightly exceed the 0.5 % Damage Control limit associated with costly realignment and potentially uneconomical repair. In contrast, most alternatives with 550×550 mm, 550×600 mm, and 600×600 mm columns keep residual drifts close

to or below 0.5 %, suggesting a higher likelihood of reparability and shorter functional recovery, which is reflected later in their higher Resiliency Index values in Section 5.6.

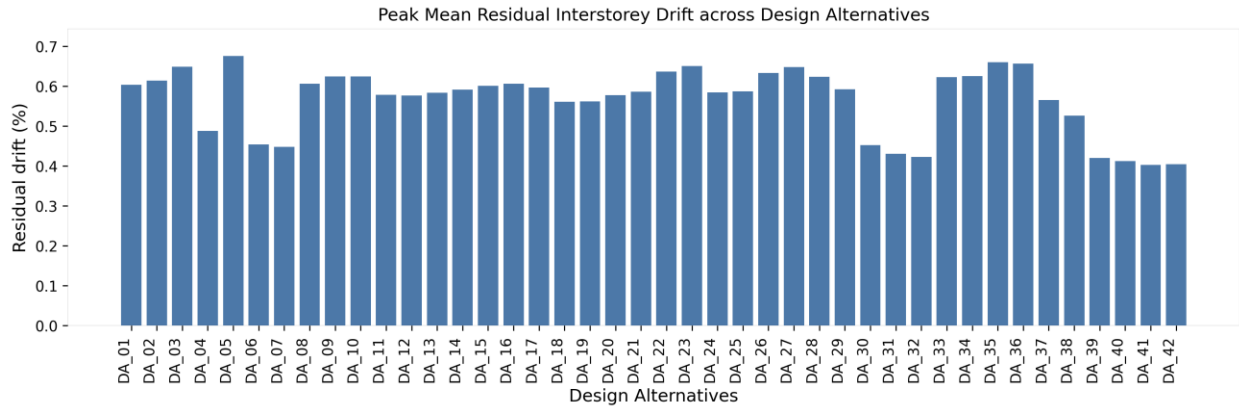


Figure 5.3: Residual Drift for all design alternatives

5.4 Life Cycle Cost Analysis results

The life-cycle cost analysis defines Total Cost as the sum of Initial Cost and LCC, where LCC aggregates damage repair, contents loss, rental loss, relocation, and casualty losses across the eight NBCC-consistent hazard levels using the EAL procedure in Section 3.3.3.2. Figure 5.2 presents Initial Cost and LCC for all design alternatives as overlaid bar charts ordered by Initial Cost from left to right, while the upper panel shows Total Cost as a line plot and the lower panel superimposes Peak Mean IDR on a secondary axis to link economic consequences with drift demand.

Initial Cost increases almost linearly from about 4,085 kCAD for the code-based design DA_01 to approximately 4,940 kCAD for DA_42, reflecting progressive enlargement of column sections and reinforcement. LCC, by contrast, varies between roughly 1,670 kCAD and 2,397 kCAD, driven primarily by the drift-dependent damage and casualty components described in Section 5.3, so that Total Cost spans a wider range from about 5,963 kCAD (DA_33) up to 6,853 kCAD

(DA_05). Designs with higher Peak Mean IDR and residual drift mainly the lightly reinforced 500×500 mm and 500×550 mm alternatives tend to sit toward the upper end of the LCC band, whereas designs that keep Peak Mean IDR below the Heavy–Major damage threshold and residual drift near or below the 0.5 % Damage-Control limit achieve noticeably lower LCC despite moderately higher Initial Cost.

The code-based design DA_01 illustrates the trade-off clearly: it has the lowest Initial Cost (4,085 kCAD) but also one of the highest LCCA values (2,058 kCAD), resulting in a Total Cost of 6,143 kCAD, which is exceeded by several more heavily reinforced value-based alternatives that control drift more effectively. At the other extreme, very stiff and strong designs such as DA_05 and DA_42 achieve some reduction in LCC relative to DA_01 but raise Initial Cost so much that their Total Costs reach 6,853 kCAD and 6,610 kCAD respectively, confirming that aggressive strengthening produces diminishing economic returns. The lowest Total Cost in the set is obtained by DA_33 (≈5,963 kCAD), which lies in a mid-range of Initial Cost (≈4,179 kCAD) while maintaining relatively low LCC through improved drift performance, with several nearby designs (DA_24–DA_26 and DA_34–DA_38) delivering comparable Total Costs within a narrow band. Casualty losses highlight the life-safety implications of these trends. For DA_01, the expected fatality cost is about 643 kCAD over the 50-year service life, computed using FEMA P-58 unit fatality cost and an assumed occupancy of 302 persons integrated over all hazard levels, whereas mid-range value-based designs with lower Peak Mean IDR and residual drift reduce this component to roughly 400–460 kCAD. This reduction reflects the shift away from residual-drift levels approaching the 1.0 % Life-Safety limit summarized by Tesfamariam and Goda (2015) toward the Damage-Control range around 0.5 %, where realignment remains feasible and demolition is less likely. Consequently, modest increases in Initial Cost not only lower Total Cost

but also decrease implicit casualty risk, indicating that the NBCC 2020 minimum-compliance design is economically and socially inefficient when evaluated over the full life cycle.

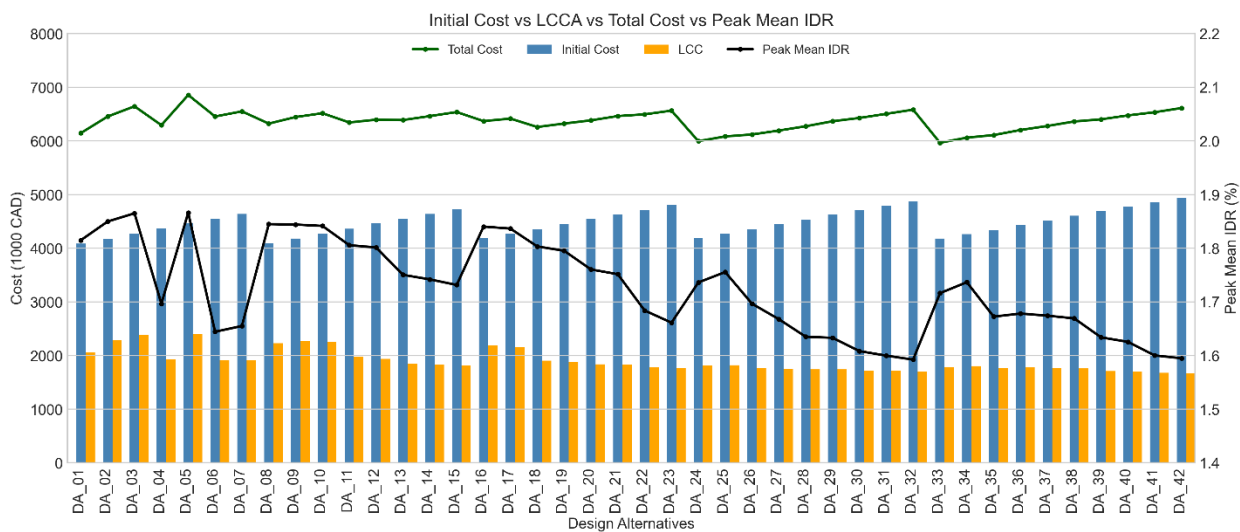


Figure 5.4: Comparison plot of Initial Cost with LCCA vs Peak Mean IDR

Overall, Figure 5.4 and Table 5.2 show that neither the cheapest nor the most heavily reinforced designs are economically optimal; the most efficient alternatives occupy a mid-range of Initial Cost where Peak Mean IDR and residual drift are sufficiently controlled to keep LCC near the lower end of its range without incurring excessive construction cost. Within this band, differences in Total Cost are dominated by relatively small variations in Initial Cost, underscoring that value-based designs which judiciously increase strength and stiffness to manage drift outperform the code-based baseline over the buildings life cycle.

