

Proceedings of **18th U.S.-Japan-New Zealand Workshop on the Improvement of Structural Engineering and Resilience**



Organized by



Applied Technology Council



Japanese Structural Consultants Association



New Zealand Society for Earthquake Engineering

Applied Technology Council

The Applied Technology Council (ATC) is an internationally recognized non-profit corporation founded by the Structural Engineers Association of California in 1973 to protect life and property through the advancement of science and engineering technologies. ATC's mission is to imagine, develop, and promote the advancement of technologies to enhance societal resistance to natural and other hazards. With proven expertise in seismic engineering, and demonstrated capabilities in wind, coastal inundation, and blast engineering, our funded research and technology transfer projects are formed through the development of nonproprietary consensus opinions on critical and important structural engineering issues.

Our library of over 300 published reports serve as significant contributions to the structural engineering profession at the national level--through development and evolution of building codes and standards; and to define seismic and other engineering design practice in the United States and internationally. ATC also identifies and encourages needed research, and disseminates technological advancements through guidelines, manuals, seminars, workshops and forums.

ATC is guided by a nationally recognized Board of Directors consisting of representatives appointed by the American Society of Civil Engineers, the National Council of Structural Engineers Associations, the Structural Engineers Association of California, the Structural Engineers Association of New York, and five at-large members representing a broad spectrum of engineering expertise. Each Board member serves a three-year term.

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Cover image: ATC-15-17 Workshop Participants in San Diego, California

ATC-15-17

Proceedings of 18th U.S.-Japan-New Zealand Workshop on the Improvement of Structural Engineering and Resilience

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Organized by
Applied Technology Council (ATC)
Japan Structural Consultants Association (JSCA)
New Zealand Society for Earthquake Engineering (NZSEE)

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Workshop Overview and Summary

The 18th installment of a workshop series (ATC-15) with the objective of collaboration among three countries: United States, Japan, and New Zealand was held on December 2-4, 2024 in San Diego, California. The event was organized by three established not-for-profit structural engineering organizations from the U.S.A., Japan, and New Zealand (the Applied Technology Council (ATC), Japan Structural Consultants Association (JSCA), and New Zealand Society for Earthquake Engineering (NZSEE), respectively).

1.1 About the Workshop Series

The ATC-15 workshop series was initiated in 1984 as a bi-lateral collaboration between ATC and JSCA to provide a forum for the rapid exchange of ideas and information for the advancement of structural engineering practice. Participants have included leading design practitioners and academic researchers from Japan, the United States, and other Pacific Rim countries. A detailed overview of the history of the Workshop series is presented in Rojahn (2014), available on the ATC website.

The 16th installment in the series (ATC-15-15) took place in Nara, Japan, in 2016 included participation from NZSEE, broadening the event to a tri-lateral collaboration. The workshop objective was broadened to bringing together leading seismic engineering researchers and practitioners to discuss and develop recommendations for improved community resilience based on topics related to current state-of-practice, innovative engineering solutions, and emerging resilience technologies.

1.2 18th U.S.-Japan-New Zealand Workshop

The 18th U.S.-Japan-New Zealand Workshop on the Improvement of Structural Engineering and Resilience (ATC-15-17) was jointly organized by ATC, JSCA, and NZSEE and hosted by ATC in San Diego, California, on December 4-6, 2024.

The Workshop assembled more than 50 participants from the three countries. A Workshop Steering Committee comprising 3 members from each country issued a call for abstracts, reviewed submissions, and also served as session moderators and presenters. Each Workshop participant was an author or presenter. All Workshop organizers and participants are listed in Appendix A.

The Workshop technical program included presentations and more than 50 technical papers on the latest advancements in the state of research, practice, and knowledge improving seismic resilience. Papers were separated into groups with similar topics and organized into sessions. The ATC-15-17 Program is presented at the end of this chapter. All papers, except for those not allowed to publish are presented in the next section, in order of presentation.

The Workshop program also allowed time for sessions where the participants engaged in discussions to develop technical and policy recommendations. A summary of these recommendations is presented in the next section. Note that the workshop presentations and discussions did not include foundation design or soil-structure interaction. These are additional considerations that should be included in further development of improving seismic resilience.

1.3 Workshop Summary and Recommendations

In December 2024, established not-for-profit structural engineering organizations from the U.S.A., Japan, and New Zealand (the Applied Technology Council (ATC), Japan Structural Consultants Association (JSCA), and New Zealand Society for Earthquake Engineering (NZSEE), respectively) hosted an international workshop with over 50 international engineering experts to present their latest work with a goal towards improving post-earthquake recovery of the built environment in support of overall community resilience. Workshop presentations, papers published in the ATC-15-17 Proceedings Document (ATC, 2025), and collaborative discussions during the event identified a number of common themes listed below to support post-earthquake recovery and resilience at the community level.

Current building codes aim at life safety performance objectives, but do not adequately address the need to reoccupy structures quickly to enable fast recovery to support community resilience. In this regard, modifications or additions to design approaches are needed with urgency for new and existing buildings, as well as infrastructure.

1. **Reducing seismic deformations** was mentioned repeatedly as one of the first measures that should be implemented to reduce structural and nonstructural damage, make repairs feasible, and hence facilitate ongoing functionality. Reducing drift to control damage has helped Japanese buildings to quickly recover to provide functionality.
2. In the U.S. and New Zealand, development of performance objectives and design guidance for **recovery-based design** are under development. It is noted that the countries have slightly different approaches with the U.S. efforts focusing on quantification of functional recovery time and the New Zealand efforts focusing on damage control through structural design. Implementation of recovery-based codes, standards, and other programs should consider functionality and economic consequences in terms of business disruption, life-cycle costs, carbon emissions, as

well as long-term costs of doing nothing in addition to considering initial cost of construction.

3. Performance of nonstructural components has a major impact on repairability and functionality of buildings following an earthquake. There was agreement that there is a need for global collaboration for the development of **new generation of nonstructural component technologies** that can accommodate earthquake demands as well as improved information collection for use in analyzing existing components.
4. When properly implemented, current safety-based building codes and related structural design standards generally use the concept of ductility to safely resist and dissipate the energy of earthquakes, but this approach inherently involves damage occurring to structural and nonstructural systems and components that must be repaired prior to achieving post-earthquake reoccupancy and functional recovery. The following actions can **enable repairability through recovery-based design**:
 - a. Coordination among the entire building team for inspections and repair, availability of relevant building drawings and structural models, and inclusion of access panels and relevant instructions can assist with observing damage and repair of vulnerable structural and nonstructural components.
 - b. Implementation of methods described in FEMA P-2335, a recently published U.S. methodology for inspection and repair of structures.
 - c. Study of field observations from sequence of earthquakes, e.g., Kumamoto earthquakes in Japan, can improve the understanding of the effect of multiple earthquakes.
 - d. Consideration of applicability and feasibility of new inspection technologies, e.g., unmanned drones, LiDar scans, should be included. Additionally, structural health monitoring during normal excitations, e.g., wind and traffic, can help identify components more likely to be damaged in an earthquake.
5. Lastly, several aspects of **outreach and education** were discussed:
 - a. Early integrated focus on building form by a broad range of engineers and professionals is needed to achieve cost effective decarbonization and resilience.
 - b. Frequent earthquakes (as in Japan) keep earthquake risks top of mind. For other countries with less frequent earthquakes (U.S. and NZ), it will be important to keep policies/tools “ready to go” when there is public demand.
 - c. Availability of simpler methods for analysis and design can enhance design quality.

1.4 Organizers

In addition to the technical presentations, the following brief information was shared about each organization:

Applied Technology Council (ATC)

- Non-profit corporation established in 1973 by the Structural Engineers Association of California (SEAOC)
- ATC's mission is to imagine, develop, and promote the advancement of technologies to enhance societal resistance to natural and other hazards
- Governed by 14-person Board (appointed by ASCE, SEAOC, SEAONY, NCSEA and at-large)
- Small technical staff (7) managing expert consultants (300 each year) to support technology transfer
- Major funding sources: U.S. Federal Agencies (FEMA, NIST, USGS), local Governments (City of San Francisco, State of Utah, California Office of Emergency Services), World Bank

Japan Structural Consultants Association (JSCA)

- Established in 1989 as an incorporated association based on the Structural Engineers Roundtable, which was founded in 1981 as a voluntary organization.
- The objective is to contribute to the realization of high-quality buildings in terms of cultural value as social assets, thereby promoting public welfare and culture.
- 12 permanent committees, 3 special committees and two expert committees
- JSCA's activities are supported by individual membership fees and voluntary activities.

New Zealand Society for Earthquake Engineering (NZSEE)

- Non-profit incorporated Society established in 1968
- The Society's mission is to gather, shape and apply knowledge to reduce the impact of earthquakes on our communities by:
- Fostering the advancement of the science and practice of earthquake engineering across all disciplines
- Promoting co-operation among scientists, engineers and other professionals in the broad field of earthquake engineering through interchange of knowledge, ideas, results of research and practical experience
- Governed by 12-person Management Society elected annually by our 1,100 members

- Publishes the quarterly NZSEE Bulletin technical journal, holds an Annual Technical Conference with roughly 400-450 delegates, produces industry design guidance documents and undertakes Learning from Earthquakes reconnaissance missions.
- Manages funded projects from the New Zealand Natural Hazards Commission (state insurer for natural hazard damage to residential properties) and other agencies.

1.5 Next Steps

At the conclusion of ATC-15-17, a representative from JSCA announced that the 19th installment of the Workshop series (ATC-15-18) will be held in Japan in 2026.

1.6 Workshop Program

The Workshop Program, as distributed at the event, is presented on the following pages. Papers available for publication in this document are linked from the program into the section entitled Technical Papers.



一般社団法人
日本建築構造技術者協会
Japan Structural Consultants Association



**18th U.S.-Japan-New Zealand Workshop
on the Improvement of Structural Engineering and Resilience**
Hilton San Diego Bayfront San Diego, California
December 2-4, 2024

WORKSHOP PROGRAM

SUNDAY, DECEMBER 1, 2024

Location: Elevation Room, Hilton Bayfront Hotel

5:00 pm – 7:00 pm: **Registration & Welcome Happy Hour** (Appetizers included, no-host bar)

MONDAY, DECEMBER, 2024

Location: Sapphire 400, Hilton Bayfront Hotel

8:30 am – 9:00 am: **Registration, Coffee/Tea**

9:00 am – 9:20 am: **Opening Remarks:** Hortacsu (US), Shintani (Japan), Ferner (NZ)

Session I

Moderator: Hortacsu (US)

9:20 am – 10:30 am: **Presentations** (5 presentations @ 12 minutes; 2 additional QA minutes per presenter)

- P1-1 Future Directions of seismic design in New Zealand - Seismic Risk Working Group Update; K. Elwood (NZ)*
- P1-2 Functional Recovery in the US – A Journey from Concept to Code; R. Kersting (US)*
- P1-3 US, Japan and New Zealand; Earthquake Resiliency and Functional Recover Concept Differences; co-presenters: K. Shintani* (Jp); K. Miyamoto (US); A. Moshref (NZ)*

10:30 am – 10:50 am: **Break**

Session II

Moderator: Shintani (Japan)

10:50 am – 12:00 pm: **Presentations** (5 presentations @ 12 minutes; 2 additional QA minutes per presenter)

- P2-1 Rethinking Importance Levels for Seismic Design in New Zealand; H. Ferner (NZ)*
- P2-2 Deaggregating Community-Level Socio-Economic Goals to Individual Building Seismic Performance; J. van de Lindt (US)*
- P2-3 Seismic Design in None-High Seismic Areas; H. Yoshikawa (Jp)*
- P2-4 Seismic Performance of Tall Steel-Framed Buildings built between 1960-1994; J. Malley (US)*
- P2-5 Comparing the performance outcomes for a reinforced concrete building designed to codified US-NZ-Japan design strategies; L. Hogan (NZ)*

*Presenting on behalf of author

12:00 pm – 1:00 pm: **Lunch** (Location: Aqua Terrace)

1:00 pm – 1:35 pm: **Discussion: Development of Technical and Policy Recommendations**
Moderated by Elwood (NZ)

Session III
Moderated by Ferner (NZ)

1:35 pm – 2:45 pm: **Presentations** (5 presentations @ 12 minutes; 2 additional QA minutes per presenter)

P3-1 Elastic Response Spectra - The Use of Ambient Excitation and Its Deconstruction for Structural Assessment; A. Jeary (US)

P3-2 Practical Techniques for Elastic Secondary Stiffness Design to Provide Seismic Stability and Enhance Seismic Resilience; L. Fahnestock (US)

P3-3 Mitigation of Seismic Collapse Risk of RC Frame Buildings by Reducing Design Drift; R. Dhakal (NZ)

P3-4 Designing for Lower Drift Limits: The Impacts on Construction Costs and Non-Structural Components; S. Pujol (NZ)

P3-5 Recent Advancements in Design and Fabrication of Eccentrically Braced Frames for Enhanced Ductility, Stability, and Resilience; P. Mortazavi (US)

2:45 pm – 3:05 pm: **Break**

Session IV
Moderated by Osada (Japan)

3:05 pm – 4:00 pm: **Presentations** (4 presentations @ 12 minutes; 2 additional QA minutes per presenter)

P4-1 Recent Developments for Seismic Design, Retrofit, and Repair of Concentrically Braced Frames; A. Sen (US)

P4-2 Overview of a Project on Resilience-based Design of Buildings with Accelerated Post-earthquake Functional Recovery; G. Tsampras (US)

P4-3 Conversion Project from Existing Supermarket to Town Hall; A. Matsuura (Jp)

P4-4 Structural Design of High-Rise Building Aiming for Higher Business Continuity Performance; M. Kuwa (Jp)

4:00 pm – 4:40 pm: **Discussion: Development of Technical and Policy Recommendations**
Moderated by Hortacsu (US)

4:40 pm – 5:00 pm: **Closure and Announcements**

6:30 pm – 9:00 pm: **Dinner (Location: Sapphire 400)** – Pacific Beach Buffett dinner, no-host bar

TUESDAY, DECEMBER 3, 2024

Location: Sapphire 400, Hilton Bayfront Hotel

9:00 am – 9:10 am: **Opening Remarks:** Ferner (NZ)

Session V Moderated by Mar (US)

9:10 am – 10:35 am: **Presentations** (6 presentations @ 12 minutes; 2 additional QA minutes per presenter)

- P5-1 New Guidelines for Post-earthquake Repair based on Performance-Critical Damage; A. Liel (US)*
- P5-2 Performance of Reinforced Concrete Buildings Damaged During the 2024 Noto Peninsula Earthquake in Japan; T. Obara (Jp)*
- P5-3 A Data-Driven Approach for Post-Earthquake Assessment of Seismic Damage in Woodframe Buildings; S. Muin (US)*
- P5-4 Risk and Resiliency Assessment in Practice: Assessment Taxonomy and Lessons Learned; B. Shao (US)*
- P5-5 Improving the Seismic Resilience of New Zealand Hospital Buildings; M. Grant (NZ)*
- P5-6 Structural Design and Construction of a Baseball Stadium with a Retractable Roof and a Seismic Isolation Roof; H. Kimura (Jp)*

10:35 am – 10:55 am: **Break**

Session VI Moderated by Yoshikawa (Japan)

10:55 am – 12:05 am: **Presentations** (5 presentations @ 12 minutes; 2 additional QA minutes per presenter)

- P6-1 Tamachi Tower Adopting Innovative Damper Controlled Building System; Y. Hotta (Jp)*
- P6-2 Detailing for Base Connection Ductility Courtesy of Replaceable Anchorage; T. Hutchinson (US)*
- P6-3 E-Isolation— Further Development of Seismic Isolation and Response Control Technology; Y. Noguchi (Jp)*
- P6-4 Lessons Learned from System-Level Response of Full-Scale Shake-Table Tests of Low Damage Buildings – Observations and linkages to component test results; G. Rodgers (NZ)*
- P6-5 Resilient Design for Functional Recovery in the United States: Emergence of Elective Non-Prescriptive Resilient Design, and Current Efforts to Codify Prescriptive Requirements for Resilient Design; C. Haselton (US)*

12:05 pm – 1:10 pm: **Lunch** (Location: Aqua Terrace)

1:10 pm – 1:45 pm: **Discussion: Development of Technical and Policy Recommendations**
Moderated by Rodgers (NZ)

Session VII
Moderated by Ferner (NZ)

1:45 pm – 2:40 pm: **Presentations** (4 presentations @ 12 minutes; 2 additional QA minutes per presenter)

- P7-1 Quality Assurance of Nonstructural Components for Functional Recovery Design; J. Valigura (US)*
- P7-2 Detailing for Seismically Resilient Steel Stair Systems: Validation in the Mass Timber (10 and 6-story) Programs; S. Sorosh (US)*
- P7-3 Seismic Performance Targets for Cladding and Curtain Wall Systems in San Francisco High-Rises: Lessons Learned Since Loma Prieta (2024 Update); M. White (US)*
- P7-4 New Zealand Code of Practice for the Seismic Performance of Non-Structural Elements; J. Stanway (NZ)*

2:40 pm – 2:55 pm: **Break**

Session VIII
Moderated by Rodgers (NZ)

2:45 pm – 4:20 pm: **Presentations** (6 presentations @ 12 minutes; 2 additional QA minutes per presenter)

- P8-1 Resilience and Sustainability Considerations in Performance-Based Seismic Design of Structures; E. Opabola (US)*
- P8-2 What can Structural Engineers do for Environmental Measures of Buildings; A. Osada (Jp)*
- P8-3 Driving Sustainable Design in the Structural Profession through Academic and Practitioner Collaboration; C. Toma (NZ)*
- P8-4 Modeling Post-Earthquake Functional Recovery of Bridges: Framework Development and Application; C. Wu (US)*
- P8-5 Quantitative Risk Analysis for Utilities; E. Jampole (US)*
- P8-6 An Integrated Resilience based Design Approach to Achieve Sustainability and Resilience; P. Brabhakaran (NZ)*

4:20 pm – 4:50 pm: **Discussion: Development of Technical and Policy Recommendations**
Moderated by Liel (US)

4:50 pm – 5:00 pm: **Closure and Announcements**

WEDNESDAY, DECEMBER 4, 2024

Location: Sapphire 400, Hilton Bayfront Hotel

9:00 am – 9:10 am: **Opening Remarks:** Shintani (Japan)

Session IX

Moderated by Yoshikawa (Japan)

9:10 am – 10:35 am: **Presentations** (6 presentations @ 12 minutes; 2 additional QA minutes per presenter)

P9-1 Seismic Design of Low Rise Buildings in New Zealand; M. Grant (NZ)

P9-2 The Salk Institute Laboratory A Performance-Based Design Case Study with FEMA P-58 Loss Modeling; D. Mar (US)

P9-3 National Activities to Improve Earthquake Resilience of Buildings; T. Mukai (Jp)

P9-4 Functional Recovery Requirements for New Buildings in the NEHRP Provisions and ASCE/SEI 7; R. Zimmerman (US)

P9-5 Performance targets for low damage seismic design in New Zealand; T. Sullivan (NZ)

P9-6 Development of Advanced Structural Health Monitoring System; K. Shintani (Jp)

10:35 am – 10:55 am: **Discussions: Development of Technical and Policy Recommendations**
Moderated by Hortacsu (US)

10:55 am – 11:15 am: **Break**

Closing Session

Moderated by Hortacsu (US), Shintani (Japan), Ferner (NZ)

11:15 am – 12:00 pm: **Workshop Summary & Wrap-Up**

Lunch on own

Technical Papers

The following table serves as a directory for all papers provided in this document.