Table 5.2: Cost Analysis Result of all DAs

DA	C_{IN}	C_{dam}	C_{con}	C_{rent}	C_{reloc}	$C_{inj,m}$	$C_{inj,s}$	C_{fat}	C_{LC}	C_{TOT}
DA_01	4085.4	936.1	352.4	83.0	37.3	2.5	3.6	642.8	2057.7	6143.1
DA_02	4170.0	942.0	354.6	84.7	38.0	2.6	4.2	855.9	2282.0	6452.0
DA_03	4266.8	943.2	355.1	85.1	38.2	2.6	4.5	946.6	2375.4	6642.3
DA_04	4363.7	922.5	347.3	83.4	37.4	2.4	3.2	530.8	1927.0	6290.7
DA_05	4456.4	945.9	356.1	86.2	38.7	2.6	4.6	962.7	2396.9	6853.4
DA_06	4545.1	919.2	346.1	82.1	36.9	2.3	3.1	517.7	1907.3	6452.4
DA_07	4633.8	921.5	346.9	82.0	36.8	2.4	3.1	521.2	1913.9	6547.7
DA_08	4094.0	954.5	359.3	83.7	1.0	2.6	4.1	820.8	2226.1	6320.1
DA_09	4177.3	959.1	361.1	81.9	36.8	2.6	4.1	819.5	2265.0	6442.3
DA_10	4266.6	958.5	360.9	81.5	36.6	2.6	4.1	802.5	2246.5	6513.1
DA_11	4361.8	935.1	352.0	77.6	34.8	2.5	3.4	574.1	1979.4	6341.2
DA_12	4457.0	926.1	348.6	76.6	34.4	2.4	3.3	544.7	1936.1	6393.1
DA_13	4546.3	895.9	337.3	73.0	32.8	2.2	3.0	496.1	1840.2	6386.5
DA_14	4633.5	892.3	335.9	72.4	32.5	2.2	2.9	487.9	1826.2	6459.7
DA_15	4720.8	889.4	334.8	71.9	32.3	2.2	2.9	481.0	1814.4	6535.3
DA_16	4181.8	936.0	352.4	77.9	35.0	2.5	3.9	776.9	2184.6	6366.4
DA_17	4263.8	929.2	349.8	77.2	34.7	2.5	3.9	752.3	2149.4	6413.2
DA_18	4356.6	906.8	341.4	74.1	33.3	2.4	3.2	538.9	1899.9	6256.5
DA_19	4450.0	903.8	340.3	73.6	33.0	2.3	3.1	514.7	1870.9	6320.9
DA_20	4543.3	896.3	337.4	72.1	32.4	2.2	3.0	493.8	1837.2	6380.5
DA_21	4629.7	895.4	337.1	71.9	32.3	2.2	2.9	488.9	1830.8	6460.5
DA_22	4715.5	883.8	332.7	70.1	31.5	2.1	2.7	453.8	1776.7	6492.1
DA_23	4801.2	880.7	331.6	69.6	31.3	2.0	2.7	442.9	1760.8	6562.0
DA_24	4182.5	892.8	336.1	70.6	31.7	2.1	2.9	473.6	1809.8	5992.4
DA_25	4263.2	893.3	336.3	70.8	31.8	2.2	2.9	480.5	1817.9	6081.1
DA_26	4348.2	882.3	332.2	69.4	31.2	2.0	2.7	449.1	1769.0	6117.1
DA_27	4439.7	879.3	331.0	68.9	30.9	2.0	2.6	436.3	1751.0	6190.7
DA_28	4531.3	884.1	332.9	68.3	30.7	1.9	2.5	418.8	1739.2	6270.4
DA_29	4622.1	887.1	334.0	68.3	30.7	1.9	2.5	417.2	1741.6	6363.7
DA_30	4706.3	882.1	332.1	67.3	30.2	1.8	2.4	402.9	1718.9	6425.2
DA_31	4790.6	880.1	331.3	67.0	30.1	1.8	2.4	398.5	1711.3	6501.9
DA_32	4874.9	878.5	330.7	66.8	30.0	1.8	2.4	394.5	1704.7	6579.6
DA_33	4179.1	887.8	334.2	69.6	31.3	2.1	2.7	456.3	1784.0	5963.0
DA_34	4258.2	892.7	336.1	70.1	31.5	2.1	2.8	464.6	1799.9	6058.1
DA_35	4337.3	893.9	336.5	69.0	31.0	2.0	2.6	433.9	1769.0	6106.3
DA_36	4424.8	897.8	338.0	69.1	31.0	2.0	2.6	435.6	1776.1	6200.9
DA_37	4514.4	892.2	335.9	68.4	30.7	1.9	2.6	429.5	1761.4	6275.8
DA_38	4603.9	890.6	335.3	68.2	30.6	1.9	2.6	426.4	1755.6	6359.5
DA_39	4692.1	871.1	327.9	66.7	30.0	1.8	2.5	407.5	1707.5	6399.7
DA_40	4774.7	867.5	326.6	66.4	29.8	1.8	2.4	403.2	1697.8	6472.5
DA_41	4857.3	860.2	323.9	65.7	29.5	1.8	2.4	391.3	1674.7	6532.0
DA_42	4939.9	859.1	323.4	65.5	29.4	1.8	2.3	388.8	1670.5	6610.4

Notes: All the cost values are in 1,000 CAD

5.5 Multi Criteria Decision Making (MCDM)

To systematically compare the performance of all design alternatives, a multi-criteria decision making (MCDM) framework was applied. Four criteria were considered: Initial Cost, LCC, Total

Cost, and Peak Mean Interstory Drift Ratio (IDR). Each criterion reflects a distinct dimension of value: construction economy, long-term loss potential, overall life-cycle economics, and seismic performance, so that together they capture both cost and demand aspects of the designs.

Since these criteria are expressed in different units and scales, they were first transformed to a common [0,1] benefit scale, where larger values indicate better performance. Under this formulation, lower costs and lower drift demands are mapped to higher normalized scores, ensuring that all criteria align directionally toward improvement. For example, DA_01 achieves the highest normalized score for Initial Cost, reflecting its lowest upfront expenditure, while its higher LCC and Total Cost values translate to low normalized scores on those axes.

To assign objective importance to each criterion, the Shannon entropy method was employed. This method evaluates the degree of dispersion in the normalized data: criteria with greater variability across the alternatives receive higher weights, while those with relatively uniform distributions receive lower weights. The resulting entropy weights are reported in Table 5.3 and form the basis for aggregating the normalized performance measures into composite scores and rankings presented in Sections 5.5.3 and 5.5.4.

5.5.1 Criteria Distributions

From the normalized plots in Figure 5.5, DA_01 achieves the best normalized score on Initial Cost, as it remains the cheapest alternative in terms of construction. On Total Cost, however, DA_01 is positioned in the lower half of the distribution, reflecting the penalty of elevated life-cycle losses that erodes the apparent advantage of its low upfront expenditure; several mid-range designs with slightly higher Initial Cost attain better normalized Total Cost scores by combining moderate construction cost with reduced losses.

For LCC, DA_01 sits toward the unfavorable end of the distribution, with many value-based alternatives achieving lower normalized LCC scores due to their improved drift and residual-drift performance. Its relatively high repair, contents, and casualty losses confirm that seismic performance deficiencies translate directly into higher long-term economic consequences.

In terms of Peak Mean IDR, DA_01 is clearly separated from the most favorable designs, occupying one of the poorest positions in the normalized IDR plot, while the majority of value-based alternatives cluster toward lower normalized IDR values

Taken together, these distributions reveal a consistent pattern: DA_01's advantage lies solely in Initial Cost, but this is outweighed once long-term performance is considered, since it performs poorly on both LCC and IDR and therefore attains a weaker position on Total Cost than many alternatives. The evidence reinforces that relying on minimum upfront expenditure is economically and structurally inefficient, whereas mid-range designs that manage drift provide more balanced and favorable outcomes across all criteria.

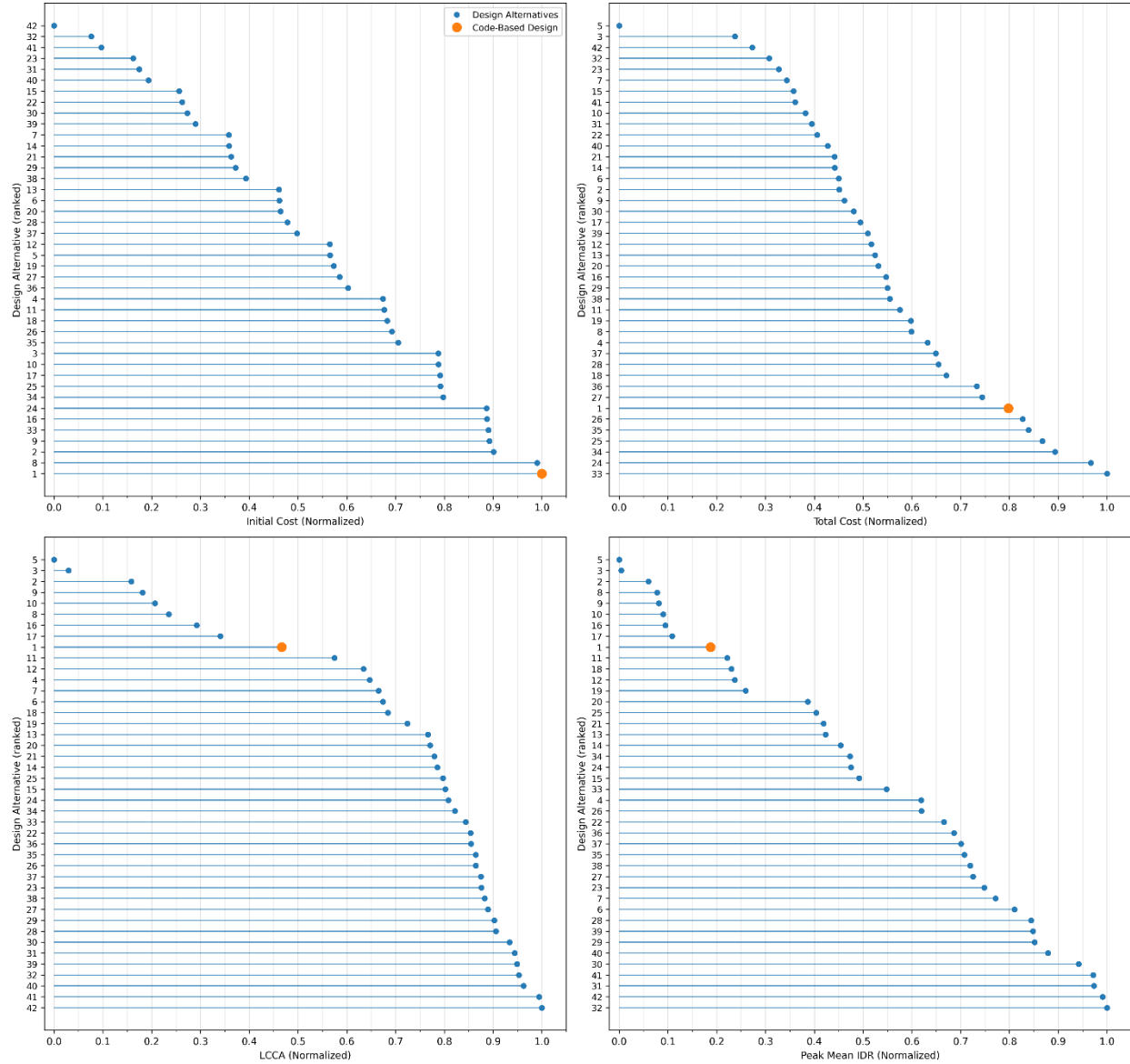


Figure 5.5: Normalized Plots (top-left: Initial Cost, top-right: Total Cost, bottom-left: LCC & bottom-right: Peak Mean IDR)

5.5.2 Entropy-based Weights

Objective weights derived using the Shannon entropy method are reported in Table 5.3. The weights reflect the degree of variability observed across the normalized criteria: measures with greater dispersion are assigned higher relative importance, while criteria with more uniform distributions are weighted lower.

The results show that Peak Mean IDR ($w = 0.397$) carries the highest weight, followed by Initial Cost ($w = 0.259$) and LCC ($w = 0.198$), while Total Cost ($w = 0.146$) is weighted lowest. This indicates that variation in drift performance is the most influential factor in distinguishing among the design alternatives, with construction cost and life-cycle losses also contributing meaningfully, and Total Cost adding relatively less new information once the other criteria are accounted for.

This weighting pattern is consistent with the broader performance trends: because Peak Mean IDR and residual drift vary noticeably across the alternatives, they retain strong discriminatory power in the normalized dataset, while differences in Initial Cost and LCC are more moderate and Total Cost exhibits the narrowest spread once those components are considered separately. As a result, the entropy-based decision emphasis falls primarily on controlling drift, with construction cost and life-cycle losses acting as secondary economic filters and Total Cost playing a comparatively minor role in the ranking.

Table 5.3: Entropy Weights

Criterion	Entropy (e_j)	Diversification (d_j)	Weight (w_j)
Peak Mean IDR	0.941	0.059	0.397
Initial Cost	0.961	0.039	0.259
LCC	0.971	0.029	0.198
Total Cost	0.978	0.022	0.146

5.5.3 Weighted-sum model ranking results

Aggregated scores from the weighted-sum model are presented in Table 5.4. The leading alternatives in the weighted-sum ranking are DA_33, DA_35, and DA_28, followed closely by DA_27 and DA_24–DA_26, all of which outperform the code-based baseline (DA_01) across multiple criteria. DA_01 itself ranks 25th, highlighting its weak overall competitiveness when evaluated on both cost and seismic performance.

Table 5.5 summarizes the performance of the top three WSM-ranked designs DA_33, DA_35, and DA_28 relative to DA_01 in native units (1,000 CAD). DA_33 requires a modest increase in Initial Cost (+93.7 kCAD, +2.29 %) but reduces LCC by about 273.8 kCAD (−13.3 %), yielding a net reduction in Total Cost of 180.1 kCAD (−2.93 %) and lowering Peak Mean IDR by 0.10 percentage points (−5.44 %). DA_35 shows a larger initial premium (+251.9 kCAD, +6.17 %) but achieves a 288.7 kCAD (−14.03 %) reduction in LCC and a smaller 36.8 kCAD (−0.60 %) reduction in Total Cost, while improving Peak Mean IDR by 0.14 percentage points (−7.85 %). DA_28 has the highest Initial Cost increase among the top three (+445.9 kCAD, +10.91 %) and the largest LCC reduction (−318.6 kCAD, −15.48 %), yet its higher Initial Cost more than offsets the savings, leading to a 127.3 kCAD (+2.07 %) increase in Total Cost despite the largest drift reduction (−0.18 percentage points, −9.91 %).

These results indicate that DA_33 offers the best overall economic trade-off relative to DA_01, combining a small Initial Cost increase with meaningful reductions in both LCC and Peak Mean IDR, while DA_35 emphasizes further drift reduction at the expense of a larger construction premium and DA_28 prioritizes seismic performance improvements over life-cycle cost efficiency.

It is also worth noting that the broader economic value of improved structural performance may extend beyond the life-cycle losses quantified in this study. In the Canadian housing context, insurance premiums constitute a significant portion of property-related expenses. More resilient structures characterized by reduced drift demands and lower repair probabilities could reasonably attract lower insurance costs. While such data were not available for integration into this analysis, the potential for insurance savings highlights an additional channel through which value-based designs may deliver long-term financial benefits.

Table 5.4: Weighted Sum Model Ranking scores for all DAs

Rank	DA	Score
1	DA_33	0.761324
2	DA_35	0.757222
3	DA_28	0.733779
4	DA_27	0.724146
5	DA_24	0.719479
6	DA_26	0.717328
7	DA_36	0.70487
8	DA_30	0.699558
9	DA_29	0.693214
10	DA_34	0.687784
11	DA_31	0.675759
12	DA_37	0.675174
13	DA_39	0.674061
14	DA_41	0.660039
15	DA_40	0.651893
16	DA_25	0.650225
17	DA_32	0.649958
18	DA_38	0.643151
19	DA_04	0.640921
20	DA_06	0.640437
21	DA_42	0.630902
22	DA_07	0.580593
23	DA_22	0.560524
24	DA_23	0.560166
25	DA_01	0.542708
26	DA_13	0.515649
27	DA_20	0.503783
28	DA_18	0.501598
29	DA_14	0.493231
30	DA_19	0.48227
31	DA_21	0.479046
32	DA_15	0.472535
33	DA_11	0.461164
34	DA_12	0.441625
35	DA_08	0.421723
36	DA_16	0.405404
37	DA_17	0.387882
38	DA_09	0.367023
39	DA_02	0.354728
40	DA_10	0.337022
41	DA_03	0.246588
42	DA_05	0.146758

Beyond direct economic effects, mid-range resilient designs may also contribute to a **reduced carbon footprint** over the building's life cycle. By minimizing damage and repair needs after earthquakes, these alternatives lower the demand for replacement materials, construction waste,

and associated emissions. Although carbon accounting was not included in this study, the synergy between **seismic resilience and environmental sustainability** underscores the broader societal value of adopting value-based design principles.

Table 5.5: Cost comparison between DA_01 with top 3 WSM ranked design

DA	Initial Δ (abs)	Initial Δ (%)	LCCA Δ (abs)	LCCA Δ (%)	Total Δ (abs)	Total Δ (%)	Peak Mean IDR Δ (abs)	Peak Mean IDR Δ (%)
DA_33	93.66	2.29	-273.75	-13.30	-180.10	-2.93	-0.10	-5.44
DA_35	251.91	6.17	-288.74	-14.03	-36.83	-0.60	-0.14	-7.85
DA_28	445.85	10.91	-318.57	-15.48	127.28	2.07	-0.18	-9.91

5.5.4 Robustness of Results

The stability of the ranking outcomes was evaluated through a sensitivity analysis in which the entropy weights were perturbed individually by $\pm 5\%$ and $\pm 10\%$ while keeping the remaining criteria proportionally adjusted. The results, summarized in Table 5.6, demonstrate that the leading alternatives consistently come from the same small set primarily DA_33, DA_35, and DA_28, with DA_24 occasionally entering the top three when the Peak Mean IDR weight is reduced by 10% confirming the robustness of the short-listed solutions.