Paper No.	Paper Title	Author(s)
P1-3	US, Japan and New Zealand; Earthquake Resiliency and Functional Recover Concept Differences; co-presenters	K. Miyamoto; A. Gilani; A. Wada; A. Moshref
P2-1	Rethinking Importance Levels for Seismic Design in New Zealand	H. Ferner; S. Abeling; S. Beaven; C. Brown; H. Cowan; D. Brunsdon; C. Dunne; K. Elwood; J. Hare; D. Gill; R. Jury
P2-2	Deaggregating Community-Level Socio-Economic Goals to Individual Building Seismic Performance	J. van de Lindt; M. Roohi; O Sediek; J. Kruse
P2-3	Seismic Design in Non-High Seismic Areas	H. Yoshikawa
P2-4	Seismic Performance of Tall Steel-Framed Buildings built between 1960-1994	J. Malley; R. Pekelnicky
P2-5	Comparing the Performance Outcomes for a Reinforced Concrete Building Designed to Codified US-NZ-Japan Design Strategies	N. Buck; M. Stephens; L. Hogan
P3-1	Elastic Response Spectra - The Use of Ambient Excitation and Its Deconstruction for Structural Assessment	A. Jeary; T. Winant; J. Bunyan; P. Drummond
P3-2	Practical Techniques for Elastic Secondary Stiffness Design to Provide Seismic Stability and Enhance Seismic Resilience	L. Fahnestock; S. Shi; R. Sabelli; M. Speicher
P3-3	Mitigation of Seismic Collapse Risk of RC Frame Buildings by Reducing Design Drift	R. Dhakal
P3-4	Designing for Lower Drift Limits: The Impacts on Construction Costs and Non-Structural Components	L. Pledger; S. Pujol
P3-5	Recent Advancements in Design and Fabrication of Eccentrically Braced Frames for Enhanced Ductility, Stability, and Resilience	P. Mortazavi; O-S Kwon; C. Christopoulos

Paper No.	Paper Title	Author(s)
P4-1	Recent Developments for Seismic Design, Retrofit, and Repair of Concentrically Braced Frames	A. Sen
P4-2	Overview of a Project on Resilience-based Design of Buildings with Accelerated Post-earthquake Functional Recovery	C.F. Mayorga; G. Tsampras; P. Bocchini; S. Mursel
P4-3	Conversion Project from Existing Supermarket to Town Hall	A. Matsuura
P4-4	Structural Design of High-Rise Building Aiming for Higher Business Continuity Performance	M. Kuwa; T. Shimizu; Y. Ohta; T. Kawamata; M. Tokuno
P5-1	New Guidelines for Post-earthquake Repair based on Performance-Critical Damage	A. Liel; K. Elwood
P5-2	Performance of Reinforced Concrete Buildings Damaged During the 2024 Noto Peninsula Earthquake in Japan	T. Obara; T. Mukai; M. Sakashita; H. Watanabe; A. Nakamura
P5-3	A Data-Driven Approach for Post-Earthquake Assessment of Seismic Damage in Woodframe Buildings	S. Ashikin; S. Muin; A. Iqbal; J. van de Lindt
P5-4	Risk and Resiliency Assessment in Practice: Assessment Taxonomy and Lessons Learned	B. Shao; I. Almufti; D. Zuloaga
P5-5	Improving the Seismic Resilience of New Zealand Hospital Buildings	D. Brunsdon; A. Thompson; J. Keen; M. Grant; P. Campbell; J. Stanway; C. Stevenson; N. Traylen; R. Wentz; I. Black; M. Stannard; L. Moon
P5-6	Structural Design and Construction of a Baseball Stadium with a Retractable Roof and a Seismic Isolation Roof	H. Kimura; K. Nagaya; T. Sato
P6-1	Tamachi Tower Adopting Innovative Damper Controlled Building System	Y. Hotta; S. Taniguchi; T. Yoshihara
P6-2	Detailing for Base Connection Ductility Courtesy of Replaceable Anchorage	M. Morano; T. Hutchinson
P6-3	E-Isolation– Further Development of Seismic Isolation and Response Control Technology	Y. Noguchi; Y. Takahashi; T. Takeuchi; S. Kishiki; M. Yoneda; Y. Shinozaki; A. Wada
P6-4	Lessons Learned from System-Level Response of Full-Scale Shake-Table Tests of Low Damage Buildings – Observations and Linkages to Component Test Results	G. Rodgers; R.S. Henry; A. Gu; Y. Lu; Q. Yang; Y. Zhou

Paper No.	Paper Title	Author(s)
P6-5	Resilient Design for Functional Recovery in the United States: Emergence of Elective Non-Prescriptive Resilient Design, and Current Efforts to Codify Prescriptive Requirements for Resilient Design	C. Haselton; E. Almeter; J. DeBock; D. Welch
P7-1	Quality Assurance of Nonstructural Components for Functional Recovery Design	J. Valigura
P7-2	Detailing for Seismically Resilient Steel Stair Systems: Validation in the Mass Timber (10 and 6-story) Programs	S. Sorosh; J. Zhang; T.C. Hutchinson; K.L. Ryan; K.W. Smith; A. Kovac; S. Pei; A.R. Barbosa; B. Simpson
P7-3	Seismic Performance Targets for Cladding and Curtain Wall Systems in San Francisco High-Rises: Lessons Learned Since Loma Prieta (2024 Update)	M.N. White
P7-4	New Zealand Code of Practice for the Seismic Performance of Non-Structural Elements	J. Stanway; A. Baird; S. Hinz; T. Sullivan; M. Rashid; G. Preston
P8-1	Resilience and Sustainability Considerations in Performance-Based Seismic Design of Structures	E. Opabola
P8-3	Driving Sustainable Design in the Structural Profession through Academic and Practitioner Collaboration	C. Toma
P8-4	Modeling Post-Earthquake Functional Recovery of Bridges: Framework Development and Application	C. Wu; H.V. Burton; Ádám Zsarnóczy; S. Chen; Y. Xie; V. Terzić; S. Günay; J. Padgett; M. Mieler; I. Almufti
P8-5	Quantitative Risk Analysis for Utilities	E. Jampole; G. Bhattacharjee
P8-6	An Integrated Resilience based Design Approach to Achieve Sustainability and Resilience	P. Brabhakaran
P9-1	Seismic Design of Low Rise Buildings in New Zealand	M. Grant; A. Hulsey
P9-2	The Salk Institute Laboratory A Performance-Based Design Case Study with FEMA P-58 Loss Modeling	D. Mar; A. Puah; S. Basereh
P9-3	National Activities to Improve Earthquake Resilience of Buildings	T. Mukai; H. Watanabe; M. Sakashita; A. Nakamura; T. Obara
P9-4	Functional Recovery Requirements for New Buildings in the NEHRP Provisions and ASCE/SEI 7	R. Zimmerman; C. Molina-Hutt; R. Kersting; A. Liel; J. Valigura; C. Haselton
P9-5	Performance Targets for Low Damage Seismic Design in New Zealand	T. Sullivan; S. Oliver; D. Brunsdon; C. Dong; T. Francis
P9-6	Development of Advanced Structural Health Monitoring System	S. Kimura; W. Sano; M. Sadamoto; J. Ono; Y. Tsuneki; K. Shintani

US, JAPAN AND NEW ZEALAND; EARTHQUAKE RESILIENCY AND FUNCTIONAL RECOVER CONCEPT DIFFERENCES

Dr. Kit Miyamoto¹; Dr. Amir Gilani²; Dr. Akira Wada³; Dr. Amir Moshref⁴

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Abstract

The seismic design of new buildings and retrofit of existing structures has been undergoing a significant shift in recent years. Due to social demand, computational capacity in practice and observations from the recent urban earthquakes in the US, Japan and New Zealand, concept of resiliency and functional recovery is a critical consideration for all three countries. However, its approaches are drastically different. In the US, adaptation of seismic protection devices is limited, and its performance is heavily relied on inelastic behavior of structural elements. This is largely driven by larger drift ratios allowed in the US compared with other countries and this higher threshold, in turn produces cost reduction of construction. In Japan, allowable drift ratios are usually half of US limits. This instigates the adaptation of energy dissipation devices and seismic isolators. In New Zealand, recent Christchurch and Kaikoura earthquakes caused interest and demand in functional recovery for both new and existing structures. Usage of advanced computational approach using FEM process and research grade testing provide realistic performance-based solutions. This approach overcame the cost issues in retrofit of existing structure for functional recovery. To address limitations of code approach in the US, the introduction of FEMA P-58 series launched the consideration of quantifying the expected performance of the building as whole (not limited to the structural system) as part of the decision-making process. The consideration of relevancy is expected to be codified in the next edition of ASCE by the incorporation of functional recovery—which is currently specified for risk category 4 buildings—for all structures and would be discussed for other risk categories in the future edition of US codes.

Introduction

In recent years significant improvements in the design of buildings have been implemented that has resulted in better seismic performance and enhanced resilience. Example applications in the US, Japan, and New Zealand are presented in the succeeding sections.

Practice in the US.

Introduction. In the US, the development of building codes and performance-based engineering has closely followed major earthquakes and knowledge gained after these events. For example, after the 1971 San Fernando Earthquake results to collapse and sever damage to the non-ductile reinforced concrete buildings; code requirements were updated to include provisions for ductile behavior of reinforced concrete members. And encourage reduction in structural irregularities. In the aftermath of the 1994 earthquake buildings that had not collapsed and had persevered life were nonetheless demolished to the high cost of repair, attention was directed toward performance-based design and the state of the structure after earthquakes; see Figure 1[1].



1971 San Fernando Earthquake



1994 Northridge Earthquake

Figure 1: Examples of structural damage in past US earthquakes

In the US, buildings designed using the requirements of modern building codes (Post UBC 1997) have performed well structurally. However, damage to the nonstructural elements in these buildings have resulted in loss of operation, business interruptions, and loss of revenues; see Figure 2 [2]



Damage to suspended ceilings



Damage to sprinkler system

Figure 2: damage to nonstructural components

The current edition of the building code in the US uses ASCE 7-22 [3] as its underlying standard. ASCE 7-22 is a reliability-based design providing approximate probability of failure for structures based on risk

category classification. For essential facilities (such as hospitals), the target reliability is 2.5% probability of failure for the MCE_R event (approximate return period of 2500 years). ASCE 7-22 writes:

Structural systems and members and connections thereof assigned to Risk Category IV shall be designed with reasonable probability to have adequate structural strength and stiffness to limit deflections, lateral drift, or other deformations such that their behavior would not prevent function of the facility immediately following any of the design-level environmental hazard events specified in this standard. Designated nonstructural systems and their attachment to the structure shall be designed with sufficient strength and stiffness such that their behavior would not prevent function immediately following any of the design-level environmental hazard events specified in this standard. Components of designated nonstructural systems shall be designed, qualified, or otherwise protected such

In other words, functionality requirements are specified for new construction in risk category IV buildings. In addition, the standard requires that the following buildings be classified as Risk category IV structures even if these buildings themselves are not essential facilities:

Buildings and other structures required to maintain the functionality of other Risk Category IV structures

Currently, the next edition of the ASCE 7 is under development. In this edition, functional recovery; see Figure 3 [4]-- safe occupancy, and restoration of building components and services to support a significant measure of pre-earthquake functionality--for buildings with lower risk category will be included as an option to encourage use of enhanced structural and nonstructural design.

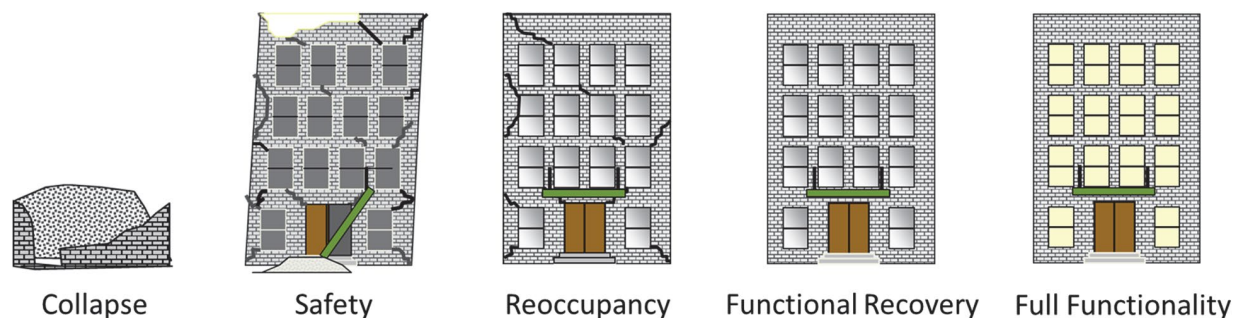


Figure 3: Illustration of functional recovery

Structural components. As discussed, in the US, a reliability-based design methodology is used for seismic action. The building codes provide prescriptive requirements for what is considered the minimum design. As illustrated in Figure 4., options such as use of enhanced design (e.g. specifying smaller drift ratio limits use of fiber-reinforced concrete) or seismic protect devices (dampers and isolators) can be utilized to improve the design reliability but with the drawback of modest to significant increase in the initial capital cost.. However, it should be noted that once user costs, reduction in downtime and repair costs are incorporated in a life-cycle cost analysis, the improved designs would likely become even more attractive and cost-effective.

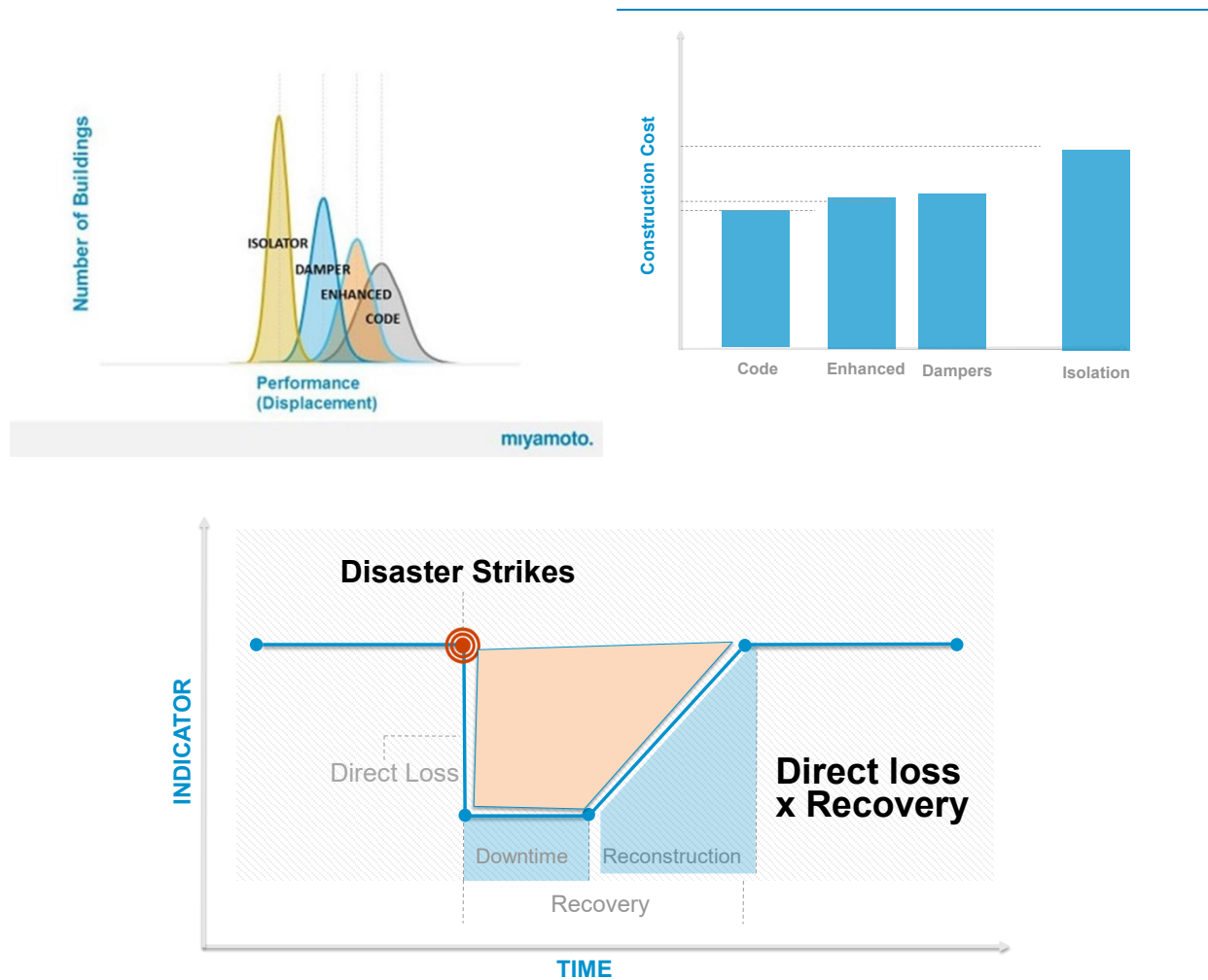


Figure 4: Various options for structural design

As an example of the use of supplementary viscous dampers to mitigate the vertical irregularity (soft story) for a building in Sacramento is illustrated here. The 14-story structure; see Figure 5, had a seismic resisting force system, of perimeter reinforced concrete moment frames. Due to the presence of soft story drift ratios at the lower levels were large and the structure would experience flexural hinges with large demand and shear hinging at the lower level. The use of dampers resulted in mitigation of soft story response and reduction of story drifts. Using FEMA P-48 methodology [5], significant reduction in the PML and probability of collapse were obtained [6]

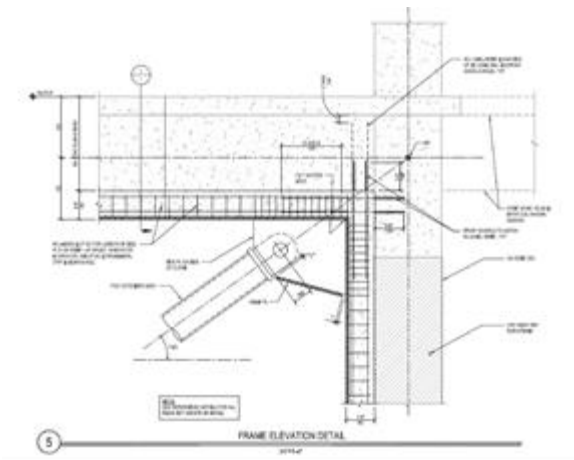
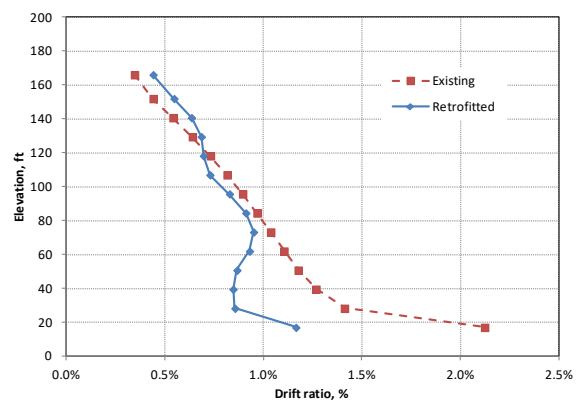
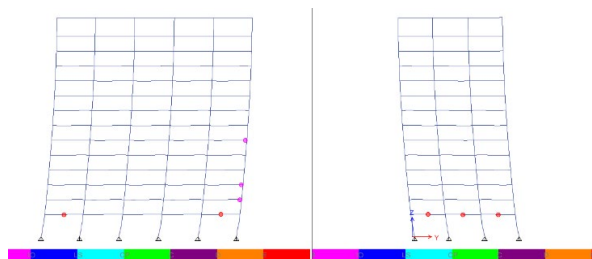
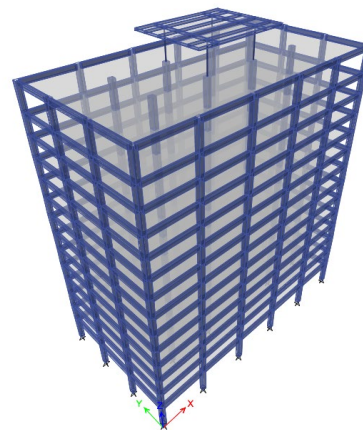
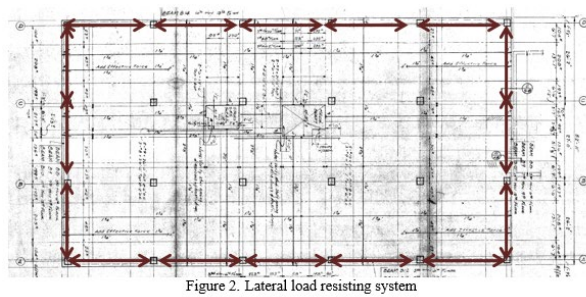


Figure 5: Improved design with seismic dampers

As an example of the use of fiber-reinforced concrete in construction of a new concrete shear wall building in a region of very high seismicity (including near-field effects) in the Los Angeles area is presented here. For the design of the multi-story structure see Figure 6, initially options of seismic isolation and energy dissipation were explored but were not considered further due to the initial cost consideration. However, because enhanced performance was desired, the use of laboratory-tested fiber reinforced concrete was further evaluated. The design resulted in acceptable performance of the walls with significantly reduced

damage. The design was further economized by reduction in the size of the walls that offset the cost associated with the use of fiber reinforcement [7].