Minor shifts were observed in the rank of the baseline design DA_01, which varied between 23rd and 25th place depending on the perturbation. Composite scores for DA_01 fluctuated only within a narrow band (≈ 0.529 – 0.557), indicating that its underperformance is consistent regardless of modest changes in weighting. Importantly, no tested adjustment brought DA_01 into a competitive range relative to the leading alternatives.

Table 5.6: Sensitivity Analysis Results

Perturbed Criterion	Change	w[IDR]	w[IC]	w[LCC]	w[TC]	Top3	Code Rank (DA_01)	Code Score
Peak Mean IDR	-5%	0.384	0.265	0.202	0.149	['DA_33', 'DA_35', 'DA_28']	25	0.550
Peak Mean IDR	+5%	0.408	0.254	0.194	0.143	['DA_33', 'DA_35', 'DA_28']	25	0.536
Peak Mean IDR	-10%	0.372	0.270	0.206	0.152	['DA_33', 'DA_35', 'DA_24']	23	0.557
Peak Mean IDR	+10%	0.420	0.249	0.190	0.140	['DA_35', 'DA_33', 'DA_28']	25	0.529
Initial Cost	-5%	0.402	0.250	0.201	0.148	['DA_33', 'DA_35', 'DA_28']	25	0.537
Initial Cost	+5%	0.392	0.269	0.196	0.144	['DA_33', 'DA_35', 'DA_28']	25	0.549
Initial Cost	-10%	0.407	0.240	0.203	0.150	['DA_35', 'DA_33', 'DA_28']	25	0.531
Initial Cost	+10%	0.387	0.278	0.193	0.142	['DA_33', 'DA_35', 'DA_28']	23	0.554
LCC	-5%	0.401	0.262	0.190	0.147	['DA_33', 'DA_35', 'DA_28']	25	0.543
LCC	+5%	0.393	0.257	0.206	0.145	['DA_33', 'DA_35', 'DA_28']	25	0.542
LCC	-10%	0.405	0.265	0.182	0.149	['DA_33', 'DA_35', 'DA_28']	25	0.544
LCC	+10%	0.389	0.254	0.214	0.143	['DA_33', 'DA_35', 'DA_28']	25	0.541
Total Cost	-5%	0.400	0.261	0.199	0.140	['DA_33', 'DA_35', 'DA_28']	25	0.541
Total Cost	+5%	0.394	0.258	0.197	0.152	['DA_33', 'DA_35', 'DA_28']	25	0.545
Total Cost	-10%	0.402	0.263	0.201	0.133	['DA_33', 'DA_35', 'DA_28']	25	0.539
Total Cost	+10%	0.391	0.256	0.195	0.158	['DA_33', 'DA_35', 'DA_28']	25	0.546

These findings confirm that the MCDM conclusions are **robust to moderate variations in criterion importance**. Even when the relative weight of Peak Mean IDR, Initial Cost, LCCA, or Total Cost was increased or decreased by up to 10%, the leading designs always belonged to the same small group (DA33, DA35, DA28, and occasionally DA24), and their relative ordering changed only slightly. This robustness further strengthens confidence in the validity of the identified mid-range alternatives as optimal choices.

5.6 Resiliency

The Resiliency Index was evaluated for all design alternatives across a range of annual exceedance probabilities, as shown in **Figure 5.6**. The distribution indicates that nearly all value-based alternatives achieve equal or higher resiliency than the code-based baseline (DA_01). The baseline consistently tracks along the lower envelope, particularly at rare, high-intensity hazard levels (e.g., 2–5% in 50 years). This outcome highlights the vulnerability of DA_01 under severe seismic scenarios and confirms the performance gap that can be closed through value-based design.

To focus on representative options, **Figure 5.7** compares DA_01 with the three top-ranked alternatives identified from the MCDM analysis: DA_33, DA_35, and DA_28, which represent efficient combinations of cost and seismic performance. All three alternatives consistently outperform the baseline across the probability range, with the advantage most evident at low exceedance probabilities, where structural resilience is most critical. Although the percentage differences appear modest, they represent meaningful improvements in expected recovery capacity under extreme earthquake events.

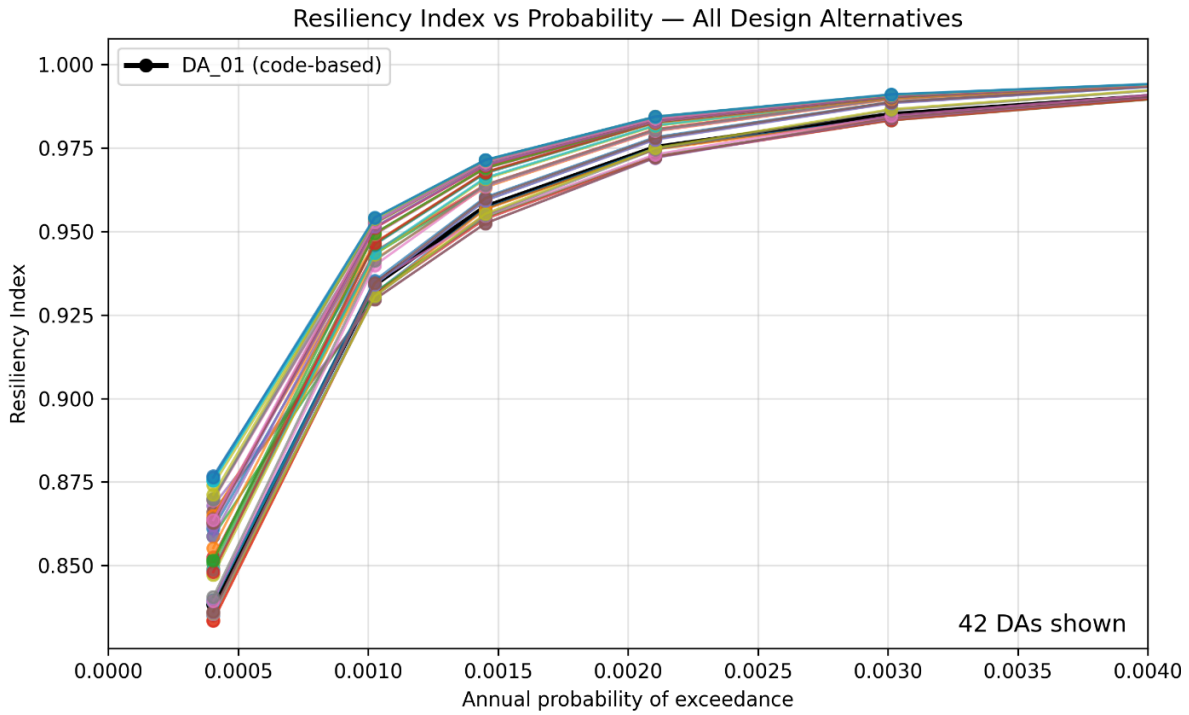


Figure 5.6: Comparison of Resiliency Index (All DAs)

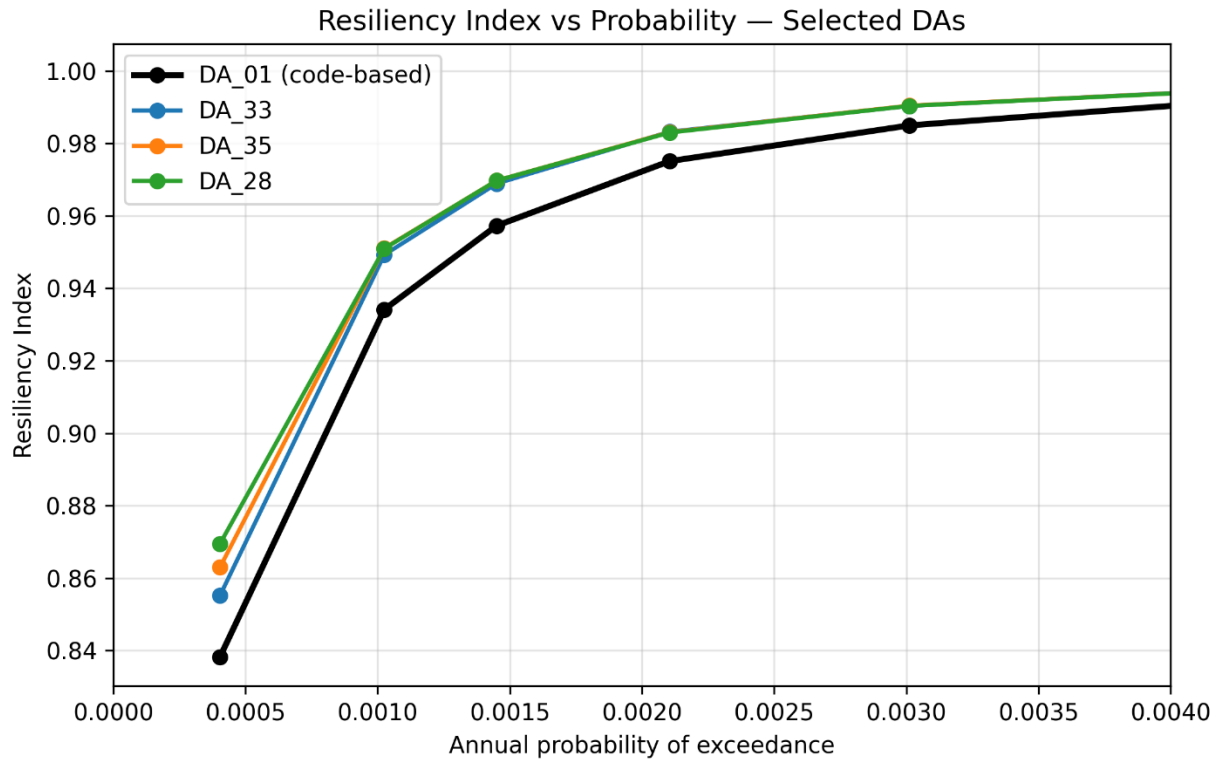


Figure 5.7: Comparison of Resiliency Index (DA_01 vs Top 3 ranked DAs)

Numerical values in Table 5.7 further illustrate these trends. At the 2 % in 50-year hazard level, DA28 achieves the largest resiliency gain relative to DA_01 (about +3.73 %), followed by DA_35 (+2.95 %) and DA_33 (+2.02 %). Even at intermediate hazard levels between 5 % and 10 % in 50 years, the value-based designs maintain gains on the order of 0.8–1.8 %, whereas differences at frequent hazards (30–40 % in 50 years) remain modest but consistently positive.

Table 5.7: Resiliency Comparisons of top 3 design alternatives with code based design

DA	40%/50 yr	30%/50 yr	20%/50 yr	14%/50 yr	10%/50 yr	7%/50 yr	5%/50 yr	2%/50 yr
DA_01	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
DA_28	0.039	0.061	0.254	0.547	0.809	1.301	1.801	3.726
DA_33	0.037	0.065	0.271	0.546	0.833	1.214	1.617	2.021
DA_35	0.040	0.061	0.257	0.554	0.820	1.301	1.812	2.951

The combined cost and resiliency outcomes can be illustrated in Figure 5.8, by comparing DA_01 with DA_33, which is both the top WSM-ranked design and one of the most resilient alternatives. DA_33 requires only a 2.29% increase in Initial Cost relative to DA_01 (+93.7 kCAD) but achieves a 13.3% reduction in LCC (−273.8 kCAD), resulting in a 2.93% decrease in Total Cost (−180.1 kCAD) and a Resiliency Index improvement of about 2.0 % at the 2 % in 50-year hazard level.

This example demonstrates that value-based designs can simultaneously enhance seismic resilience and reduce life-cycle cost compared to the code-based baseline, with DA_35 and DA_28 exhibiting similar patterns but placing slightly more emphasis on drift reduction than on pure economic efficiency.

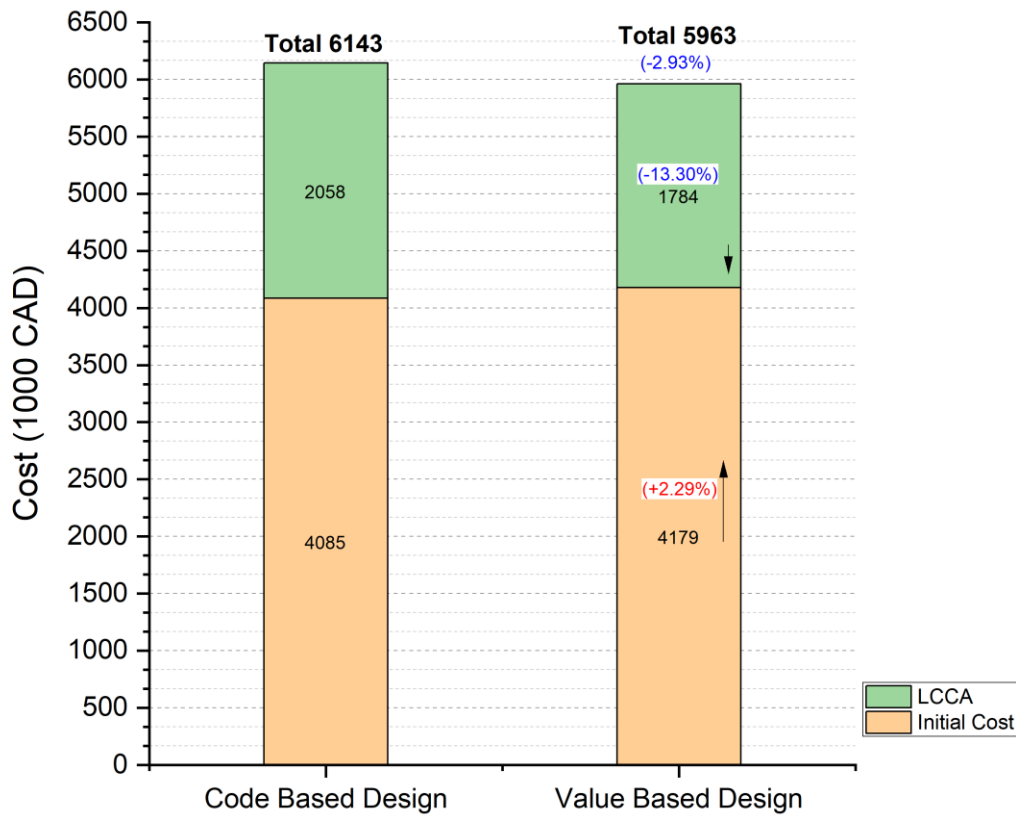


Figure 5.8: Cost Comparison of Code Based vs Value Based Design

5.7 Final Synthesis of Results

The results of this study demonstrate that the code-based baseline (DA_01), while offering the lowest Initial Cost, is not the most efficient option when life-cycle performance and resilience are considered. Its higher repair, contents, and casualty losses increase LCC, and its higher Peak Mean IDR and residual drift reduce the Resiliency Index, leaving it economically and structurally less competitive than several value-based alternatives.

The life-cycle cost analysis showed that Total Cost does not rise monotonically with Initial Cost; instead, the lowest Total Costs are achieved by mid-range designs such as DA_33, DA_35, and nearby alternatives, which reduce life-cycle losses sufficiently to offset modest increases in construction cost. The MCDM analysis, using entropy-based weighting of Peak Mean IDR, Initial

Cost, LCC, and Total Cost, confirmed that DA_33, DA_35, and DA_28 consistently occupy the top positions, while DA_01 ranks only 25th, indicating that its low Initial Cost cannot compensate for its higher losses and weaker seismic performance. Sensitivity tests under $\pm 5\text{--}10\%$ perturbations of each weight showed that the leading set of designs (DA_33, DA_35, DA_28, occasionally DA_24) remains unchanged and that DA_01 never approaches the top tier, confirming the robustness of the rankings.

The Resiliency Index provided additional confirmation that value-based designs outperform the code-based baseline under rare seismic events. At the 2 % in 50-year exceedance level, DA_28, DA_35, and DA_33 improve resiliency by approximately +3.73 %, +2.95 %, and +2.02 %, respectively, relative to DA_01, with consistent advantages of roughly 0.8–1.8 % at intermediate hazard levels between 5 % and 10 % in 50 years. A combined cost–resiliency comparison shows that DA_33, for example, requires only a 2.29 % increase in Initial Cost (+93.7 kCAD) but reduces LCC by 13.3 % (–273.8 kCAD) and Total Cost by 2.93 % (–180.1 kCAD), while delivering higher resiliency across all hazard levels.

Taken together, these findings highlight that the most efficient outcomes lie in mid-range designs, where modest upfront investments deliver long-term savings and measurable resilience gains. Neither the cheapest nor the most expensive alternatives provide the best economic outcome: DA_01 underperforms due to elevated drift-induced losses, while very high-cost designs such as the heaviest alternatives provide only marginal additional resilience at the expense of higher total expenditure. The value-based design framework therefore identifies optimal solutions typified by DA_33 and DA_35 that achieve a superior balance across cost, seismic safety, and resilience, going beyond the limitations of prescriptive code-based approaches.