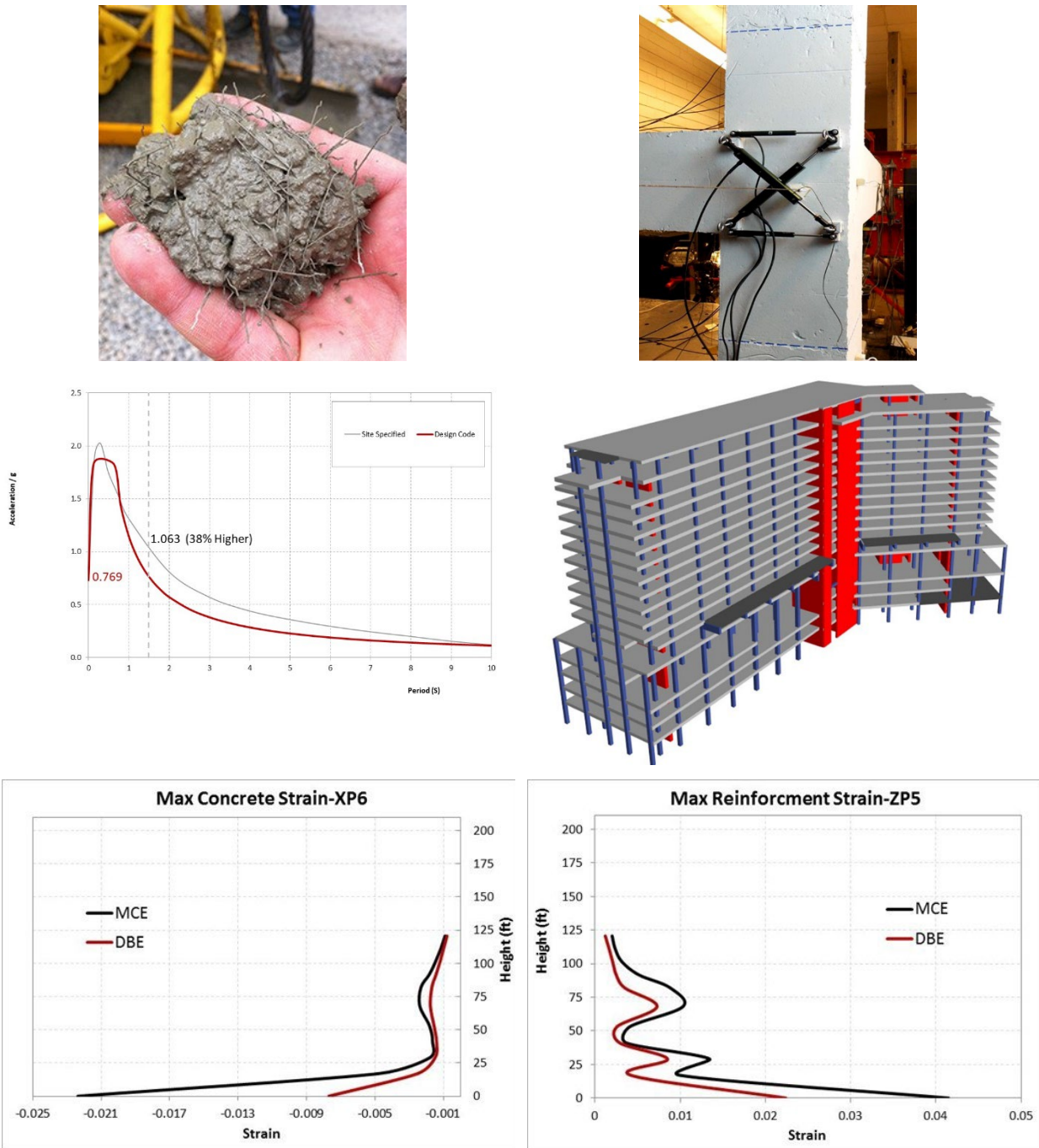


Figure 6: Improved design with steel fiber reinforced concrete

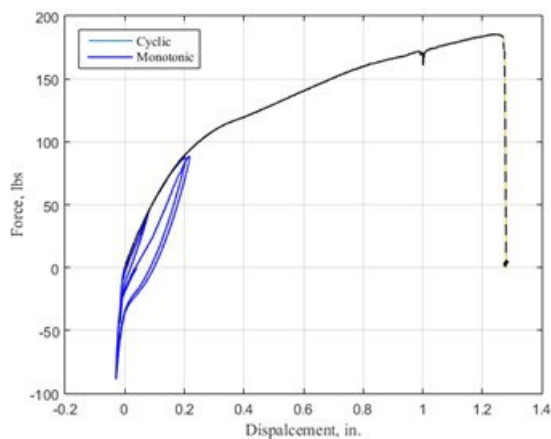
Nonstructural components. In recent years, significant improvements in the design and installation of nonstructural components have been made. For example, suspended ceilings have experienced significant damage worldwide in the past earthquakes. To address such vulnerability, manufacturers have innovated and developed new products and research centers have developed significant time in both shake table and quasi-static testing of ceiling components and systems. For example, [8] details testing of systems with various perimeter installation including at-the-time code prescribed pop rivets at the perimeter and the option of using seismic clips; see Figure 7. Based on these and similar tests, the seismic perimeter clips have now been adopted into building codes. For hospital buildings in California, seismic certification buy shake table testing of nonstructural components (e.g., generators chillers, etc.) are required. The equipment would have to have a tag confirming certification prior to installation.



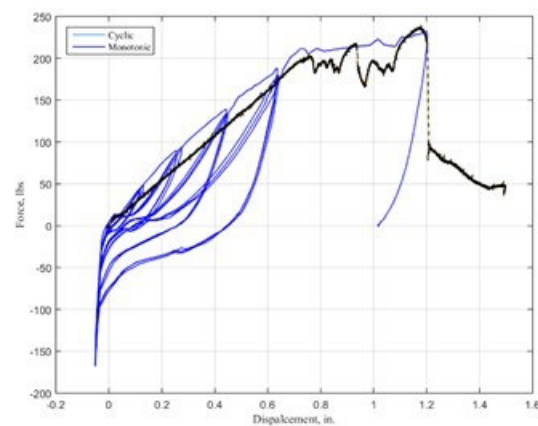
Cyclic test setup



Shake table test setup



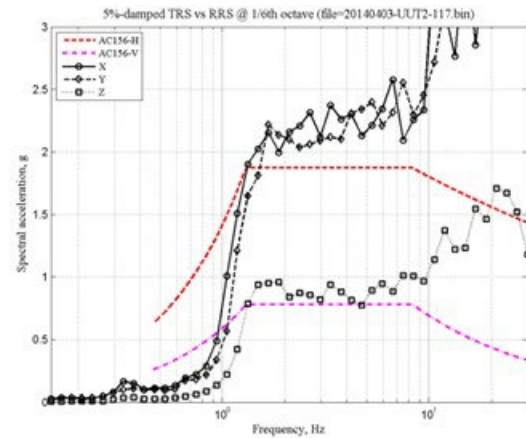
Pop rivet static results



Seismic clip static test results



Grid instrumentation



Target and test response spectra

Figure 7: Testing of various perimeter installations for suspended ceilings

In the aftermath of the 2014 Earthquake in Napa, California several fire sprinkler systems failed [8]. For a particular parking structure, ambient studies were conducted and the fundamental frequency of the slabs supporting the sprinkler system were determined; see Figure 8. Based on the results of ambient surveys and scans, a finite element model of the pipe was prepared and subjected to the acceleration data from the recorded strong motion data recorded during the earthquake to determine the level of stress in the failed locations.

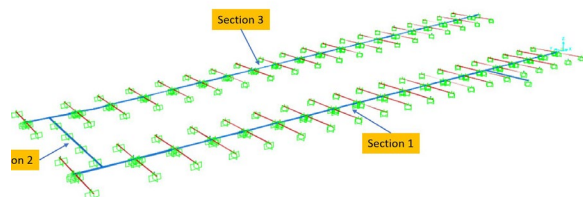
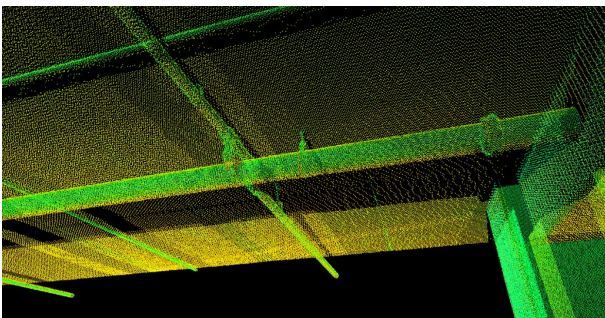
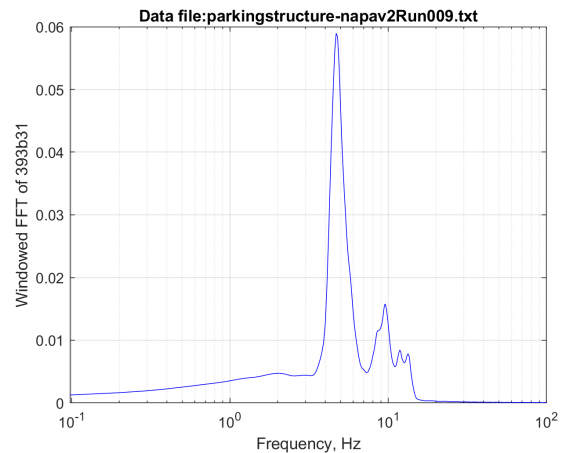


Figure 8: Research into performance of sprinkler system

Practice in Japan

Life spans of buildings versus those of cities Devastating earthquakes are very rare for individual buildings. The seismic design of a building starts from setting up a target performance under certain levels of earthquakes. Although there is no agreement among the seismologists, earthquake engineers decided to consider a possible maximum earthquake that will occur in a place or a location once in 2500 years from our engineering judgments. Accordingly, it seems to have become a consensus of defining the performance objectives for urban buildings and infrastructure at three levels of return periods of earthquakes, that is, once in 50 years, once in 500 years, and once in 2500 years. It approximately corresponds to a 63%, 10%, and 2% probability of exceedance in a 50-year period of time, which matches the specified life cycle of a building in cities. Although different names are used, the seismic codes around the world share the same definitions of the earthquake levels in their seismic design of buildings, such as the design-basis earthquakes (DBE) and maximum credible earthquakes (MCE) in the US [10], the Level 1 and Level 2 earthquakes in Japan [11] and the frequent, design, and rare earthquakes in China [12]. An earthquake stronger than the maximum considered earthquake, i.e., a once-in-2500-year earthquake, is assumed to be beyond the reach of engineering efforts. Once such a natural disaster happens, it is essential for the world to come together and help one another.

In addition to the consensus of earthquake levels for seismic design, relevant people including the government, researchers, engineers, building owners around the world also come to agree with the seismic design objectives at the three levels of earthquakes as follows.

- (1) In once-in-50-year earthquakes (63% in 50 yrs), building structures should remain essentially elastic to minimize or eliminate damage.
- (2) In once-in-500-year earthquakes (10% in 50 yrs), building structures could sustain plastic deformation in a controlled manner, and therefore exhibit some extent of ductility to withstand the earthquakes. Live safety of the people should be securely guaranteed, but many buildings are expected to be demolished after the earthquake because of their structural damage.
- (3) In once-in-2500-year earthquakes (2% in 50 yrs), some buildings may collapse and kill many people. The city may be ruined with the many collapsed and heavily damaged buildings and infrastructure, waiting for the outside world to come to rescue.

It is intuitive to relate the seismic design of buildings with the life span of the buildings themselves, which is usually assumed to be 50 years. However, buildings are the building blocks of our civilization, and therefore cannot be separated from the seismic safety of our civilization. Instead, we must acknowledge that the life span of great cities is much longer than its building blocks. The Chinese civilization started in 5000 BC, the Mesopotamian civilization started in 3000 BC, the Egyptian civilization started in 3000 BC, and the Indus civilization started in 2500 BC. Buildings in a city may appear and disappear in several tens of years, but the city remains for hundreds and thousands of years.

Professor Yukio Tamura and his colleagues in Chongqing University [13][14] illustrate the relationship between short-life temporary structures and the longer-life city block in Figure 9. They argued that “If a strong earthquake or a strong typhoon attacks this city, all buildings, including general buildings (white blocks) and scaffolding or construction work offices (green blocks) would experience the same level of seismic load or wind load. There is no difference between the permanent and temporary blocks in terms of the existing period and the external environment.”[13] Therefore, they believe that the design wind load for the so-called temporary structures should be exactly the same as the permanent structures.

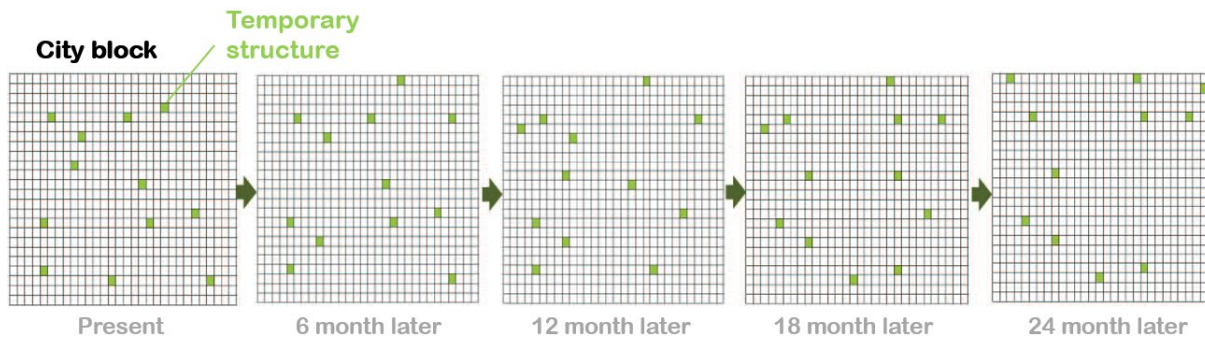


Figure 9: Schematic diagram of movement of scaffolding or construction work offices and general buildings in a city model (every 6 months snapshots) (Tamura et al, 2018^[13]).

The same idea applies to earthquake engineering. It is short-sighted for building owners and structural engineers to decide to design the buildings according to the building's own life span, but overlook much longer life spans of cities, from where the buildings cannot be cut off. It is not enough to consider a building itself you are designing, but you must imagine a long-life city beyond the building.

Design for the safety of cars versus for that of cities Automobiles are designed to protect the driver's and passengers' life safety in car accidents. For example, the engine room in front of a car is intended to crush to buffer the impact and dissipate energy in a car accident (Figure 10). This is similar to the idea of protecting life safety in buildings by means of structural ductility, which is, in other words, damage. After a car accident, the cars are often so badly damaged that they cannot be used or repaired anymore. In most cases, the insurance companies will pay for the loss of such unfortunate accidents. Such a system runs well around the world.



Figure 10: Research into performance of sprinkler system Crash safety tests of a car

However, if the same idea is followed for the seismic design of urban buildings, it will be a different story. The consequence of major earthquakes is not limited to a single or several buildings like in a car accident. Urban communities are composed of many individual houses and buildings. The difference is that it is zero that tens of thousands of cars are broken by car accidents at the same time in a city, but it is very common for tens of thousands of buildings in the city to be damaged or even collapse during a single big earthquake (Figure 11). When the life of a building constructed in a place is 50 years as ductile structures, the next building having 50 years of life will be constructed in the same place. The same methods will be repeated in the next 50 years and the future. After these steps, there should be 50 buildings having the same poor seismic performance in 2500 years in the city. The next big earthquake will cause severe damage to many buildings, and almost all buildings cannot be used after the big earthquake. There is so much evidence in Japan, China, Italy, Türkiye, Haiti, and so on in the 21st century.

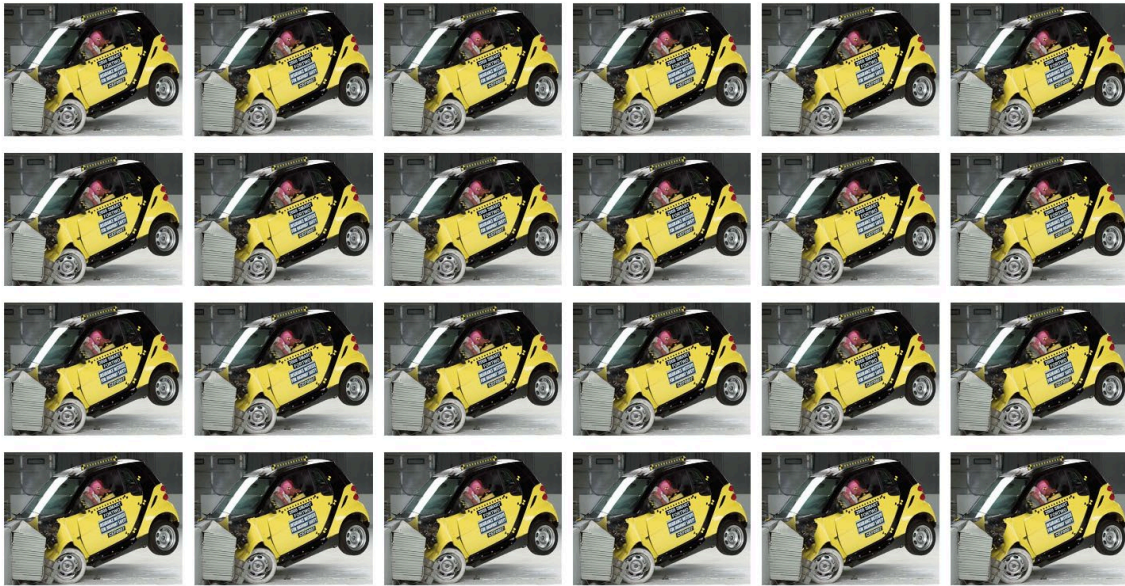
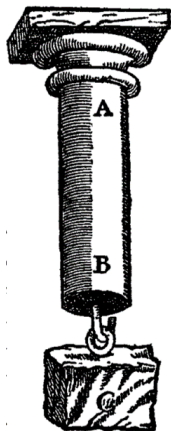


Figure 11: Crash safety tests of a car City attacked by a major earthquake is like a city with so many crashed cars.

Ductility is damage. The Great Galileo Galilei published his last major work, “*Two New Sciences*” in 1638. Following this pioneering work, many researchers and engineers discussed about structural mechanics, structural engineering, and earthquake engineering in the past 400 years. This book not only laid the foundation of classical mechanics, but also enhanced the fundamental idea of modern science to check hypotheses and ideas against experimental evidences in the real world. After witnessing the 1908 Messina earthquake in Sicily, Italy, Professor Silvio Canevazzi and his committee first proposed the idea of seismic design of structures by introducing “shear coefficients” in the world. Such a strength-based seismic design methodology later evolved to include ductility as another important performance indicator for building structures. However, considering the life span of cities, where most of us live and work, earthquake engineers must change their minds about the seismic design of structures and understand that “ductility is damage”, a concept attributed to Professor Vitelmo V. Bertero, the University of California, Berkeley, said. In his opinion, the concept of ductile structures is an old idea. If we hope our society and countries remain for another 100,000 years or more, we must build our cities with long lifespans in mind.



Silvio Canevazzi

16 Marzo 1852 - 13 Marzo 1918

Higher seismic performance for the sake of resilient cities. Buildings equipped with seismic isolation and/or supplemental damping devices have shown higher seismic performance in recent earthquakes. These types of structural systems and devices are not expensive anymore, allowing us to design and construct buildings that can remain operational under once-in-500-year earthquakes at an affordable cost.

In 2013, the Turkish government mandated all new hospitals of 100 beds or more in seismic regions to adopt seismic isolation [15]. The many newly built seismically isolated hospital buildings outperformed their non-isolated counterparts and played a key role of maintaining the function of the local medical care systems in the 2023 Turkey earthquake doublets (Figure 12)[16].