In summary, the results confirm that value-based alternatives, particularly those in the mid-cost range, deliver the most favorable balance between construction economy, life-cycle losses, and seismic resilience. These findings provide the empirical foundation for the following discussion, where their broader implications are examined in the context of resilience-based seismic design, Canadian practice under NBCC 2020, and future opportunities for integrating additional value dimensions such as insurance premiums into the evaluation framework.

CHAPTER 6: SUMMARY AND CONCLUSIONS

6.1 Summary of Study

The central objective of this research was to evaluate the seismic resilience of a six-story reinforced concrete (RC) building located in Vancouver, Canada, under the requirements of the National Building Code of Canada (NBCC 2020). The study applied a **Value-Based Design (VBD) framework**, integrating advanced seismic analysis, life-cycle cost evaluation, and resilience metrics to identify designs that optimize both structural and economic performance.

A key methodological innovation was the use of the **Endurance Time Method (ETM)**, implemented through **Endurance Time Excitation Functions (ETEFs)**. Two approaches were investigated: (i) a segmented, single-record strategy (ETEF_C) and (ii) multiple individually scaled records (ETEF_A). Both were benchmarked against Incremental Dynamic Analysis (IDA) with real ground motions to establish accuracy and reliability.

Subsequent analyses included **Life-Cycle Cost Analysis (LCCA)**, **Multi-Criteria Decision Making (MCDM)** using entropy-based weighting, and **Resiliency Index evaluation**. These analyses enabled systematic comparison of 42 design alternatives, including a code-based baseline (DA_01) and multiple value-based alternatives. Together, these methods provided a comprehensive assessment of the economic and performance outcomes of different design choices.

6.2 Key Findings

Based on the study the following observations are made.

- a) **Life-Cycle Cost and Resilience Trade-offs:** Code-minimum design (DA_01) minimized Initial Cost (4.09 M CAD) but incurred a very high lifecycle economic burdens, with total cost almost 6.14 M CAD driven by large structural repair and casualty components, including

about 0.64 M CAD in expected fatality losses. Value-based alternatives in mid-range, particularly DA_33, reduced structural LCC by 13.3% and total cost by about 2.9% relative to DA_01 despite only a 2.3% increase in initial cost. Neither the cheapest nor most expensive alternatives provided optimal outcomes: DA_01 economically inefficient once losses are included, whereas high-cost designs (e.g., DA_42) exhibited diminishing returns.

- b) **Multi-Criteria Decision-Making (MCDM) Ranking:** The MCDM analysis, incorporating normalized cost and performance criteria weighted through the Shannon-entropy method, consistently ranked DA_33, DA_35, and DA_28 as the top-performing designs. These alternatives outperformed the code-based baseline in both economic and seismic metrics, confirming that small premiums in Initial Cost can be traded for significant reductions in life-cycle losses. Sensitivity analysis demonstrated that the ranking was robust under $\pm 10\%$ perturbations of criterion weights, with the top three designs remaining unchanged in all but one scenario, where DA_24 temporarily replaces DA_28 when the Peak Mean IDR weights is reduced by 10% across all scenarios.
- c) **Seismic Resilience Evaluation:** The Resiliency Index further confirmed the superiority of value-based alternatives over the code-based baseline. At the design-basis hazard (2% in 50 years), mid-range designs such as DA33, DA35, and DA28 achieved Resiliency Index improvements on the order of 2–4 percentage points relative to DA01, with consistent smaller gains at moderate hazard levels. These designs combine reduced drift demands and shorter recovery times, translating into higher functionality retention across the hazard ladder while remaining economically competitive in terms of Total Cost.

These results highlight a clear **cost–resiliency relationship**: value-based alternatives not only reduced life-cycle losses but also improved seismic recovery capacity compared to the code-

based baseline. The most efficient outcomes clustered around **mid-range designs**, where moderate increases in Initial Cost produced disproportionate gains in both economic and resilience performance. In contrast, high-cost designs exhibited diminishing returns, with large upfront investments failing to translate into meaningful resiliency advantages.

Overall, the findings underscore the importance of **integrating cost, performance, and resilience metrics** into the design process. This balanced perspective operationalizes the cost–resiliency link emphasized in recent literature and demonstrates the practical benefits of a value-based design framework.

d) **ETEF Validation and Performance**

The study implemented two types of endurance time excitation functions: ETEF_A, consisting of eight separate 40-second spectra-matched functions, and ETEF_C, a combined 40-second record built from 5-second hazard-specific segments. Validation against Incremental Dynamic Analysis (IDA) with real ground motions confirmed the credibility of ETM. **ETEF_A** showed excellent alignment with IDA median responses, with very low scatter ($\sigma \approx 0.0401$). **ETEF_C**, while efficient, exhibited conservative bias and higher variability ($\sigma \approx 0.0655$), attributed to its segmented merging approach.

These results reinforce that spectrum-matched ETEFs can serve as practical proxies for real ground motion ensembles, though merging strategies require refinement.

6.3 Limitations

This study was conducted within a clearly defined scope, which ensured depth of analysis but also sets boundaries on the generality of the findings.

- a) **ETEF Generation Strategy:** The study implemented two forms of endurance time excitation functions: hazard-specific spectra-matched ETEFs (ETEF_A) and a simplified merged approach (ETEF_C). While ETEF_C provided efficiency for exploratory comparison, its use of short 5-second concatenated segments does not fully capture cumulative hazard duration and may have introduced bias. This was an intentional simplification to test the practicality of spectrum-matching software (SeismoMatch) in the Canadian context.
- b) **Fixed Duration of 40 Seconds:** Both ETEF_A and ETEF_C were limited to 40 seconds. This aligns with many applications of ETM in the literature and was sufficient for comparative analysis; however, it may not fully reflect the long-duration shaking expected from Cascadia subduction earthquakes. Recognizing this provides a pathway for extending ETM to more region-specific hazard conditions in future work.
- c) **2D frame and diaphragm idealization:** The nonlinear analysis used a two-dimensional frame model with lumped floor masses and in-plane rigid diaphragm assumptions; diaphragm flexibility and associated torsional effects were not explicitly modeled.
- d) **Scope of Economic Losses:** The life-cycle cost analysis focused on **structural losses** (damage repair, contents, rental, relocation, and casualty). Non-structural components, such as partitions and building services, were not included due to data unavailability, even though they often govern downtime and repair costs. Similarly, potential effects on insurance premiums and long-term occupancy impacts were outside the present scope. This choice allowed the study to isolate the structural performance–cost relationship, while leaving broader economic integration as a future research opportunity.
- e) **Design Alternative Space:** Design alternatives were generated by modifying **column dimensions and reinforcement ratios**, providing a controlled comparison set. In practice,

many other parameters (beam detailing, material grades, system-level variations such as dual systems or base isolation) could be explored. The restricted scope here ensured methodological consistency while demonstrating proof of concept for value-based design. Future work can expand the design space to capture a wider spectrum of solutions.

6.4 Future Scope

This study provides a foundation for applying value-based design (VBD) and endurance time methods (ETM) in the Canadian context but also opens several directions for future research.

- a) **Design Alternative Exploration:** Expanding the design space beyond column dimensions and reinforcement ratios would allow for a more comprehensive evaluation of efficiency. Future studies could incorporate variations in beam detailing, material grades, joint configurations, or even system-level strategies such as dual systems, supplemental damping, and base isolation. Such an expanded parameter set could reveal design solutions with even stronger cost–resilience performance than those observed here.
- b) **Integration of Non-Structural Losses:** The present life-cycle cost analysis focused on structural components. Including non-structural elements such as partitions, cladding, and building services is critical, as they often drive repair time, downtime, and functionality losses. Their inclusion would yield a more realistic assessment of both economic consequences and resilience.
- c) **Insurance and Socioeconomic Dimensions:** Improved structural performance has the potential to lower insurance premiums and reduce broader socioeconomic losses in the Canadian housing market. While not captured here due to data limitations, integrating these factors could provide stakeholders with stronger incentives to adopt value-based alternatives.

- d) **Refinement of ETEF Methodology:** Future work should investigate hazard-consistent ETEFs with longer, intensity-dependent durations to better capture the cumulative effects of strong-motion shaking. Approaches such as smooth wavelet-domain merging or hazard-level-specific ETEFs, as recommended by Estekanchi et al., could reduce bias and improve alignment with Incremental Dynamic Analysis (IDA) results.
- e) **Application to Regional Hazard Scenarios:** Extending the framework to site-specific hazard conditions, such as long-duration shaking from Cascadia subduction earthquakes in Vancouver, would provide more regionally tailored insights. Combining VBD with region-specific hazard spectra would increase the practical applicability of the findings for Canadian seismic design.

6.5 Concluding Remark

This study demonstrates that value-based design, when combined with endurance time analysis, offers a powerful framework for balancing cost, performance, and resilience in seismic design. By systematically evaluating alternative designs against both life-cycle economic metrics and recovery-based resilience indicators, the research shows that modest mid-range investments can achieve meaningful reductions in losses while enhancing seismic resilience. Although limited in scope to structural components and a focused set of design variations, the findings provide a strong proof of concept for applying VBD in the Canadian context. The integration of economic and resilience perspectives highlights the potential for seismic design to move beyond prescriptive compliance toward a more holistic approach that maximizes value for both stakeholders and society.

References

- AIR Worldwide. (2013). *Study of Impact and the Insurance and Economic Cost of a Major Earthquake in British Columbia and Ontario/Québec*. Insurance Bureau of Canada. https://a-us.storyblok.com/f/1013643/x/4c2aead3fb/ibc_earthquakestudy_2013.pdf.
- Al Atik, L., & Abrahamson, N. (2010). An Improved Method for Nonstationary Spectral Matching. *Earthquake Spectra*, 26(3), 601–617. <https://doi.org/10.1193/1.3459159>.
- Alath, S., & Kunnath, S. (1995). Modeling Inelastic Shear Deformation in RC Beam-Column Joints. *Engineering Mechanics*. <https://www.semanticscholar.org/paper/Modeling-Inelastic-Shear-Deformation-in-RC-Joints-Alath-Kunnath/11a0b4f03001d6e706038d588e2700583d771ffb>.
- Alcocer, S. M. (2012). *Overview of the Revised ACI-ASCE 352 Committee Recommendations for Design of Beam-Column Connections in Monolithic Reinforced Concrete Structures*. 1–9. [https://doi.org/10.1061/40558\(2001\)69](https://doi.org/10.1061/40558(2001)69).
- Applied Technology Council. (2022). *ATC-138: Guidelines for the Estimation of Functional Recovery Time* (Technical Report No. ATC-138). Applied Technology Council.
- Atkinson, G., & Macias, M. (2009). Predicted Ground Motions for Great Interface Earthquakes in the Cascadia Subduction Zone. *Bulletin of The Seismological Society of America - BULL SEISMOL SOC AMER*, 99, 1552–1578. <https://doi.org/10.1785/0120080147>.
- Badal, P. S., & Tesfamariam, S. (2023). Seismic resilience of typical code-conforming RC moment frame buildings in Canada. *Earthquake Spectra*, 39(2), 748–771. <https://doi.org/10.1177/87552930221145455>.
- Belton, V., & Stewart, T. (2002). *Multiple Criteria Decision Analysis: An Integrated Approach*. <https://doi.org/10.1007/978-1-4615-1495-4>.
- Bruneau, M., Chang, S. E., Eguchi, R. T., Lee, G. C., O'Rourke, T. D., Reinhorn, A. M., Shinozuka, M., Tierney, K., Wallace, W. A., & von Winterfeldt, D. (2003). A Framework to Quantitatively Assess and Enhance the Seismic Resilience of Communities. *Earthquake Spectra*, 19(4), 733–752. <https://doi.org/10.1193/1.1623497>.
- Charkhtab Basim, M., E. Estekanchi, H., & Vafai, A. (2016). A Methodology for Value Based Seismic Design of Structures by the Endurance Time Method. *Scientia Iranica*, 23(6), 2514–2527. <https://doi.org/10.24200/sci.2016.2310>.
- Chopra, A. K. (2017). *Dynamics of structures: Theory and applications to earthquake engineering* (Fifth edition). Pearson; WorldCat.
- Cimellaro, G., Reinhorn, A., & Bruneau, M. (2010). Framework for analytical quantification of disaster resilience. *Engineering Structures*, 32, 3639–3649. <https://doi.org/10.1016/j.engstruct.2010.08.008>.

- Ciurlanti, J., Bianchi, S., & Pampanin, S. (2023). Raising the bar in seismic design: Cost–benefit analysis of alternative design methodologies and earthquake-resistant technologies. *Bulletin of Earthquake Engineering*, 21(5), 2723–2757. <https://doi.org/10.1007/s10518-023-01625-x>.
- Dhakal, R. (2024). Seismic design of buildings: Where to next? *Bulletin of the New Zealand Society for Earthquake Engineering*, 57(1), 1–17. <https://doi.org/10.5459/bnzsee.1680>.
- Lignos, D. and Krawinkler, H. (2011). *Deterioration Modeling of Steel Components in Support of Collapse Prediction of Steel Moment Frames under Earthquake Loading*. 137(11), 1291–1302. [https://doi.org/10.1061/\(ASCE\)ST.1943-541X.0000376](https://doi.org/10.1061/(ASCE)ST.1943-541X.0000376).
- Estekanchi, H. E., Mashayekhi, M., Vafai, H., Ahmadi, G., Mirfarhadi, S. A., & Harati, M. (2020). A state-of-knowledge review on the Endurance Time Method. *Structures*, 27, 2288–2299. <https://doi.org/10.1016/j.istruc.2020.07.062>.
- Estekanchi, H. E., Valamanesh, V., & Vafai, A. (2007). Application of Endurance Time method in linear seismic analysis. *Engineering Structures*, 29(10), 2551–2562. <https://doi.org/10.1016/j.engstruct.2007.01.009>.
- FEMA. (1997). *FEMA 273: NEHRP Guidelines for the Seismic Rehabilitation of Buildings*. (No. FEMA 273).
- FEMA. (2000). *FEMA 356: Prestandard and Commentary for the Seismic Rehabilitation of Buildings* (No. FEMA 356).
- FEMA. (2018). *Seismic Performance Assessment of Buildings (FEMA P-58)* (No. FEMA P-58-2). Federal Emergency Management Agency.
- Greco, S., Ehrgott, M., & Figueira, J. (2016). *Multiple Criteria Decision Analysis: State of the Art Surveys*. <https://doi.org/10.1007/978-1-4939-3094-4>.
- Hwang, C.-L., & Yoon, K. (1981). Methods for Multiple Attribute Decision Making. In C.-L. Hwang & K. Yoon (Eds.), *Multiple Attribute Decision Making: Methods and Applications A State-of-the-Art Survey* (pp. 58–191). Springer Berlin Heidelberg. https://doi.org/10.1007/978-3-642-48318-9_3.
- Jebelli, S. T., & Behnam, B. (2024). Intelligent Life Cycle Cost-Based Framework for Seismic Design of Conventional Structures. *Buildings*, 14(9), Article 9. <https://doi.org/10.3390/buildings14092597>.
- Krawinkler, H., & Miranda, E. (2004). 9.1. A Perspective of Performance-Based Earthquake Engineering. *Earthquake Engineering: From Engineering Seismology to Performance-Based Engineering*; CRC Press: Boca Raton, FL, USA, 87.
- Lowes, L., & Mitra, N. (2004). *A Beam-Column Joint Model for Simulating the Earthquake Response of Reinforced Concrete Frames*. <https://www.semanticscholar.org/paper/A-Beam-Column-Joint-Model-for-Simulating-the-of-Lowes-Mitra/d49e0ed0cb86a8c850d3bf76581c7f715db5f8fe>.