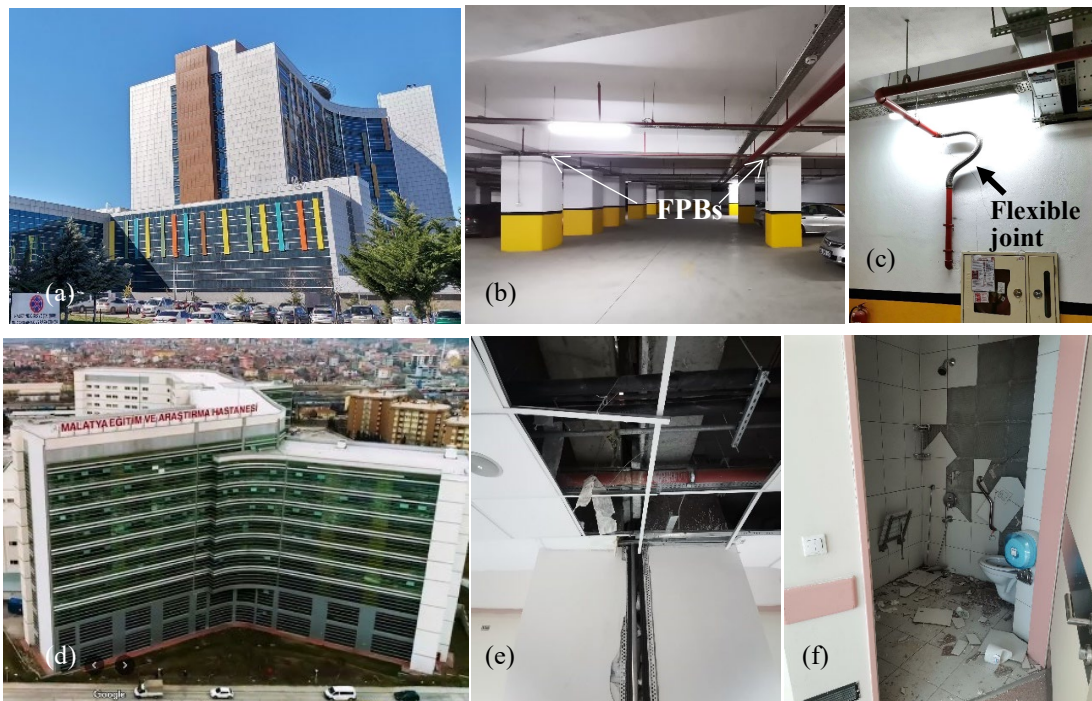


Figure 12: Base-isolated and non-isolated hospital buildings in the same campus in the 2023 Turkey earthquake doublets: (a) base-isolated Malatya Maternity and Children's Hospital, (b) friction pendulum bearings in the isolated hospital, (c) flexible joints for piping; and (d) non-isolated Malatya Education and Research Hospital, (e) damage near the expansion joints and (f) falling tiles of a toilet.

A similar regulation was enforced by the Chinese government in 2021. It mandates the use of seismic isolation and/or supplemental damping systems in newly-built key facilities, such as schools, kindergartens, hospitals, day-care centers, emergency response centers, emergency shelters, and radio and television stations, in earthquake prone regions so that they can properly function under design basis earthquakes [17]. This regulation has greatly promoted the implementation of base isolation and supplemental damping systems in China.

Practice in New Zealand

Introduction. The 2010 Canterbury earthquake (magnitude 7.1) and the 2011 Christchurch aftershock (magnitude 6.3) caused widespread destruction, resulting in 185 deaths and significant damage to housing and commercial buildings. Nearly three-quarters of the region's homes were affected [18] with 9,100 rendered uninhabitable[19], and total residential property repair costs estimated at NZ\$13 billion. Insured losses approximately NZ\$35 billion, with over 70% of these losses covered by offshore reinsurers—equivalent to more than 100 years of earthquake premiums[18] . Christchurch's central business district (CBD) also suffered extensive damage, with 47% of buildings deemed unsafe or requiring restricted access, and over 600 commercial buildings were demolished [20]. The 2016 Kaikōura earthquake (magnitude 7.8) caused more than NZ\$1 billion in damage to Wellington, with over 90% of the losses related to commercial properties. Some newly constructed, code-compliant buildings were deemed total losses, leading to significant insurance payouts and a 300% rise in premiums[21].

In 2006, The New Building Standard (%NBS) index was developed, to identify buildings that pose a higher seismic risk compared to newly constructed buildings. Following the Canterbury earthquakes, a higher NBS rating became a common requirement in commercial tenancy contracts, a trend reinforced after the Kaikōura earthquake. Many companies now require a minimum of 70% NBS for leasing office spaces as part of their health and safety policies. As a result, NBS ratings have become a critical factor in leasing advertisements.

Recent seismic events have also revealed vulnerabilities in widely used structural components in New Zealand, such as precast floor units. Research and observations prompted updates to the seismic assessment guidelines, which are now more stringent, particularly for buildings with precast floors. These updates have led to a significant reduction in the NBS ratings of many buildings, affecting their commercial viability. Recent seismic events have also exposed vulnerabilities in common structural components, such as precast floor units. Updates to seismic assessment guidelines, particularly for buildings with precast floors, have led to a significant reduction in NBS ratings and affected commercial viability. The combination of increasing insurance premiums and the requirement for a minimum 70% NBS rating has driven building owners to strengthen their properties. However, finding commercially viable solutions using traditional methods has become increasingly challenging in the post-pandemic economic climate.

Tailored Solutions Through Research-Level Testing and Modeling. The New Zealand Seismic Assessment of Existing Buildings [22] provides guidelines for evaluating the seismic performance of existing structures. While the available experimental data for most structural components are extensive, limited information is available for certain elements. This gap necessitates experimental testing and advanced modeling to develop cost-effective solutions. The use of observational data and testing in design is permitted under AS/NZS 1170.0:2002 [23], which facilitates compliance checks for territorial authorities.

Beam-column joints play a crucial role in reinforced concrete structures under seismic loads. Current seismic codes prioritize joint shear capacity and mandate lateral reinforcement for stability. However, many older buildings do not meet these modern standards, making it difficult to assess joints with deficiencies such as inadequate rebar, shear strength, and confinement. These challenges are particularly pronounced in corner or perimeter beam-column joints that feature plain bars and hooks at the ends, as is shown in *Figure 13*. NZSEE C5 [23] suggests decreasing shear capacity of existing beam-column joints reinforced by this anchorage detail based on curvature ductility demand, as illustrated in *Figure 14* :

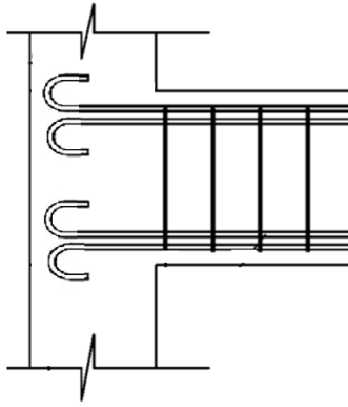


Figure 13: Common detail of unreinforced beam-column joint with plain bars

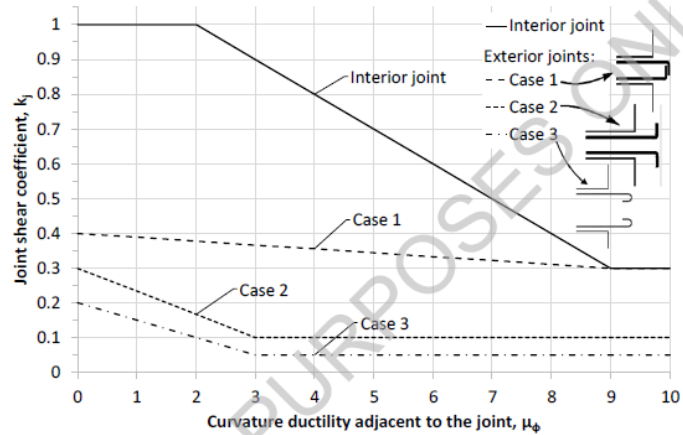


Figure 14: Joint shear coefficient [23]

Figure 14 reveals that the value of joint shear coefficient tend to 0.05 at curvature ductility demand of 3. This means the shear capacity of case 3 joints significantly decreases at curvature ductility demand of larger than 3. NZSEE C5 similarly recommends a reduction factor of 0.7 on shear capacity to account for bidirectional effects in the absence of further studies. Finding economically viable strengthening options for existing buildings with these types of beam-column joints is particularly challenging, and the difficulty increases when dealing with heritage buildings.

To overcome this issue bi-directional cyclic tests were conducted on full scaled beam-column joint specimens from a building in Wellington. The details of the specimens and loading protocol are provided in [24]. The hysteresis behavior of the reference specimen (RS), along with the estimated shear capacity from the NZSEE guidelines (without applying the reduction factor for concurrent loading), is shown in Figure 15a. The results indicate that the proposed drop in joint shear coefficient as well as 30% reduction in shear strength under bidirectional loading is overly conservative for this case.

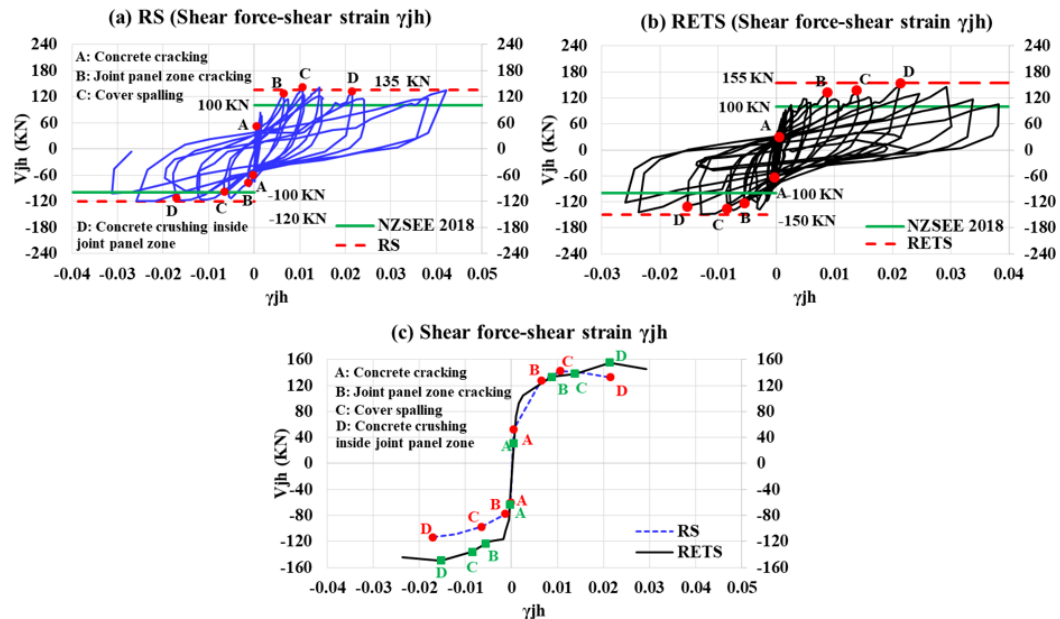


Figure 15: Response of beam-column joint a) Reference Specimen b) Retrofitted Specimen c) back-bone curves

Following this, an innovative retrofitting solution using diagonal reinforcement and additional "L"-shaped bars, as shown in *Figure 16.*, was proposed and tested for comparison with the reference specimen. As demonstrated in the *Figure 15b* and *c*, the proposed strengthening method increased joint shear capacity by 14-24% and delayed damage initiation. These results confirm the effectiveness of the proposed retrofit in increasing shear capacity and delaying damage, making it a viable cost-effective solution for older buildings with deficient joints.

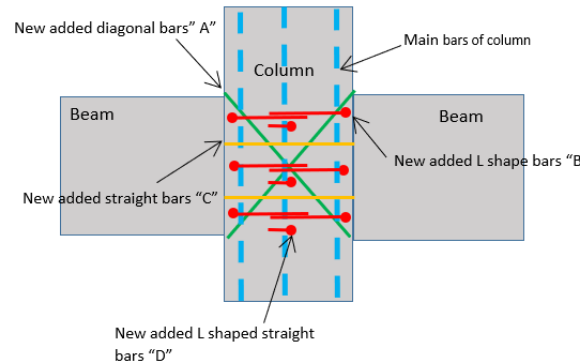


Figure 16: Schematic retrofit method

Another case study demonstrating how advanced analysis and experimental testing can lead to viable strengthening solutions is the Morrison Kent House in Wellington. This building presents a unique challenge due to its use of concrete-encased steel lattice frames, a structural system less studied compared to traditional concrete-encased steel columns. Understanding the performance of these frames under seismic loads is crucial for developing effective strengthening strategies. Morrison Kent House is a 23-storey building, designed and constructed in the 1960s, featuring a 16-storey tower supported by a 6-storey podium. The lateral load-resisting system consists of concrete-encased, structurally braced steel frames, as shown *Figure 17*.

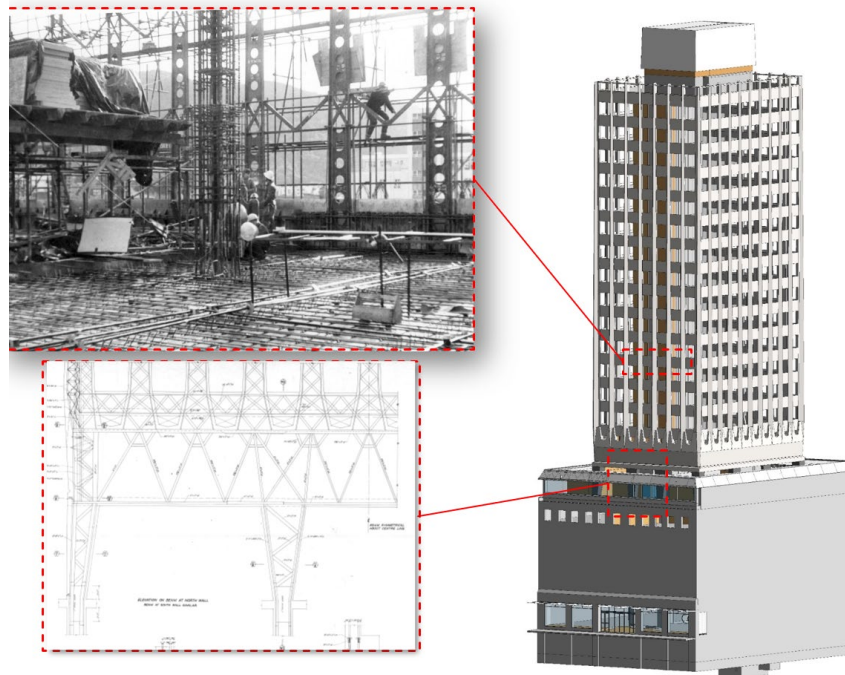


Figure 17: The Morrison Kent House building in Wellington

Composite concrete-encased steel sections are widely used in high-rise buildings due to their combined strength and material efficiency. However, the behavior of such sections under seismic loads, particularly when concrete encases a lattice structure rather than individual steel columns, remains less understood. Shear transfer between the concrete and steel is crucial for seismic performance, and insufficient research has been conducted to fully understand these mechanisms in lattice structures. The shear transfer mechanism concrete-encased lattice structures may differ from concrete-encased steel section, as it can occur both through nodal zones and via chemical bonding. To gain a deeper understanding of the building's structural performance and to identify a cost-effective strengthening solution, cyclic tests were conducted on four half-scale specimens, as presented [25]. The lateral force versus top displacement (drift) curve for corner column specimen above the podium is shown in Figure 18.

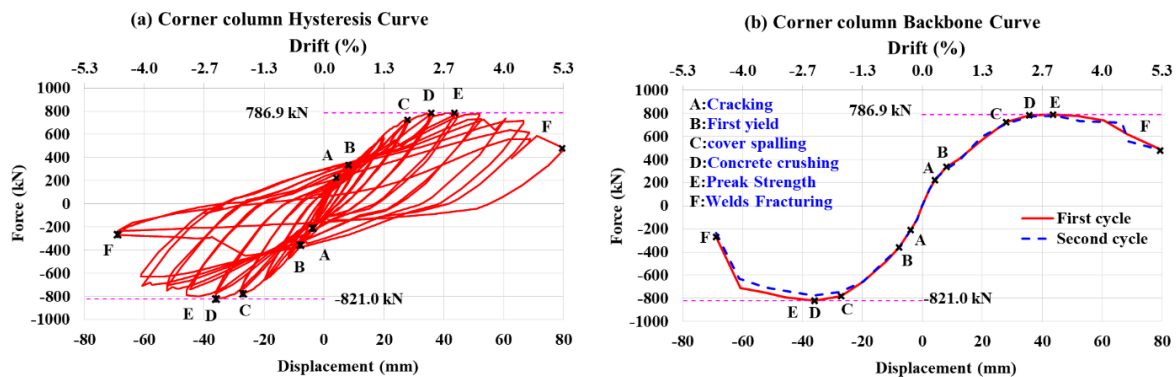


Figure 18: Lateral force-top displacement for the corner columns

Using these testing it has been shown that the structural components have a reasonable deformation capacity and energy dissipation. Therefore, the scope of strengthening works can be significantly reduced.

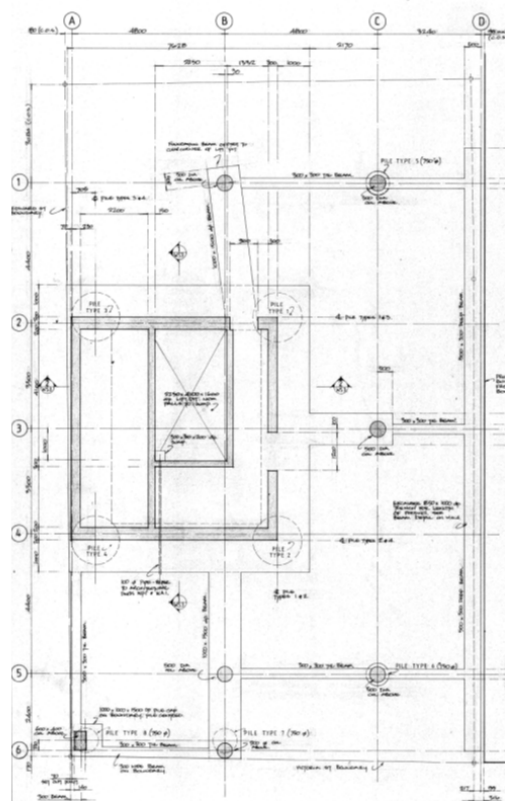
Controlled Rocking Mechanism Many new and existing buildings do not have enough self-weight to counteract the overturning forces generated by earthquakes without experiencing uplift in structural elements. While tension piles can be used to prevent uplift, they come with significant cost increases and may introduce higher loads to the structure. However, both observational and analytical evidence suggests that localized uplift and rocking do not necessarily hinder seismic performance; in fact, they may help reduce the forces transmitted to the structure [26],

NZS 1170.5:2004 addresses the issue of rocking structures in Section 6.6, which mandates that when energy dissipation occurs through rocking, a special study must be conducted. This study involves developing a detailed computer model of the structure and performing a time-history analysis to evaluate its response to potential earthquake scenarios.

The main challenge with rocking mechanisms in tall buildings is managing the bearing capacity at the base and the P-Delta effect, which can lead to collapse. Controlled rocking mechanisms, however, can mitigate this issue. This approach was used in an 18-storey shear-core wall building in Wellington, supported by four bored piles 8 meters deep. The building begins to rock at 40% of the Design-Based Earthquake (DBE) demand. Traditionally, additional piles would be added to prevent rocking, but the limited footprint of the building (260m²) restricted this option. Instead, a cost-effective solution was developed by leveraging the advantages of rocking.



3-D view from Google Earth



b) Foundation Plan

Figure 19: Case study 3: 18-storey building in Wellington.

With a height-to-width aspect ratio of seven, the bearing capacity of the piles is quickly reached after rocking begins, and beyond a certain displacement, the P-Delta effect leads to collapse. To address this, micropiles with specialized detailing at the top were introduced, as is shown in Figure 20, engaging the new micropiles under specific rotational demands on the foundation. The efficiency of this proposed system was demonstrated through nonlinear time-history analysis using OpenSees, incorporating soil-structure interaction.

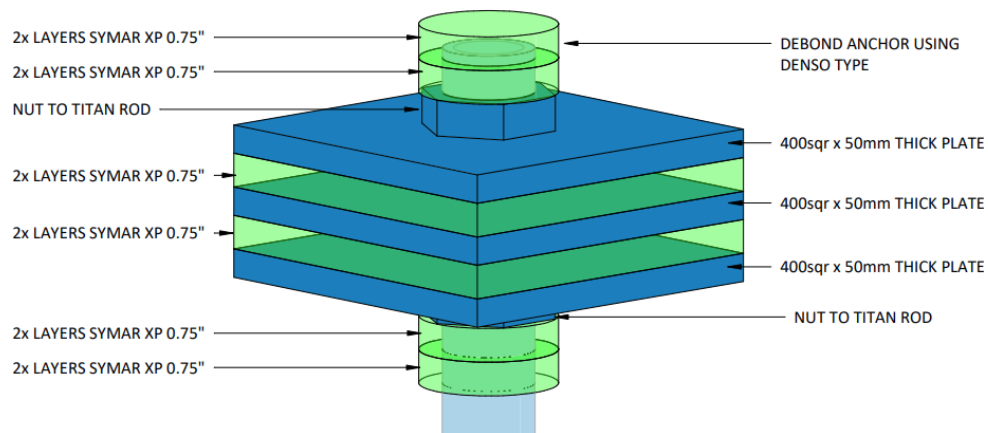


Figure 20: Innovative detailing at the top of the micropiles was implemented to delay their engagement

Discussions

In the US, recent advances in research and testing, analysis, and dissemination of data from earthquakes and strong motion records have made the enhanced seismic design more economical. The improved design have a super performance and expected to sustain less damage and downtime in the event of earthquake. In many instances the additional initial cost is at least partially offset by savings in concrete and steel reinforcement reduction. The upcoming edition of the US code will emphasize the option of incorporating functional recovery in the seismic design of new buildings.

In Japan, with the advances in earthquake engineering, we can our cities withstand once-in-500-year earthquakes without any suspension of the normal activities in the city, where no building collapses or needs repair. It is also possible to make structures that would kill no people under a once-in-2500-year earthquake. The widespread application of such structures in buildings and infrastructure in cities will lead to resilient cities. In this manner, our cities will become a safer place against future earthquakes.

In New Zealand, the need for resilient structures in major cities is more pressing than ever. However, transforming the stock of existing buildings into resilient structures poses significant challenges, particularly due to financial constraints. Any effort to build a resilient society must take economic factors into account. In this context, investing time and resources into comprehensive analyses, supported by experimental testing, to optimize the scope of strengthening work appears to be one of the most practical approaches to achieving this goal

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RETHINKING IMPORTANCE LEVELS FOR SEISMIC DESIGN BASED ON SOCIETAL EXPECTATIONS

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Abstract — Recently completed research identified a snapshot of societal expectations for how New Zealanders want buildings to perform during and after an earthquake. The research findings showed that while life safety was of primary importance, people expected more than protection from injury, and identified that social and economic recovery were very important objectives. Protection of function and speed of recovery was identified as a particular priority for some building types, including supermarkets, community centers and homes – that are currently not a priority in the Building Code.

This research was completed to inform the Resilient Buildings project, a project initiated and led by the New Zealand Society for Earthquake Engineering (NZSEE) aimed at developing a framework relating societal expectations to key aspects of building performance. The aim of the project was to inform future development of seismic loadings codes in New Zealand and to facilitate consideration of potentially higher preferred levels of seismic performance for new buildings by owners and investors.

This paper explores what the social research revealed about people's expectations for different building types during and after earthquakes; *why* certain buildings were valued by their communities and the *consequences* that would result from these important buildings being damaged.

The different building usage categories identified through the research are then related to different aspects of building performance in a systematic way informed by these societal preferences. The building usage categorization system thus rethinks the current Importance Level system¹ currently used in the New Zealand loadings Standard, NZS1170 (2002), to identify certain buildings for enhanced performance compared with “normal” buildings.

Introduction

The earthquakes that occurred in New Zealand (NZ) over the past fifteen years caused many and varied impacts. These ranged from mass casualties, costly repairs and slow complex recoveries with many unforeseen effects, Potter et al (2015), to significant impact to the social fabric of New Zealand communities, associated economic impacts and significant environmental effects, Stevenson et al (2016), Fleisher (2019), Parker and Steenkamp (2012), Environment Canterbury (2013).

These impacts were widely observed and criticized by the community. The Canterbury Royal Commission enquiry, (CERC 2012) recommended that the treatment of seismic risk in NZ building standards should be reviewed to ensure the current knowledge of seismic hazard and risk are broadly aligned with societal expectations of building performance in earthquakes. The New Zealand Society for Earthquake Engineering (NZSEE) initiated the Resilient Buildings Project with funding from Toka Tū

¹ Closely aligned with the building usage categories identified by importance factors in ASCE/SEI 7-22,

Ake EQC in response to these observations and recommendations with the aim of researching contemporary New Zealand building users' expectations of seismic performance and relating these findings to building performance in order to inform the development of future design practice and New Zealand seismic loadings Standards.

Key findings included the priorities people placed on different building types and usages, their expectations for earthquake performance and how this varies from current code settings in New Zealand. This paper first explores the societal expectations research findings for different building types and usages and then presents a building usage categorization system developed based on these findings. A categorization system that is directly related to different aspects of building performance so can be used for the subsequent setting of building design objectives and loading Standards.

Societal Expectations Research

The research entailed a series of interviews and focus groups with diverse stakeholders across New Zealand to sample perspectives on the desired future performance of buildings. It involved 59 individuals and a total of 140 hours of face-to-face engagement. The focus was on exploring expectations for future potential earthquake events and included consideration of risk tolerance of different size earthquakes from rare significant events (less than once every 2500 years) to small events occurring at least once in a typical building's life (once every 0 - 50 years).

Participants were deliberately chosen to span and represent a wide range of building user groups, industries, and interests, different socioeconomic groups and cultural contexts, public and private sector, rural and urban context, low and high seismic hazard zones. The interviews focused on understanding each participant's current role, background, and earthquake experience and their expectations of new building performance during a significant earthquake and during minor to moderate earthquakes. The geographically based focus groups covered three urban centers and three smaller towns with differing levels of seismic hazard. The focus groups each comprised individuals representing different community perspectives (local civil defence, business community, health sector, welfare sector, environmental interests, and indigenous Māori).

What Do New Zealanders Want from their Buildings in Earthquakes?

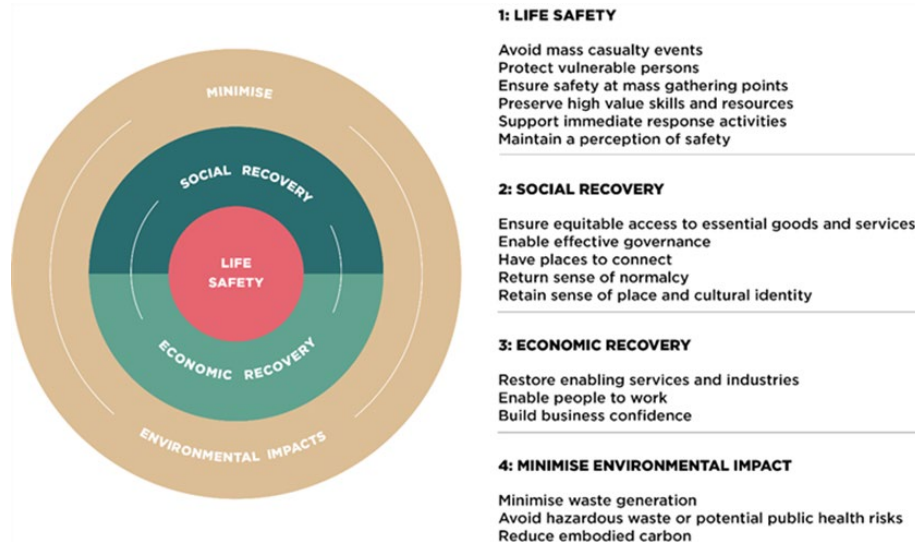


Figure 1. Seismic resilience performance expectations

Life safety. The research findings showed that life safety was of primary importance for buildings in earthquakes. This aligns with New Zealand’s current seismic code focus. The research though found the priorities for life safety are not simply about the number of people occupying a building but include consideration of the types of individuals likely to be in the building. Participants identified vulnerable people and those with essential skills for response and recovery (including economic recovery) as requiring enhanced protection. The research participants also identified the importance of avoiding the potential for mass casualty events.

Participants agreed that prioritizing buildings with post-disaster functions, such as hospitals, was important (as is provided for in current seismic settings in New Zealand) but considered priority should be extended to more building types/usages, including supermarkets and food production facilities as well as multi-purpose spaces that can be used to support disaster recovery. Locations likely to experience or attract large numbers of people immediately after an earthquake, such as schools and community centers, were also identified as locations where higher building performance is expected due to the danger of aftershocks.

The research findings were compiled into a research report. Brown et al (2022), a detailed report on interviews, Abeling et al (2022) and a detailed report on focus groups Horsfall et al (2022).

Earthquake recovery. The findings identified that New Zealanders consider a focus on life safety is not sufficient, with social and economic recovery identified as important objectives following an earthquake. This is consistent with years of public discourse and judicial rulings on recovery processes including the settlement of insurance claims, and the enormity of financial losses overall following the Canterbury Earthquake sequence, CERC (2012). Access to essential goods and services, sustaining social connection and restoring normalcy that supports cultural values and economic wellbeing were all noted as important for social and economic recovery.

The research identified safe housing and confidence and certainty in the recovery process as core expectations of the period immediately following a major earthquake. This framing of expectations included prioritizing mental wellbeing and enabling conditions for individuals to contribute to the social and economic recovery of their community. Participants considered the ability to shelter in place in their homes important and desired an early return of electricity, internet, and telecommunications even if water and sanitary systems were not functional. This expectation is based on the Christchurch experience after 2010, where many people continued to live in their damaged single-family homes throughout that 14-month-long earthquake sequence, despite not having access to indoor plumbing. This expectation for sheltering in place was corroborated by recent experiences during the COVID-19 pandemic.

The research highlighted the very strong synergies between economic and social recovery and that many buildings support both. The need for households to generate income soon after an earthquake was identified as important, along with schools reopening quickly, both to enable parents to return to work and to reduce stress on families. Schools also provide a vital function in assisting students in regaining a sense of normality by attending class and seeing their peers, thus contributing to mental wellbeing. “Returning to normal” is considered a critical factor for recovery. While “normalcy” may look different for different communities, reopening schools, retail and arts and recreation facilities and access to buildings that support cultural wellbeing and identity were noted as key parts of the return to normalcy.

Speed of recovery and levels of damage. Continued building functionality is expected for critical infrastructure including buildings such as hospitals and emergency services facilities with post disaster functions. This expectation is consistent with current NZ building code settings for such facilities.

Findings from the study showed the expectations for restoring other buildings to function after an earthquake vary with building type. Speed of return to function was identified as a particular priority for some building types that are not currently specified, including supermarkets, aged care facilities, community meeting places and homes. The expectation for return to function for these buildings is days and weeks not months and years

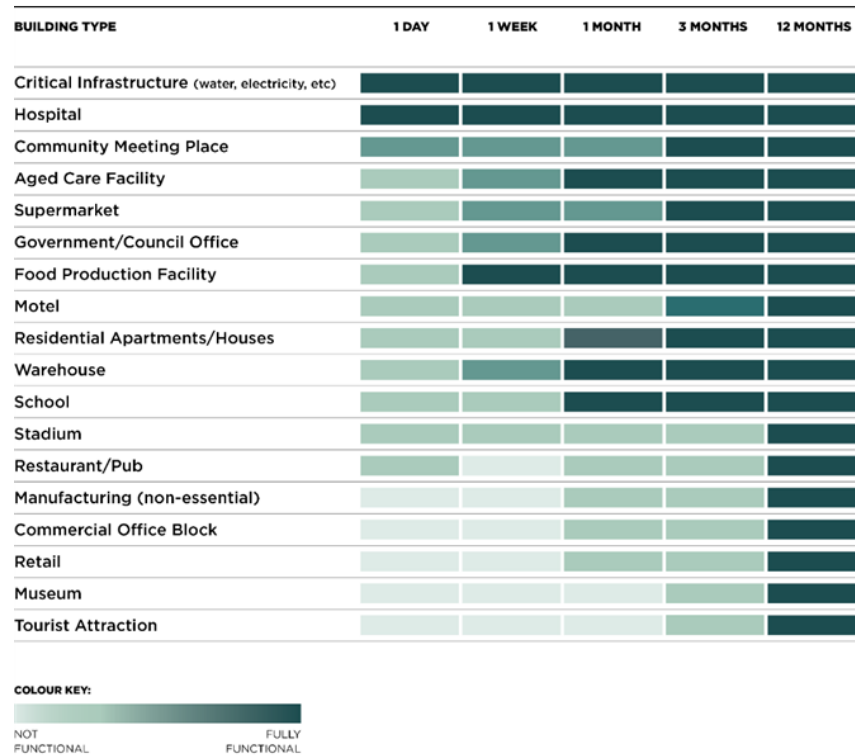


Figure 2. Time to restore function

What about moderate earthquakes? The research shows that in addition to an expectation of safety in moderate earthquakes people expect minimal or no impact on building functions, therefore requiring very limited damage to the non-structural elements that support building uses. In homes, the expectation is that kitchens and bathrooms should remain usable. In offices, building systems (e.g., HVAC systems, and emergency systems such as fire protection systems) should continue to work uninterrupted. Buildings are expected to remain watertight. It is typically expected that the building contents will have moved around and there may be some cosmetic cracking (e.g., cracks in plasterboard). However, any damage should be non-structural and both minor in nature and limited in extent such that it remains easily repairable while the building remains occupied.

Priorities Attributed to Different Building Types and Usages

The societal expectations research clearly showed some building usages are more valued than others. The reasons why certain buildings were important varied; some buildings were seen as important because they are likely to house vulnerable people for example, while others, because they enabled society to function normally. In general, buildings were viewed as less important if there were readily available alternatives to their use e.g. working from home or redundancy in building function. The importance of buildings also differed depending on the geographical and community context. Buildings located along major arterial routes, for example, were identified as having the potential to cause acute consequences for impeding post disaster response and recovery if they were to be damaged in an earthquake. Expectations

for building performance differed between urban and rural settings. Some rural communities had strong social and economic ties to a particular business or primary industry processing plant. In urban areas some people would prefer enhanced building seismic performance given the concentration of risk. For example disruption to medium and high-density housing could create a resident displacement challenge post-earthquake significantly impacting community wellbeing.

Building Usage Categorization System Developed Based on the Societal Expectations Research

A consistent approach was used to identify *why* certain buildings are valued by their communities and the *consequences* that would result from these important buildings being damaged. The reasons why certain building are valued were mapped to three dimensions of building performance: protection from injury, protection of property and protection of amenity and function. These three dimensions were identified as part complementary work (see Abeling et al 2024) and closely align to the three design limit states (ULS, SLS1 and SLS2²) in the current New Zealand loadings Standard, NZS1170, Standards New Zealand (2002).

The vision for the building usage categorization system is that each building usage group is considered across all three dimensions of building performance separately and so can have performance outcomes, and subsequently, performance objectives, independently set for each building dimension. That way the specific aspect(s) of building performance identified as particularly important can have higher performance objectives set for those specific aspects. For example, a large stadium might need enhanced seismic performance for life safety given the large crowds but have few enhanced requirements for damage prevention or for ensuring ongoing functionality. A supermarket may need enhanced seismic performance to ensure ongoing functionality but not necessarily enhanced protection for life safety compared with a “typical” building.

This approach contrasts with the current NZ loading Standard NZS1170, Standards New Zealand (2002) where not all three design limit states are required to be considered for all building types and usages. The current Standard is very much life safety focused with enhanced protection for a limited range of buildings (major structures affecting crowds and buildings with post disaster functions) by requiring these to be designed for a longer return period earthquake for the ultimate limit state than “normal” structures. Only buildings with a post disaster function are also required to consider operational continuity after an earthquake by requiring them to also be designed for the SLS2 Serviceability Limit State 2. Onset of damage that repair is required (SLS1, Serviceability Limit State 1) while applying to all buildings but minor structures (farm sheds and the like) is currently set at such a low threshold that it usually has little impact on building design.

Building Usages Identified for Enhanced Performance

Protection from Injury. The research identified several building types/usages, including buildings with high occupancy or vulnerable occupants or those with special post disaster functions all warrant enhanced protection from injury compared with “normal” buildings.

The research identified that high occupancy is much more than large crowds. Analysis indicates structures many be considered “high occupancy” based on a range of criteria: (a) the maximum number of

² ULS: Ultimate limit state; state of significant structural damage but at which life safety is generally assured. SLS1: Serviceability Limit State 1; the structure and non- structural components do not require repair after the SLS1 earthquake, SLS2: Serviceability Limit State 2; the structural maintains operational continuity after the SLS2 earthquake.

people in a building at any one time, (b) the maximum number of people in a single area of a building at any time. (c) the average number of people in a building, or (d) the average weekly usage (i.e. per person-hours per week). The findings from the research reflected the belief that life safety risk should be lower in these buildings given the concentration of risk and societal intolerance of catastrophic structural failures involving multiple casualties.

Types of occupants identified as vulnerable in terms of life safety risk are those with mobility limitations (e.g. users of hospitals, aged care residents) and those that may require direction or management (e.g. dementia care patients, preschoolers, prisoners). This is consistent with international guidelines on performance based design, ICC (2021) where buildings are considered a serious life safety risk if a significant percentage of occupants are members of vulnerable population groups such as infants and small children, elderly people and people with medical disabilities, and where a significant percentage of occupants are unlikely to take protective action during an earthquake (e.g. young children or elderly people) or could require supervised evacuation following ground shaking (e.g. detainees).

In addition the research identified that higher life safety standards may be desirable (but lesser priority than the categories of users noted above) if building users (1) are likely to have a limited choice about whether they enter a building or not (e.g. a prison, school, lawcourt or hospital), (2) are not expected to be familiar with a building and its layout (tourists) and (3) are sleeping occupants as they may struggle to evacuate promptly. The impact of consequential stressors, the indirect effects building owners and users experience during the earthquake, the impacts from loss of amenity and function and the stress of the recovery process were also considered but no additional user or building group was identified as requiring enhanced consideration beyond those identified under casualties.

Analysis of the research findings identified that while the desired outcomes broadly align with the current code settings for life safety protection across all levels of earthquake shaking the range of building usages and occupancies prioritized for enhanced protection for life safety should be considerably expanded.

Protection of Property. The research found that protection of property is important to limit damage to assets, minimize direct financial losses, building waste and consequential stressors. However the research also identified there are few building types/usages that require enhanced performance to reduce damage compared with a “normal” building. An exception are facilities that house cultural treasures or have a high cultural value (e.g. museums). These buildings were identified as needing a higher level of protection from damage compared with “normal” buildings in order to protect the cultural capital of New Zealand. The project team also identified facilities that house important public interests (e.g. police data centers) as requiring enhanced protection from damage compared with “normal” buildings. In addition the research identified a desire for some enhanced protection for buildings with high economic value contents (data centers, specialist laboratories) and buildings likely to house vulnerable occupants with limited financial means to repair damage to ensure equitable recovery.

Analysis of the research findings identified that the current NZ code settings provide significantly less (order of magnitude less) protection of property for all buildings and across all levels of shaking than is desired suggesting a change in the current SLS1 limit state setting would be appropriate. Participants clearly identified that delaying the onset of damage and reducing the levels of damage overall compared with current code settings are important goals.

Protection of Amenity and Function. The research identified that enhanced protection of amenity and function is important for a wider range of building usages than is provided within the current code settings. Buildings recognized as important to provide enhanced protection of amenity and function are those that support post-disaster response functions, recovery enablers (those buildings needed to

effectively support community recovery following a major earthquake), buildings with functions vulnerable occupants are dependent on, buildings needed for community wellbeing support (those buildings required for a community to function normally) and those buildings required to contain hazardous materials to avoid environmental consequences.

In addition the research identified it would also be desirable to provide some enhanced protection of amenity and function in order to reduce disruption for facilities vital for economic output (regional or national), facilities that house agencies for recovery, community facilities that contribute to cultural identity, facilities that contribute to community connection and or sense of place, facilities with occupants sensitive to visible damage and large accommodation facilities.