- Mander, J. B., Priestley, M. J. N., & Park, R. (1988). Theoretical Stress-Strain Model for Confined Concrete. *Journal of Structural Engineering*, 114(8), 1804–1826. [https://doi.org/10.1061/\(ASCE\)0733-9445\(1988\)114:8\(1804\)](https://doi.org/10.1061/(ASCE)0733-9445(1988)114:8(1804)).
- Mckenna, F. (2011). OpenSees: A Framework for Earthquake Engineering Simulation. *Computing in Science & Engineering*, 13, 58–66. <https://doi.org/10.1109/MCSE.2011.66>.
- Mirfarhadi, S. A., & Estekanchi, H. E. (2020). Value based seismic design of structures using performance assessment by the endurance time method. *Structure and Infrastructure Engineering*, 16(10), 1397–1415. <https://doi.org/10.1080/15732479.2020.1712436>.
- Mitchell, D., Paultre, P., Tinawi, R., Saatcioglu, M., Tremblay, R., Elwood, K., Adams, J., & DeVall, R. (2010). Evolution of seismic design provisions in the National building code of Canada. *Canadian Journal of Civil Engineering*, 37, 1157–1170. <https://doi.org/10.1139/L10-054>.
- Mitropoulou, C., Lagaros, N., & Manolis, P. (2011). Life-cycle cost assessment of optimally designed reinforced concrete buildings under seismic actions. *Reliability Engineering and System Safety*, 96, 1311–1331. <https://doi.org/10.1016/j.res.2011.04.002>.
- NBCC. (2020). *National Building Code of Canada: 2020 - NRC Publications Archive*. <https://nrc-publications.canada.ca/eng/view/object/?id=515340b5-f4e0-4798-be69-692e4ec423e8>.
- Noureldin, M., & Kim, J. (2020). Parameterized seismic life-cycle cost evaluation method for building structures. *Structure and Infrastructure Engineering*, 17(3), 425.
- Ogrodnik, K. (2019). Multi-Criteria Analysis of Design Solutions in Architecture and Engineering: Review of Applications and a Case Study. *Buildings*, 9(12), 244. <https://doi.org/10.3390/buildings9120244>.
- Porter, K. (2015). Beginner's Guide to Fragility, Vulnerability, and Risk. In *Encyclopedia of Earthquake Engineering* (pp. 235–260). Springer, Berlin, Heidelberg. https://doi.org/10.1007/978-3-642-35344-4_256.
- Rad, M. A. H., Jalaee, F., Golpour, A., Varzande, S. S. H., & Guest, G. (2021). BIM-based approach to conduct Life Cycle Cost Analysis of resilient buildings at the conceptual stage. *Automation in Construction*, 123, 1. <https://doi.org/10.1016/j.autcon.2020.103480>.
- Saltelli, A., Ratto, M., Andres, T., Campolongo, F., Cariboni, J., Gatelli, D., Saisana, M., & Tarantola, S. (2008). Global Sensitivity Analysis. The Primer. In *Global Sensitivity Analysis. The Primer* (Vol. 304). <https://doi.org/10.1002/9780470725184.ch6>.
- Sattar, S., Ryan, K., Arendt, L., Bonowitz, D., Comerio, M., Davis, C., Deierlein, G., & Johnson, K. J. (2021). Recommended Options for Improving the Built Environment for Post-Earthquake Reoccupancy and Functional Recovery Time. *NIST*. <https://www.nist.gov/publications/recommended-options-improving-built-environment-post-earthquake-reoccupancy-and>.

- Scott, M., & Fenves, G. (2006). Plastic Hinge Integration Methods for Force-Based Beam–Column Elements. *Journal of Structural Engineering-Asce - J STRUCT ENG-ASCE*, 132. [https://doi.org/10.1061/\(ASCE\)0733-9445\(2006\)132:2\(244\)](https://doi.org/10.1061/(ASCE)0733-9445(2006)132:2(244)).
- Shannon, C. E. (1948). A Mathematical Theory of Communication. *Bell System Technical Journal*, 27(3), 379–423. <https://doi.org/10.1002/j.1538-7305.1948.tb01338.x>.
- Smith, A. F. M. (1977). Decisions with Multiple Objectives: Preferences and Value Tradeoffs. *Royal Statistical Society. Journal. Series A: General*, 140(4), 556–557. <https://doi.org/10.2307/2345300>.
- Sullivan, T. (2020). *How would changes to serviceability limit state design criteria impact likely repair costs*.
- Tesfamariam, S., & Goda, K. (2015). Loss estimation for non-ductile reinforced concrete building in Victoria, British Columbia, Canada: Effects of mega-thrust Mw9-class subduction earthquakes and aftershocks. *Earthquake Engineering and Structural Dynamics*, 44(13), 2303–2320. <https://doi.org/10.1002/eqe.2585>.
- The Building Center of Japan. (2016). *The Building Standard Law of Japan*. The Building Center of Japan.
- Theilig, K., Lourenço, B., Reitberger, R., & Lang, W. (2024). Life cycle assessment and multi-criteria decision-making for sustainable building parts: Criteria, methods, and application. *The International Journal of Life Cycle Assessment*, 29(11), 1965–1991. <https://doi.org/10.1007/s11367-024-02331-9>.
- Tremblay, R., & Atkinson, G. M. (2001). Comparative Study of the Inelastic Seismic Demand of Eastern and Western Canadian Sites. *Earthquake Spectra*, 17(2), 333–358. <https://doi.org/10.1193/1.1586178>.
- Triantaphyllou, E. (2000). *Multi-Criteria Decision Making Methods: A Comparative Study* (Vol. 44). <https://doi.org/10.1007/978-1-4757-3157-6>.
- UNISDR. (2015). *Sendai Framework for Disaster Risk Reduction 2015—2030*. <https://www.unisdr.org/we/inform/publications/43291>.
- Vitiello, U., Asprone, D., Di Ludovico, M., & Prota, A. (2017). Life-cycle cost optimization of the seismic retrofit of existing RC structures. *Bulletin of Earthquake Engineering*, 15(5), 2245–2271. <https://doi.org/10.1007/s10518-016-0046-x>.
- Zeleny, M. (1982). *Multiple Criteria Decision Making*. McGraw-Hill. <https://books.google.ca/books?id=vTayAAAAIAAJ>.
- Zhu, M., McKenna, F., & Scott, M. (2018). OpenSeesPy: Python library for the OpenSees finite element framework. *SoftwareX*, 7, 6–11. <https://doi.org/10.1016/j.softx.2017.10.009>.

Appendix

Design Base Shear Calculation per NBCC 2020

This appendix documents the calculation of the design base shear for the six-storey ductile reinforced-concrete moment-resisting frame using the Equivalent Static Force Procedure (ESFP) prescribed in NBCC 2020, Part 4, Article 4.1.8.11.

Fundamental period and seismic weight

The approximate fundamental lateral period in the loading direction, T_a , is evaluated using the NBCC 2020 period formula for moment-resisting frames, expressed as a function of building height h_n :

$$T_a = 0.075(h_n)^{\frac{3}{4}}$$

The computed period for the case-study building is $T_a = 0.76\text{s}$

The seismic weight W is taken as the sum of all dead loads and the specified portion of snow loads, consistent with NBCC 2020 Article 4.1.8.11.(5)

$$W = \sum_{i=1}^n W_i$$

where W_i is the tributary weight at level i , including slab self-weight, beams, columns, partitions, finishes, and the appropriate fraction of live load. The total seismic weight of the six-storey frame is $W = 7,295 \text{ kN}$

Site parameters and design spectrum

Seismic hazard values for the Vancouver site are obtained from the NBCC 2020 online hazard tool for Site Class C, which provides design spectral accelerations $S(T)$ directly at multiple periods, already incorporating site effects for V_{s30} consistent with Class C conditions. The building is

classified as Normal importance residential, with importance factor $I_E = 1.0$ in accordance with NBCC 2020 Part.

Base shear

For structures where the Equivalent Static Force Procedure (ESFP) is permitted, NBCC 2020 defines the design base shear V at the base of the structure as

$$V = \frac{S(T_a) M_v I_E}{R_d R_o} W,$$

The code also imposes lower and upper bounds on this value. The minimum base shear is specified as

$$V_{min} = \frac{S(2.0) M_v I_E}{R_d R_o} W$$

$$V_{max} = \max \left\{ \begin{array}{l} \frac{2}{3} \frac{S(0.2) I_E W}{R_d R_o} \\ \frac{S(0.5) I_E W}{R_d R_o} \end{array} \right.$$

where:

- M_v is the higher-mode factor from Table 4.1.8.11,
- R_d is the ductility-related factor for the seismic force resisting system,
- R_o is the overstrength-related factor for the system.

For a ductile RC moment-resisting frame detailed to CSA A23.3, NBCC 2020 assigns $R_d = 4.0$ and $R_o = 1.7$. For the six-storey frame, the higher-mode factor is taken as $M_v = 1.0$, as the period and height fall within the range where Table 4.1.8.11 does not require amplification.

Substituting these values gives

$$V = \frac{S(T_a) (1.0) (1.0)}{4.0 \times 1.7} W = \frac{S(T_a)}{6.8} W.$$

Using the site-specific spectrum ordinate at T_a and the total seismic weight, the design base shear of the baseline design is calculated.

The corresponding base shear coefficient

$$C_v = \frac{V}{W}$$

These values are used as the target for the ESFP design in Section 3.2 and to define the reference demand level for the nonlinear dynamic analyses.

Vertical distribution of lateral forces

The storey shear and lateral forces applied in the static design are obtained by distributing the base shear V along the height in proportion to the product of storey weight and elevation, in accordance with NBCC 2020 Article 4.1.8.11.(7):

$$F_x = V \frac{W_x h_x}{\sum_{i=1}^n W_i h_i},$$

where F_x is the lateral force at level x , W_x is the seismic weight at level x , and h_x is its height above the base. The resulting floor forces and cumulative storey shears were used to design beams and columns of the baseline NBCC-compliant frame.