Analysis of the research findings identified a desire for strengthened (order of magnitude) provisions for protecting building amenity and reducing time of loss of function for all building types/usages for moderate and strong levels of earthquake shaking compared with the current code settings. In addition the research participants identified a much longer list of building types/usages for enhanced protection of amenity and function compared with a “normal” building than is currently considered in NZ.

Protection from Injury	Protection of Property	Protection of Amenity & Function
Population risk exposure (e.g., occupancy rate of building) Vulnerable occupants (e.g., mobility and support requirements)	Cultural significance (e.g., contains taonga or items of public interest)	Post-disaster response functions (e.g., critical infrastructure) Recovery enablers (e.g., schools supermarkets community meeting places) Vulnerable occupants (e.g., life supporting equipment, those heavy impacted through displacement) Community wellbeing (e.g., secure facilities, large residential facilities) Impact on environment (e.g., hazardous materials)

Figure 3. Building usage attributes for enhanced building performance

Building Usage Attributes Summary. The full results of the findings have been collated in tabular form for each dimension of building performance listing the building usage attribute requiring enhanced building performance along with reasoning and building usage examples based on the research findings. For example supermarkets are identified as a recovery enabler under the outcome indicator social disruption. This is to enable individual recovery because of the risk of dislocation without these services. Supermarkets are identified as requiring enhanced protection compared with a “normal” building within the protection of amenity and function building dimension, suggesting the SLS2 Serviceability Limit State 2 should be raised for design of these structures compared with current code settings. The full multipage tables are all listed in Appendix H of the Resilient Buildings Project Report for Stage 3, Abeling et al (2024).

Conclusions

The societal expectations research highlighted that while the current New Zealand code settings generally meet societal expectations for life safety, New Zealanders want an increased focus for all buildings on

reducing loss of function and protecting property – delaying the onset of damage and reducing its consequences overall. In addition New Zealanders want a much wider range of building types and usages prioritized for enhanced performance compared with “normal” buildings than provided within the current code provisions. The reasons vary for the different building usages from life safety protection, protection from damage to protection of amenity and function.

The building usage categorization system developed based on the findings of the societal expectations research provides a clear simple mechanism to identify why certain building usages should have enhanced performance and which dimensions of building performance these relate. This then allows performance objectives to be set for these specific expectations important to that building usage from life safety to limiting damage and maintaining function.

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DEAGGREGATING COMMUNITY-LEVEL SOCIO-ECONOMIC GOALS TO INDIVIDUAL BUILDING SEISMIC PERFORMANCE

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Abstract

Resilient buildings are a necessary but not sufficient condition for community resilience. Design codes and standards in the U.S. are applied at the individual building/facility level, but quantifying the effect of changes in building performance at the community level is difficult. This connection between community-level goals/objectives and design code changes for functional recovery at the individual building level requires a comprehensive interdependent, and interdisciplinary modeling environment. In this paper, the de-aggregation of community goals across physical services stability and economic stability will be demonstrated as a function of adopting individual building retrofit levels for a metropolitan area subjected to earthquake hazard. Specifically, the ability to determine what level of seismic retrofit will provide the physical and economic objectives dictated by a community. The science behind the algorithms within IN-CORE and a demonstration of the modeling and de-aggregation process will be provided using Salt Lake County, UT, and its large number of unreinforced masonry buildings as an example.

Introduction

The performance of buildings and civil infrastructure systems directly affects a community's resilience following an earthquake. However, modern design codes and standards only provide methods to design at the individual facility level, e.g., a building, a bridge, etc. In 2015, the U.S. National Institute of Standards and Technology (NIST) proposed a general framework to help communities develop resilience plans that focused on building clusters (a group of buildings that support a community function such as education) and infrastructure associated with social and economic systems (NIST, 2015). Over the last decade, researchers have focused on utilizing multi-disciplinary measurements at the community level (e.g., Roohi et al. 2020, 2024; Wang et al. 2021). Engineering outcomes can be quantitatively coupled with socio-economic performance for risk-informed decision-making with the public interest in mind (McAllister, 2016). Community resilience modeling can help accelerate the development of building codes and standards to meet the resilience goals of the broader built environment at a higher level, consistent with the performance objectives of individual buildings throughout their service lives (e.g., Ellingwood et al. 2019; Masoomi and van de Lindt, 2019). For example, in the United States, building codes and standards have focused on life safety goals. However, the role of individual building performance in fulfilling community resilience goals has yet to be discovered (Ellingwood et al., 2019). The physical performance of buildings has been quantitatively linked to community-wide social and economic outcomes in only several studies by Roohi et al. (2020, 2024) and Wang et al. (2021).

In this paper, economic and physical services goals at the community level are utilized as targets in a community-level model using the Interdependent Networked Community Resilience Modeling Environment (IN-CORE) (van de Lindt et al., 2023). IN-CORE integrates a broad range of scientific, engineering, and socioeconomic algorithms and data to enable analysts to simulate the effects of different natural hazards on a community and provide decision support to stakeholders, practitioners, and researchers. It is an open-source computational environment that serves as a programming field for students, researchers, and others to program their algorithms. The science behind IN-CORE is the result of a 14-university collaboration with the NIST to measure community resilience in terms of metrics that fall within four community stability areas: population, economics, physical services, and social services; the product of more than 200 archival journal papers (IN-CORE Publications, 2024)—explicitly answering questions such as that proposed in this study, e.g. What number of buildings should be retrofitted in a city in order to achieve community-level socio-economic and physical service goals across the community? Although not presented in this study, IN-CORE has the ability to calibrate building (or other) code levels required to achieve community-level performance goals, making it the ideal platform to understand the complex interactions between buildings, infrastructure such as water and electric power, and households and businesses. The remainder of this paper focuses on an example of using IN-CORE to determine what percentage of unreinforced masonry buildings need to be retrofitted to achieve community-level goals.

Methodology. This section presents the approach used for community-level seismic multidisciplinary resilience modeling to deaggregate community-level socio-economic goals to individual building seismic performance and develop community-level mitigation and implementation plan for buildings. The methodology consists of **i) *Multi-disciplinary community data collection*** in terms of physical and socioeconomic systems utilizing publicly available datasets such as the National Structure Inventory (NSI), local tax assessor data, municipal government databases, the U.S. Census Bureau’s PUMS, the Consumer Expenditure Survey (CE), LEHD, LODS, and financial records like CAFR and SLC Assessor’s data for resilience modeling. **ii) *Seismic multi-hazard analysis*** to determine the hazard intensity measure in the area of interest and assess the risk of liquefaction for a given susceptibility group using the available liquefaction susceptibility data and the Hazus guidelines; **iii) *Damage, functionality, repair, and recovery analysis*** in which *damage analysis* uses building fragility functions to estimate the likelihood of exceeding specific damage states (e.g., slight damage to complete collapse) based on seismic intensity; *functionality analysis* evaluates structural usability after seismic events, classifying buildings as functional or non-functional and considering interdependencies with essential services like electric power systems, and *repair and recovery analysis* quantifies economic losses, repair costs, and recovery time using the Hazus methodology, factoring in variables like structural damage, occupancy class, and restoration curves. **iv) *Economic impact analysis*** uses a Spatial Computable General Equilibrium (SCGE) model to assess how earthquake-induced damages disrupt local economies by measuring output, employment, wages, and tax revenue. Integrating engineering models and economic dynamics captures the direct and indirect effects on economic indicators such as employment, total household income, and domestic supply. **v) *Community-level resilience gap analysis*** incorporates a Playbook-based approach to identify resilience gaps by comparing community goals with

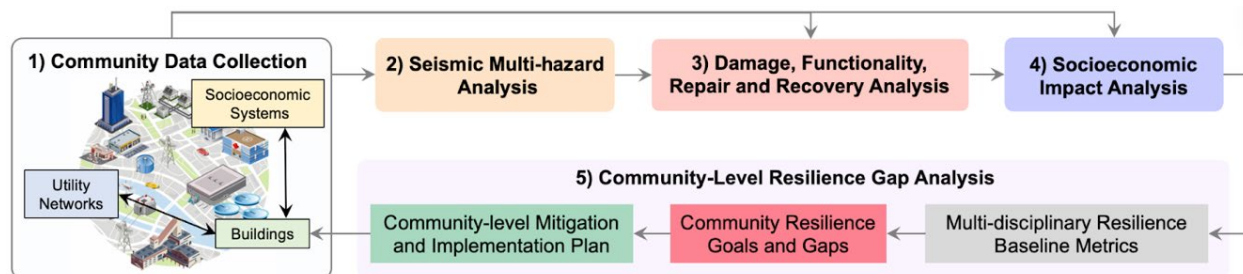


Figure 1. An overview of the proposed method of approach for deaggregating community-level socio-economic goals to individual building seismic performance.

the current baseline performance. This analysis is subsequently used to prioritize mitigation strategies and develop retrofit plans that enhance the functional recovery of physical systems, which results in enhanced resilience of community systems aligned with their objectives. Figure 1 presents an overview schematic for the proposed methodology, illustrating the interaction between different steps.

Resilience Modeling and Partnership: Salt Lake City, UT. Salt Lake City, Utah, and the surrounding county face notable seismic risk due to its proximity to several active faults, one of which is a 343 km long Wasatch Fault. The fault dynamics and seismicity also contribute to the increased risks for liquefaction hazards. Salt Lake County has a population of 1.186 million, and Salt Lake City has a population of 200,478, making it heavily populated with risk exacerbated due to its proximity to the Wasatch Fault, which can produce large $>M_w 7$ earthquakes. A comprehensive multi-disciplinary community resilience model was co-produced with city officials in IN-CORE. Community officials have leveraged the model to prioritize seismic design upgrades based on cost-benefit analysis and to investigate the optimization of seismic retrofit resource allocation. The testbed encompasses extensive data and information on seismic multi-hazard (including liquefaction susceptibility and seismic ground motion scenarios), infrastructures (buildings, electric power facilities and substations, and water distribution networks) and their interdependencies, local repair and retrofit cost data, as well as social and economic data (employment, labor, commuting patterns, financial capital, and demographic profiles). Comprehensive Jupyter notebooks were developed and are available for expansion to run multi-disciplinary community resilience modeling, including physical damage and functionality analysis, housing recovery analysis, population dislocation analysis, economic analysis using computable general equilibrium (CGE) models (Roohi et al., 2024), and automated community-level seismic mitigation analysis (Sediek et al., 2024a, 2024b).

The NIST Community Resilience Planning Guide—with an abbreviated version known as the NIST Community Resilience Playbook—is a framework developed by the NIST to assist communities in planning for natural hazard resilience of their infrastructure and the systems they support. The framework consists of a six-step process: The process starts with forming a collaborative planning team and identifying the leader, team members, and key stakeholders. Then, to understand the baseline situation of the community, the dimensions of the socially built environment and their linkage need to be characterized. This information is leveraged to determine goals and objectives, which involves establishing long-term community goals and performance goals and defining community hazards to determine anticipated performance. This information is used to evaluate existing gaps and develop resilience solutions by developing a community-level resilience plan and its execution, which requires continuous evaluation and updating (Figure 2).

Building Clusters		Design Hazard Performance							
		Phase 1: Short-Term			Phase 2: Intermediate			Phase 3: Long-Term	
		Days			Weeks			Months	
		0	1	1-3	1-4	4-8	8-12	4	4-24
Housing and Neighborhoods	Critical Retail		30%	60%	90%		X		
	Religious and Spiritual Centers			30%	60%	90%		X	
	Single and Multi-family Housing (Full Function)			30%		60%		90%	X
	Schools			30%	60%	90%	X		
	Hotels & Motels			30%		60%	90%	X	

Figure 2. Example building cluster table from NIST Playbook for housing and neighborhoods to demonstrate desired performance goals in phase 1: short-term, phase 2: intermediate term, and phase 3: long-term.

The playbook process is a qualitative process, which highlights the need for a quantitative computation framework to support resilience planning. As part of the NIST Community Resilience Center of Excellence to make advanced resilience modeling frameworks usable and beneficial for communities, a community

application interface was designed to integrate IN-CORE with the Playbook’s six-step planning process to support stakeholder and community engagement by quantifying metrics of the potential impacts on community infrastructure and services, allowing stakeholders—including local government, business leaders, and residents—to make informed decisions about risk priorities and resilience goals. Through the community app, officials can develop adaptive resilience plans, refining strategies as new data becomes available or as community needs evolve. The models ensure that plans remain robust under various future scenarios, making them adaptable to the changing risk landscape. For instance, the community application for SLC (Figure 3) has been implemented by the co-author in close collaboration with city officials and NCSA to empower Salt Lake City with an advanced seismic resilience planning tool (IN-CORE: Multi-disciplinary Community Resilience, 2024).

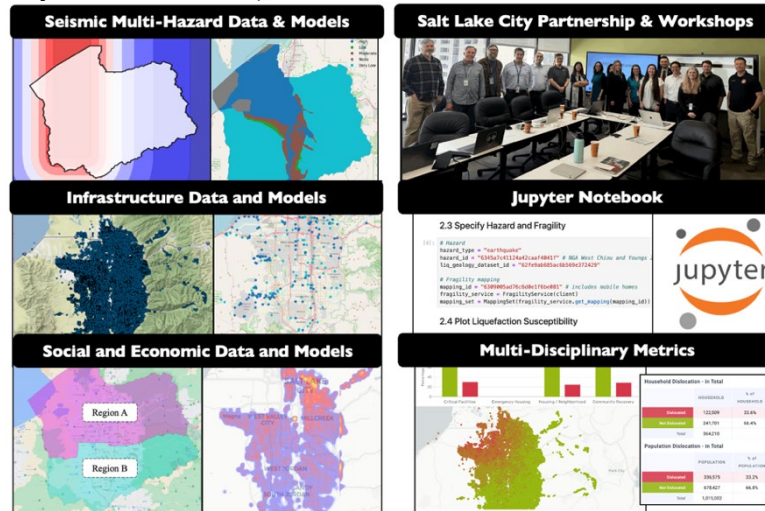


Figure 3. Salt Lake County, UT, multi-disciplinary testbeds consist of seismic hazard data/models, interdependent infrastructure-socioeconomic system data and models, and multi-disciplinary metrics.

Illustrative Case Study of Salt Lake County, Utah: Results and Discussion. This section illustrates the implications of the proposed approach (Figure 3) for Salt Lake County, Utah, implemented using the IN-CORE platform. The study intends to simulate two earthquake ground motions with magnitudes M_w 6.5 and M_w 7.1 at the east bench of Salt Lake City using Chiou and Young's (2014) ground motion prediction equation (GMPE) to generate spatial intensity measures for the study region. It is estimated that a magnitude M_w 7 earthquake on the Salt Lake County segment of the Wasatch Fault could result in over \$33 billion in immediate economic loss and up to 2500 fatalities (Pankow et al., 2015). As a result, the city’s leaders have taken several proactive steps to prepare for the possibility of a significant earthquake. As part of several efforts, such as the FEMA Fix the Bricks program, the city has received federal funding to upgrade unreinforced masonry buildings to withstand a strong earthquake. Many structures have already undergone seismic retrofitting. Community leaders have also worked to inform the public about the danger of residing in a region where a significant earthquake is possible and have created emergency plans to guarantee the safety of the city’s citizens should such an event occur. The proposed methodology uses IN-CORE, incorporating a risk-based decision-making approach that enables quantitative comparisons of alternative resilience strategies. On the IN-CORE platform, data from the community can be seamlessly integrated, allowing users to optimize community disaster resilience planning and post-disaster recovery strategies intelligently using physics-based models of interdependent physical systems combined with socio-economic systems.

Two community-level goals are defined to guide mitigation strategies: **Goal 1** targets improving the 7-day functional recovery of buildings to 90.4% and reducing employment losses to less than 2% in both regions. **Goal 2** focuses on reducing total household income loss to less than 5% of the base value for both earthquake scenarios. The baseline metrics (Table 1) show significant losses across functionality,

employment, and household income. Due to its greater intensity, the Mw7.1 earthquake consistently resulted in higher impacts than Mw6.5. Under Mw7.1, building functionality after 7 days reached only 78%, with an even lower combined functionality of buildings and electric power systems (EPS) at 66.5%. The latter is obtained by considering the interdependence between buildings and electric power substations. The economic analysis defines two northern and southern SLC regions: A and B (Figure 4). For Region A, household income losses were \$1.22B (-8.21%) for Mw7.1, compared to \$816M (-5.48%) for Mw6.5. Region B performs better, with losses below the 5% threshold under both scenarios. Regarding employment impact, Region A experienced more significant employment losses, with Mw7.1 reducing employment by -13,115 jobs (-2.68%) compared to -8,028 jobs (-1.64%) under Mw6.5. Region B showed smaller but notable losses.

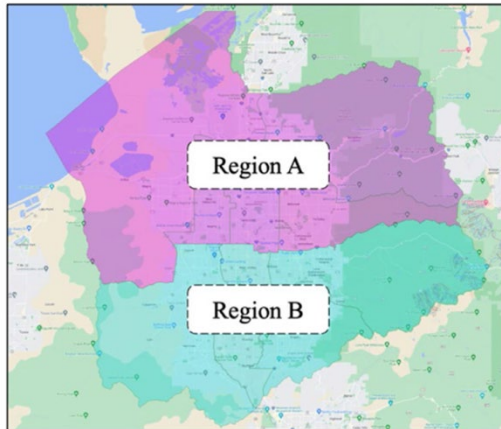


Figure 4. Disaggregated two economic regions in SLC.

This study simulates five cases to study the effect of different levels of mitigation actions. Case 1 is the case where all the buildings in SLC are unretrofitted. Whereas cases 2, 3, 4, and 5 are the cases that have 25%, 50%, 75%, and 100%, respectively, of the URM in SLC retrofitted following the approach discussed earlier. It should be noted that the retrofitted buildings are assigned randomly even though the authors have optimized the seismic retrofit allocation on the community level, which is outside the scope of this paper.

The analysis for Goal 1, to improve functional recovery and employment, shows that moderate retrofitting (50% or Case 3) is sufficient to achieve functionality and employment targets. Regarding building functional recovery, Case 3 (50% retrofit) improves 7-day building functionality to 90.4%, meeting the target for Mw6.5 and Mw7.1. Regarding employment, for Mw7.1, Case 3 reduces employment losses in Region A to -2.21% and Region B to -2.00%, meeting the <2% target. For Mw6.5, baseline employment losses already fall below the target. The findings highlight the efficacy of moderate retrofitting in achieving functionality and employment recovery, demonstrating the cost-effectiveness of Case 3 strategies compared to more extensive retrofits.

The analysis for Goal 2, to reduce income loss, demonstrates that achieving a total household income loss below 5% is feasible with moderate retrofitting strategies. For Mw7.1 Earthquake, a 75% retrofit (Case 4) reduces Region A's income loss to \$0.82B (-5.49%), close to the target. Region B's baseline loss already meets the target. For the Mw6.5 earthquake, a 75% retrofit (Case 4) further decreases Region A's income loss to \$0.48B (-3.23%), offering enhanced resilience. Region B's baseline performance meets the goal without retrofitting. This demonstrates the disproportionate economic vulnerability of Region A compared to Region B, emphasizing the need for targeted retrofitting efforts in high-impact areas.

Table 1. Baseline metrics for Mw6.5 and Mw7.1 earthquakes including functionality, employment, household income, and domestic supply losses for Regions A and B. The table compares base values to unmitigated losses, providing a foundation for assessing the impact of retrofitting strategies.

Analysis Case			Mw6.5	Mw7.1
Metric	Region	Base Value (Without Loss)	Baseline (Loss)	Baseline (Loss)
Buildings Functional	Combined	100%	88.39% (7-day recovery)	78.80% (7-day recovery)
Buildings + EPS Functional	Combined	100%	77.8% (7-day recovery)	66.52% (7-day recovery)
Employment Impact	A	488,950 jobs	-1.64% (-8,028 jobs)	-2.68% (-13,115 jobs)
	B	229,468 jobs	-1.36% (-3,113 jobs)	-2.35% (-5,396 jobs)
Household Income Loss	A	\$14.87B	\$0.82B (-5.48%)	\$1.22B (-8.21%)
	B	\$12.82B	\$0.27B (-2.14%)	\$0.58B (-4.49%)
Domestic Supply Loss	A	\$48.46B	\$2.86B (-5.91%)	\$4.54B (-9.37%)
	B	\$27.78B	\$0.97B (-3.48%)	\$1.83B (-6.58%)

Table 2. Goal 1 analysis to improve functional recovery and employment under Mw7.1 earthquake. The table presents baseline metrics, predefined targets, gaps, and retrofitting requirements to close the gap.

EQ	Metrics	Region	Baseline (Case 1)	Target	Gap	Retrofit Level Needed
Mw 7.1	Buildings Functional (7-day recovery)	Combined	78.00%	90.00%	12.40%	Case 3 (90.4%)
	Employment	A	-2.68% (-13,115 jobs)	< -2.0%	+0.68% (+3,336 jobs)	Case 3 (-2.21%)
		B	-2.35% (-5,396 jobs)	< -2.0%	+0.35% (+1,199 jobs)	Case 3 (-2.00%)

Table 3. Goal 2 analysis to reduce total household income loss for Regions A and B under Mw6.5 and Mw7.1 earthquake scenarios. The table highlights gaps between baseline losses and predefined goals (<5%) and identifies retrofit levels required to meet these goals.

EQ	Metrics	Region	Base Value (mil \$)	Baseline (Case 1)	Target (5% or less)	Gap	Retrofit Level Needed
Mw 7.1	Household Income Loss	A	14876	\$1.22B (-8.21%)	\$0.74B (-5.0%)	+\$0.48B (+3.21%)	Case 4 (\$0.82B, -5.49%)
		B	12823	\$0.58B (-4.49%)	\$0.64B (-5.0%)	Met	No Retrofit
Mw 6.5		A	14876	\$0.82B (-5.48%)	\$0.74B (-5.0%)	+\$0.072B (+0.48%)	Case 4 (\$0.48B, -3.23%)
		B	12823	\$0.27B (-2.14%)	\$0.64B (-5.0%)	Met	No Retrofit

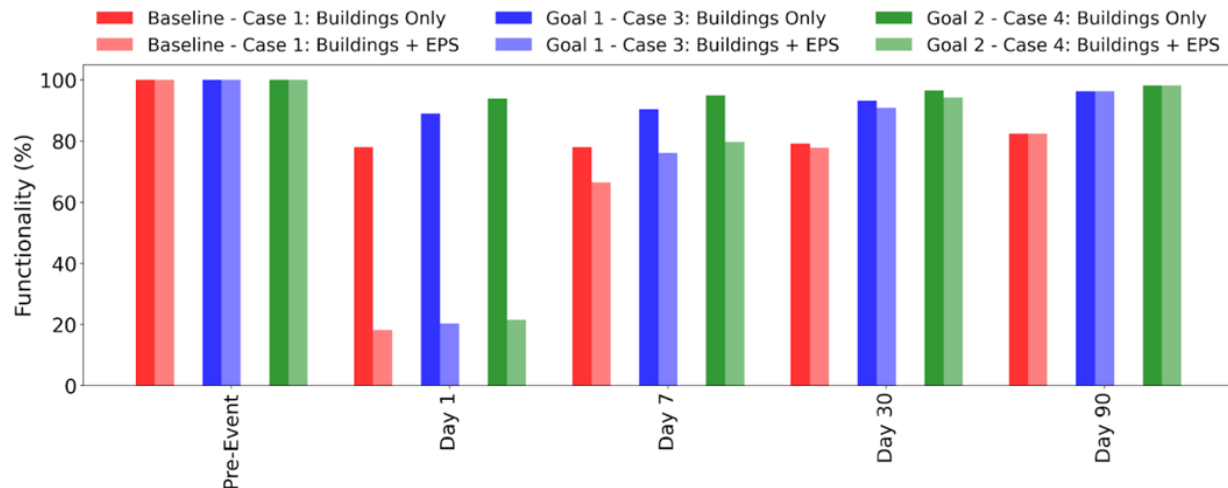


Figure 5: Recovery of buildings with and without EPS considered

Acknowledgments

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SEISMIC DESIGN IN NONE-HIGH SEISMIC AREAS

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Abstract

As many people know, a whole area in Japan is a high seismic area and the major target of lateral force design is seismic design. On the other hands, seismic risk differs per the country or area. Some areas in some other countries are none-high seismic areas. In these areas, seismic load may not govern the lateral force design.

I have worked as a structural engineer not only in Japan but also in several countries. Through my experience especially in None-High Seismic Area, I feel some differences between Japanese design and others.

As a matter of fact, US design standards are applied or referred in some of these areas. Even in these none-high seismic areas, a kind of “Special” Seismic Force-Resisting System is likely to be applied without any further consideration because many engineers just believe it is better than a kind of “Ordinary” Seismic Force-Resisting System. However, I wonder whether it is true and enough. I think the prefix “Special” might mean only the grade of ductility but might not mean the grade of the building structure itself.

I would like to post what I found and feel through my experience including the comparison with Japanese design.

Introduction

US design standards are used or referred in many countries. Because they cover the various design conditions including seismic design. ASCE7 is a well-known standard as “Minimum design loads for buildings and other structures”. The design concept of ASCE7 is referred in many countries because it is applicable from low seismic region to high seismic region.

The big difference between seismic design and others is that it expects and relies on post yield performance of the structure regardless of applied analysis procedure. Therefore, it is expected for structural engineers to imagine the status of the structure against several level of earthquakes.

Seismic Design Concept per US Design Standards

Figure 1 shows seismic design concept of US design standards. ASCE7 specifies that seismic design force is calculated based on the design response spectrum at the building place with the fundamental period of the building and R factor per seismic force-resisting system which structural engineer selects. For seismic force-resisting system, several systems are specified in ASCE7, such as reinforced concrete moment frames, steel braced frames, etc. Besides, each system is classified into 3 categories such as “Ordinary”, “Intermediate”, and “Special” by the degree of ductility. Low ductility corresponds to small R factor and high ductility corresponds to large R factor. Therefore, reinforced concrete moment frames with high ductility is called as Special reinforced concrete moment frames.

Figure 2 is the extraction from ASCE7. Applicability of each seismic force-resisting system is regulated based on Seismic Design Category (B-F) and building height. Taking steel moment frames as example, Steel special moment frames can be applied to every seismic design category without any height limitation. On the other hand, Steel intermediate moment frames can be applied to seismic design category B-D with height limitation of 35ft for seismic design category D. Steel ordinary moment frames can be applied to seismic design category B & C only.

This information may lead to structural engineers to select a kind of “Special” seismic force-resisting systems for the building in severe seismic design category or high-rise (= important) buildings.

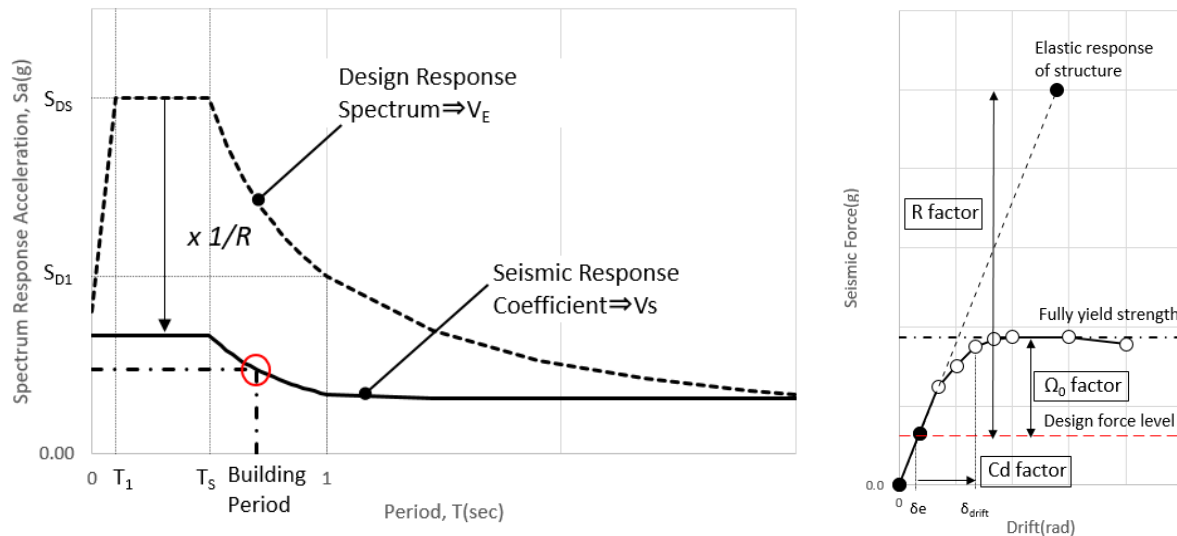


Figure 1. Seismic Design Concept per US Design Standards

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, Ω_o^b	Deflection Amplification Factor, C_d^c	Structural System Limitations Including Structural Height, h_n (ft) Limits ^e				
					Seismic Design Category				
					B	C	D ^d	E ^d	F ^d
C. MOMENT-RESISTING FRAME SYSTEMS									
1. Steel special moment frames	14.1 and 12.2.5.5	8	3	5½	NL	NL	NL	NL	NL
2. Steel special truss moment frames	14.1	7	3	5½	NL	NL	160	100	NP
3. Steel intermediate moment frames	12.2.5.7 and 14.1	4½	3	4	NL	NL	35 ^b	NP ^b	NP ^b
4. Steel ordinary moment frames	12.2.5.6 and 14.1	3½	3	3	NL	NL	NP ^b	NP ^b	NP ^b
5. Special reinforced concrete moment frames ^a	12.2.5.5 and 14.2	8	3	5½	NL	NL	NL	NL	NL
6. Intermediate reinforced concrete moment frames	14.2	5	3	4½	NL	NL	NP	NP	NP
7. Ordinary reinforced concrete moment frames	14.2	3	3	2½	NL	NP	NP	NP	NP

Figure 2. Extraction from ASCE7-10

Figure 3 shows seismic design requirements for the structure with different R-factor, respectively. The structure with larger R factor requires higher ductility but allows smaller seismic design force. It may result in smaller stiffness due to smaller structural members. On the contrary, the structure with smaller R factor allows lower ductility but requires larger seismic design force. It may result in higher stiffness due to larger structural members.

Structural strength needs to satisfy against design earthquake level with load factors. The design earthquake level is specified as two-thirds of the risk-targeted maximum considered earthquake. The basic performance objective based on ASCE41 is “life safety” and “collapse prevention” for each earthquake level, respectively. The criteria against smaller earthquakes like mean return period 50 years are not specified clearly.

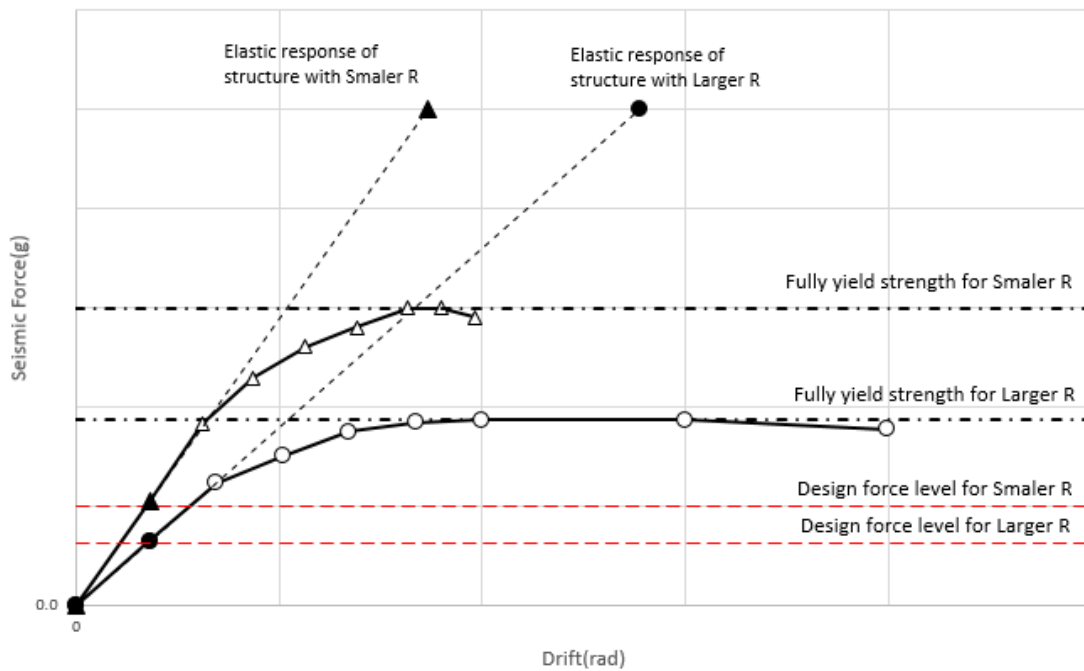


Figure 3. Comparison of Seismic Design Requirements with different R-factor

Seismic Design Concept per Japanese Design Standards

There is not big difference in seismic risk everywhere in Japan. Figure 4 shows seismic design concept of Japanese design standards. Strength requirement in fully yield state differs per seismic force-resisting system which structural engineer selects. The system with lower ductility requires larger strength and the system with higher ductility requires smaller strength, which is similar to US design standards. Except, Japanese standards don't have height limitation per each system.

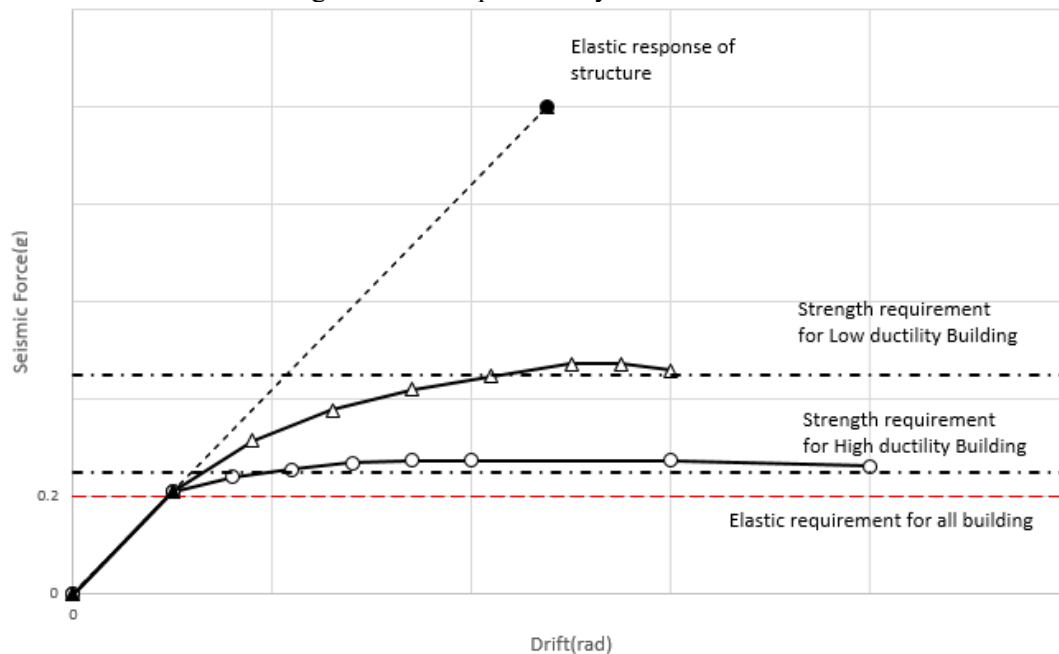


Figure 4. Seismic Design Concept per Japanese Design Standards

In addition to the ductility concept, two-stage design is required in Japan. The 1st stage design is for the seismic design load which the building may suffer several times in the building lifetime. Each structural member is required to be elastic against this seismic load. Japanese standards specify this load as 20% of the building weight at the base. This value is uniform for every building regardless selected seismic force-resisting system. The 2nd stage design is for the seismic design load for the big earthquake, which is 100% of the building weight at the base. In this stage, fully yield strength of the building is confirmed to exceed the required strength.

Therefore, the buildings in Japan are expected to keep elastic through their lifetime and to be safe against the big earthquake in case it happens. The regulation is based on the theoretics and it is compensated empirically.

Effect on structural design by different ductility

Building with lower ductility can't deform so much. Therefore, it requires larger strength to absorb the energy. On the other hand, building with higher ductility can deform well. Therefore, it allows smaller strength to absorb the energy. However, this is character in plastic condition and it is only after some structural members yield.

Structural Design in None-High Seismic Areas

A kind of "Special" Seismic Force-Resisting System is likely to be applied without any further consideration because many engineers just believe it is better than a kind of "Ordinary" Seismic Force-Resisting System. Therefore, some engineers just apply the system with large R factor and they believe it is enough for the seismic design without further thought. However, achieving high ductility is not enough item for structural engineer to consider. Applying larger R factor means the designed building yields relatively smaller lateral force than the building designed with small R factor. The building stiffness also may be smaller due to the smaller structural members.

As a matter of fact, the structural design may be governed by gravity design for some buildings in low-moderate seismic region. In these regions, it would not have a big impact on the whole structural design even if seismic design force is increased intentionally.

Points what Structural engineers need to consider

Following items are important points what structural engineers should keep in mind for structural design.

1.Speciality of seismic design

Gravity design and wind design is elastic design. However, seismic design is based on plastic design regardless the analysis procedure is linear or non-linear procedure. Especially the structure members of seismic force-resisting system, such as beams & walls, with large R factor are expected yield mostly.

2.Damage to main structural members

It is not easy to rectify or replace the structure members of seismic force-resisting system, such as beams & walls, if they yield fully. Therefore, structure shall be kept elastic as much as possible.

3.Damage to non-structural components

The damage of non-structural components, such as ceilings & partition walls, may be caused by deflection of the building. Higher stiffness of the building may help to mitigate the damage.

Case Study

Assuming 4-story office building in Seismic design category B (Site Class = B, $S_{DS} = 0.2$, $S_{D1} = 0.1$), case study was conducted. Table 1 shows the description of each case. Figure 5 shows Design response spectrum, Figure 6 shows schematic structural framings, and Figure 7 shows structural members for each case, respectively.

Table 1. Case Study

Case	Description	
Case-1	Ordinary reinforced concrete moment frames	(R=3, I=1.0)
Case-2	Special reinforced concrete moment frames	(R=8, I=1.0)
Case-3	Special reinforced concrete moment frames	(R=8, I=8/3)
Case-4	Special reinforced concrete moment frames with damper	(R=8, I=1.0)

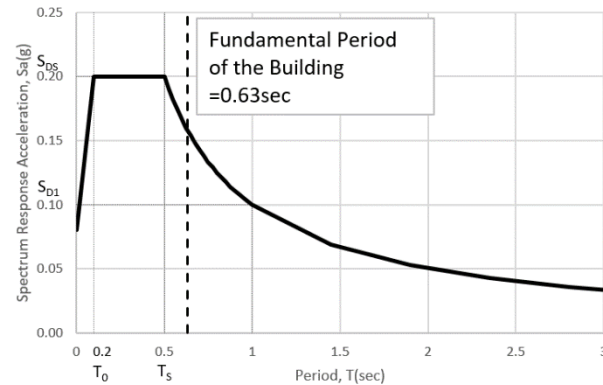
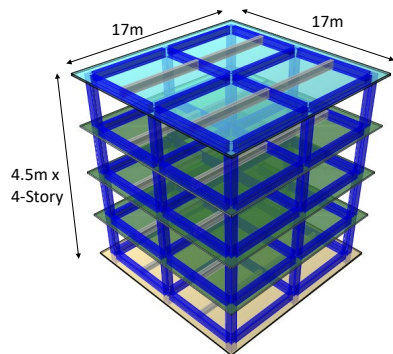


Figure 5. Design Response Spectrum



Material
Concrete: $f_c' = 28\text{MPa}$
Rebar: ASTM A706/A615 Gr.60

Concrete Secondary Beam Schedule

Symbol	B1			B2		
	Outer End	Middle	Inner End	Both Ends	Middle	Middle
Roof & 4th						
Longitudinal Top Bar	3-D19	3-D19	4-D19	3-D19	3-D19	4-D19
Longitudinal Bottom Bar	3-D19	6-D19	3-D19	3-D19	5-D19	3-D19
Transverse Bar	D10-@800	D10-@800	D10-@800	D10-@800	D10-@800	D10-@800

Concrete Slab Schedule

Symbol	Thickness (mm)	Position	Short Span Dir.		Long Span Dir.	
			End	Middle	End	Middle
S15	150	Top	D10@D13-@200	←	D10-@200	←
		Bottom	D10-@200	←	D10-@200	←
S15A	150	Top	D10@D13-@200	←	D10-@200	←
		Bottom	D10@D13-@200	←	D10-@200	←

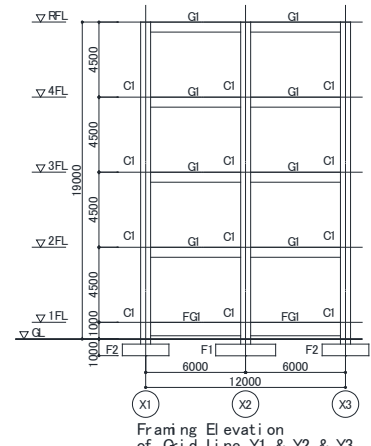
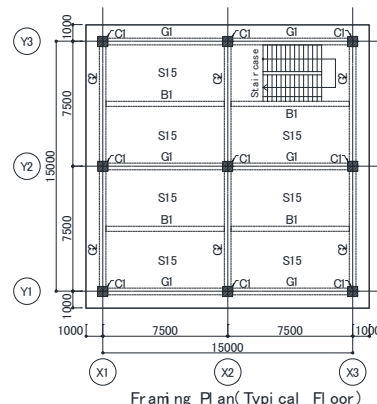
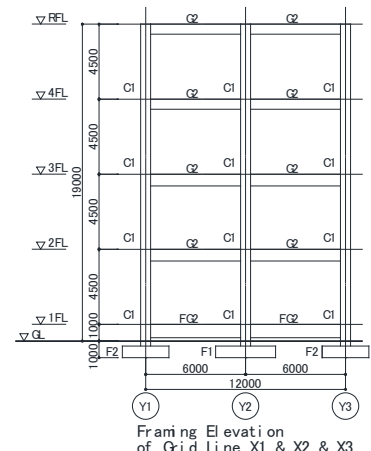
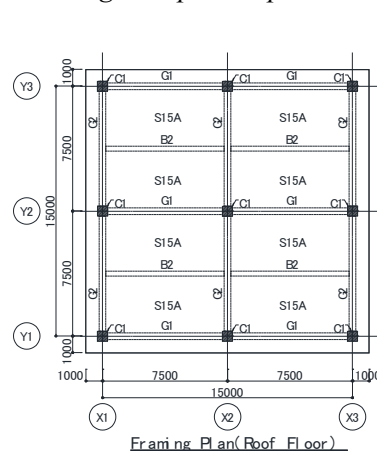


Figure 6. Image of the Building

Concrete Column Schedule

Symbol	C1
Typical Zone	
Longitudinal Bar	4-D22 + 8-D19
Tie Bar TYP.	D10-BB-@300
Lo Zone (Lo=0mm)	
Longitudinal Bar	
Tie Bar at Lo	

Concrete Girder Schedule

Symbol	G1		G2	
	Both Ends	Middle	Both Ends	Middle
Roof & 4th				
Longitudinal Top Bar	4-D22	3-D22	6-D22	3-D22
Longitudinal Bottom Bar	3-D22	3-D22	3-D22	5-D22
Tie Bar	D10-□-@250	D10-□-@250	D10-□-@200	D10-□-@250
3rd & 2nd	Both Ends	Middle	Both Ends	Middle
				
Longitudinal Top Bar	5-D22	3-D22	6-D22	3-D22
Longitudinal Bottom Bar	3-D22	3-D22	3-D22	4-D22
Tie Bar	D10-□-@250	D10-□-@250	D10-□-@300	D10-□-@300

Legend

 Increased Item
 Reduced Item
 (Compared with Case-1)

(Case-1)

Concrete Column Schedule

Symbol	C1
Typical Zone	
Longitudinal Bar	4-D22 + 8-D19
Tie Bar TYP.	D10-BB-@100
Lo Zone (Lo=650mm)	
Longitudinal Bar	4-D22 + 8-D19
Tie Bar at Lo	D13-BB-@100

Concrete Girder Schedule

Symbol	G1		G2	
	Both Ends	Middle	Both Ends	Middle
Roof & 4th				
Longitudinal Top Bar	3-D22	2-D22	5-D22	3-D22
Longitudinal Bottom Bar	2-D22	3-D22	3-D22	5-D22
Tie Bar	D10-□-@100	D10-□-@200	D13-□-@125	D13-□-@175
3rd & 2nd	Both Ends	Middle	Both Ends	Middle
				
Longitudinal Top Bar	4-D22	2-D22	6-D22	3-D22
Longitudinal Bottom Bar	3-D22	3-D22	3-D22	5-D22
Tie Bar	D13-□-@125	D13-□-@250	D13-□-@125	D13-□-@175

(Case-2)

Concrete Column Schedule

Symbol	C1
Typical Zone	
Longitudinal Bar	4-D22 + 8-D19
Tie Bar TYP.	D10-BB-@100
Lo Zone (Lo=650mm)	
Longitudinal Bar	4-D22 + 8-D19
Tie Bar at Lo	D13-BB-@100

Concrete Girder Schedule

Symbol	G1		G2	
	Both Ends	Middle	Both Ends	Middle
Roof & 4th				
Longitudinal Top Bar	4-D22	3-D22	6-D22	3-D22
Longitudinal Bottom Bar	3-D22	3-D22	3-D22	5-D22
Tie Bar	D13-□-@125	D13-□-@250	D13-□-@100	D13-□-@125
3rd & 2nd	Both Ends	Middle	Both Ends	Middle
				
Longitudinal Top Bar	5-D22	3-D22	6-D22	3-D22
Longitudinal Bottom Bar	3-D22	3-D22	3-D22	4-D22
Tie Bar	D13-□-@125	D13-□-@250	D13-□-@125	D13-□-@175

(Case-3)

Concrete Column Schedule			Concrete Girder Schedule			
Symbol	C1		Symbol	G1		G2
4th ~ 1st	Typical Zone		Roof & 4th	Both Ends	Middle	Both Ends Middle
	Longitudinal Bar	4-D22 + 8-D19				
	Tie Bar TYP.	D10-@100				
	Lo Zone (Lo=650mm)					
3rd & 2nd	Longitudinal Bar	4-D22 + 8-D19	3rd & 2nd	Both Ends	Middle	Both Ends Middle
	Tie Bar	D13-@100				
	Lo Zone					
	Tie Bar	D13-@100				
D1 Damper Schedule						
Symbol	CDI (4 No. @ Story)					
4th ~ 1st	F _{max} = 250(kN) V _{max} = 150(kN/s) C = 1.67(kN·s/mm)					

(Case-4)

Figure 7. Schedule of Structural member

Table 2. Summary of Each Structure

Case	Design Base Shear	Condition of structure			Structural Quantity Comparison with Case-2
		50% of Design Earthquake	100% of Design Earthquake	150% of Design Earthquake	
Case-1	0.053	Elastic	Plastic	Fully Plastic (without margin)	Rebar: 88% Form: 100% Concrete: 101%
Case-2	0.020	Plastic	Plastic	Fully Plastic (with margin)	Rebar: 100% Form: 100% Concrete: 100%
Case-3	0.053	Elastic	Elastic	Plastic	Rebar: 102% Form: 100% Concrete: 101%
Case-4	0.015	Elastic	Elastic	Elastic	Rebar: 99% Form: 100% Concrete: 100% +Damper

Table 2 is the summary of each structure. The condition of structure can be classified based on the theory of standards. This table shows following things,

- Condition of structure against 50% of Design earthquake
Case-1, 3, 4 can be elastic. However, Case-2 is plastic due to small design load.
- Condition of structure against 100% of Design earthquake
Case-3 & 4 can be elastic. However, Case-1 & 2 are plastic. Because they are designed based on the code minimum requirements.
- Condition of structure against 150% of Design earthquake
Case-4 can be elastic. Case-3 is plastic. Case-1 & 2 are fully plastic.

4. Structural Quantity

The concrete quantity of Case-1 and Case-3 is slightly large due to the larger design base shear force. The rebar quantity of Case-1 is the smallest due to the less requirements of rebar details. The difference of structural quantity is not so big. Especially, the quantity of Case-2 and Case 3 is almost same.

Conclusion

1. Assuredly, selecting the seismic force-resisting system with higher ductility is important for the safe structure.
2. It is not only ductility for structural engineers to take care for structural performance but also other items to be minded such as, elastic limit, stiffness, ultimate strength, etc.
3. Selecting the seismic force-resisting system with large R factor and designing with smaller design load mean that the structural members of designed building yield in small earthquakes.
4. It is not easy to rectify or replace the structure members of seismic force-resisting system, such as beams & walls, if they yield fully.
5. In none-high seismic area, structure can be kept elastic until certain level of earthquakes by increasing the seismic design force. Because original seismic force is not so big.
6. Adding damper is also one option to achieve the structure with good performance.

Structural engineers need to figure out the actual performance of the structure and need to imagine what happens against several level of hazards. R factor implies the degree of ductility. Definitely, high ductility is important item for good structure. But ductility is only for plastic condition. There are many other items which structural engineers need to consider for better performance.

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SEISMIC PERFORMANCE OF TALL STEEL FRAMED BUILDINGS BUILT BETWEEN 1960 - 1994

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Abstract

The 1994 Northridge, California earthquake demonstrated the potential vulnerability of older tall steel-framed buildings to large earthquakes. This paper will focus on the seismic performance of many tall steel framed structures in San Francisco designed between 1960 and 1990. The paper will discuss how the structural design and detailing of these buildings evolved due to changes in construction technology, architectural considerations, engineering design techniques, and analysis software. Detailed modeling techniques used to simulate the potential for fracture of both the welded beam-to-column connections and the partial joint penetration column splices will also be presented. Evaluation of the results clearly demonstrated that it is difficult, if not impossible, to draw broad conclusions about the seismic performance of this entire class of buildings. However, these detailed evaluations allowed for comparison between the different seismic performance features and provide the ability to make some observations about individual building performance characteristics.

Introduction

On April 17, 2018, the New York Times published an article titled “San Francisco’s Big Seismic Gamble” that discussed potential seismic concerns related to steel moment frame construction. From the early 1900s until through the 1990s, the most common structural system for tall commercial office buildings in California has been steel moment frames. The Times article correctly pointed out that this structural system is prevalent in most of the office buildings in San Francisco, along with most large cities in regions of high seismicity. At issue is the potential vulnerabilities of these buildings due to brittle fracture in welded beam-column connections that was observed in the 1994 Northridge earthquake. The fracture of welds joining the beam-flanges to the column flanges that occurred in approximately 100 moment frame buildings during Northridge earthquake was a major surprise to the structural engineering community. Following the damage caused by the Northridge earthquake, the Federal Emergency Management Agency funded a \$12 million six-year program to study this issue by the “SAC Joint Venture”. Based on the SAC recommendations, the American Institute of Steel Construction developed detailed provisions required for testing, analysis and design to re-establish the use of prequalified connections that could be demonstrated to develop the required inelastic rotation capacity of the beam prior to fracture. The work of the SAC project and AISC led to major changes in design practice for new construction, but the knowledge of the vulnerabilities of existing steel moment frame buildings did not lead to any overarching evaluation or retrofit ordinances for such structures. The main issue is at what level of earthquake shaking do these vulnerabilities become significant risks and what are the consequences of such risks? This paper provides a high-level discussion of those questions based on seismic evaluations of over thirty steel moment frame buildings built between 1910 and 1995, with the majority built between 1960 and 1990.

Historic Performance Other than from the Northridge Earthquake

In the 1906 San Francisco Earthquake, most tall commercial buildings in San Francisco were made of riveted steel frames, almost always combined with unreinforced masonry infill walls. In general, these buildings performed well, without a single collapse. This exemplary performance led engineers to believe a complete steel moment frame system to be the superior earthquake resisting system. Building code provisions in the decades that followed incentivized the use of this system by

permitting lower design forces than other structural systems. Prior to the 1994 Northridge earthquake, the only major failure of a structural steel building due to earthquake occurred in the 1985 Mexico City earthquake. One of the five twenty-story towers of the Pino Suarez building complex collapsed, primarily attributed to the local buckling of the box columns [3]. In the 1995 Great Hanshin earthquake that struck Kobe, Japan some steel framed buildings performed poorly [4], with several building collapses and many buildings sustaining irreparable damage. The more robust Japanese beam-column connection led to a significant increase in damage to the building columns. Of note was the failure of welded column splices and failures in the column where it connected to the beam.

Evolution of Steel Moment Framed Building Design and Construction

The earliest mid-rise office buildings employed complete steel frames with riveted connections between the beams and columns. In addition to the riveted beam-column connection, the entire frame was typically encased in concrete for fire protection, but also likely provided additional strength. Masonry or clay tile walls provided additional stiffness in these buildings, acting as struts to help limit interstory building drift. Early column splices were made with riveted plates placed over the top and bottom column sections. In the 1950s, high strength bolts began to replace rivets in the beam column connection, with the complete moment frame still encased in concrete for fireproofing. The practice changes as welding became more common in the late 1950s. Welding was initially done where clip angles or bolted “T” sections were replaced with plates welded to the column and in the fabrication shop and then bolted to the beam in the field. Bolts also replaced rivets in the column splices.

In the early 1960s, the advent of field welding led to changes where the beam flanges and web were welded directly to the column. While the connection changed, the practice of connecting every beam-column connection rigidly to provide a space frame continued to be the norm. In the late-1960s, concurrent with tests performed at UC Berkeley [1,2] the connection was modified to allow the web to be bolted to the beam, (Figure 1). Elimination of the web weld reduced the cost of this connection leading this detail to become the standard connection; now referred to as the “Pre-Northridge connection”. This is the connection detail damaged in the Northridge earthquake. Field welding of the columns splices also became the norm in the 1960s, typically via partial-penetration welds ranging from about one-third to just over one-half the thickness of the flange or web.

In the 1970s, the design of welded steel moment frames moved from distributed space frames with every beam-column connected rigidly, toward perimeter moment frames. The reduction in the number of frames led to the use of larger beam and column sections. As practice moved into the 1980s, the amount of frame bays along the perimeter began to decrease. Economic considerations of labor and steel weight pushed practice toward reducing the number of moment frame connections needed, with the trade-off being much larger beam and column sections. Irregularities, such as building setbacks and discontinuous or offset columns, within the moment frame lines became more frequent in 1980s buildings. This might have been due to the more commonplace use of computerized structural frame analysis programs that made more complicated geometries possible to evaluate. Such practices continued until the 1994 Northridge Earthquake.

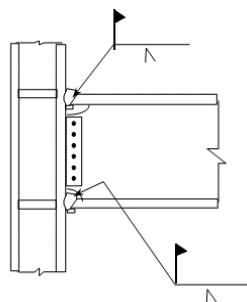


Fig. 1 - Welded Unreinforced Flange w/ Bolted Web

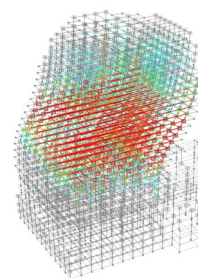


Fig. 2 - Multi-Story Mechanism

Building Evaluation Observations

Of the more than thirty tall steel framed buildings evaluated, fifteen evaluated using nonlinear response history analyses provided the best data for understanding performance of this class of structures. All buildings evaluated with nonlinear response history analyses were between twenty and sixty stories tall, designed and constructed between 1960 and 1990. The buildings were evaluated using the nonlinear dynamic procedures specified in ASCE 41-17. In most cases the buildings were subjected to ground motions suites corresponding to each of the four seismic hazards specified in ASCE 41. Often the same suite of records was used for the different hazards, with different scale factors applied to each ground motion.

One of the first observations made was the difference in design practice between firms and even within the same firm between buildings. The choice of frame layout, whether it be perimeter or distributed space frame, often changed between buildings presumably because of other factors, such as architectural form or building program. Overall, none of the existing buildings analyzed met the Collapse Prevention structural performance level of ASCE 41 at the BSE-2N, the ASCE 7 Risk Targeted Maximum Considered Earthquake (MCER) intensity, or at the BSE-2E – the 975-year uniform hazard intensity. All buildings had at least two records that indicated collapse potential, but buildings with significant irregularities often indicated collapse for more than half the suite's records. Most of the existing building analysis models failed via a multi-story mechanism (Figure 2), caused by softening of the beams and columns to the point where there was not sufficient restoring stiffness to overcome the P-Delta loads and displacements. Most regular buildings designed to the 1976 UBC or later met the ASCE 41 Collapse Prevention structural performance level in the BSE-1N / ASCE 7 DE intensity. Several of the buildings that met the Collapse Prevention structural performance level would not have met the Life Safety structural performance level in ASCE 41-17 due to the acceptance criteria for force-controlled actions demands on the partial penetration welded column splices. The same concerns were also identified in the BSE-2E and -2N evaluations. Like the BSE-1N, most of the buildings met the Collapse Prevention level at the BSE-1E, but some did not meet the Life Safety performance level due to column splices being overstressed. The buildings that did not meet the Collapse Prevention level at the BSE-1E or -1N had one of three features:

- (1.) They were designed prior to the 1976 UBC and simply did not have enough strength and stiffness in the moment frames.
- (2.) They had column splices with less than half the flange thickness welded, leading to earlier incidences of fractures in the analysis models.
- (3.) They had a significant change in strength in the framing, such as a set-back or the number of moment frame bays reduced up the height of the building that created irregular response.

Interestingly, the pre-Northridge moment connection had a significant effect on the performance of the building primarily when there was a strength irregularity. It makes sense that more brittle response mechanisms created by fracturing connections will be more sensitive to irregular configurations than more ductile mechanisms that have more capacity to re-distribute demands to other portions of the building. The other two issues, column splice fracture (which will be discussed in a subsequent section) and global stability failure due to lack of strength and stiffness, led to collapse of the building regardless of whether the connections were pre-Northridge or AISC 358 conforming connections. While the presence of pre-Northridge connections did worsen the performance, the building would still not meet the target performance levels without either significant column strengthening, the addition of new lateral elements or supplemental damping devices.

Linear Versus Nonlinear Analysis

One of the most significant observations coming from these evaluations is the potential inadequacy of linear dynamic analysis in accurately assessing the performance of these buildings. In several buildings, the linear analysis predicted borderline acceptable performance, while the nonlinear analysis predicted significant issues that prevented the building from meeting the targeted performance level. Figure 3 shows a comparison between the linear and the nonlinear analysis drift. The linear analysis does not properly predict the multi-story collapse mechanism. Once a story is weakened, the dynamic behavior of the building changes and demands above and below that story are significantly increased. The propagation of yielding or fracture cannot be captured in a linear analysis because of the code assumption of uniform yielding throughout the structure. Drifts where the multi-story mechanisms occur, or strength irregularities are present are significantly underpredicted in linear analyses.

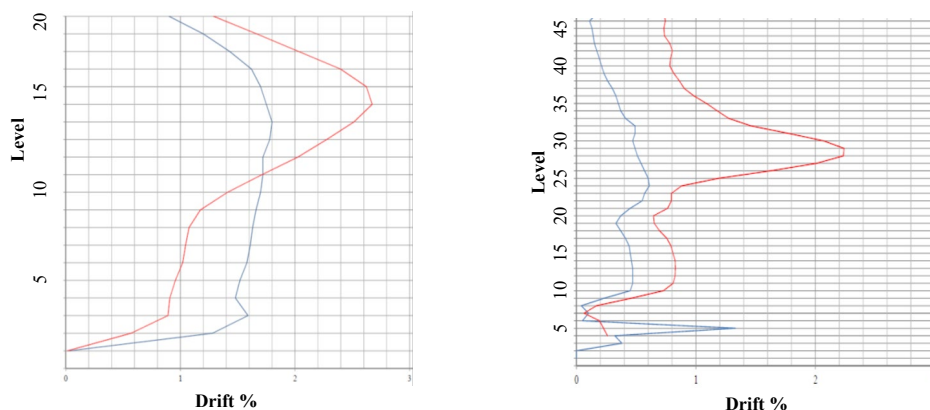


Fig. 3: Linear (in Blue) Versus Nonlinear (in Red) Analyses Drifts for Two Example Buildings

Column Issues

All the building evaluations identified issues with the building columns being more significant than the pre-Northridge connections. Box columns were not prevalent in the buildings damaged by the Northridge earthquake but were used in several of the studied buildings. Box columns were frequently used in tall moment frame buildings, especially when the design used a full space frame. The original designers reduced the plate thickness until at some point along the height of the structure, the columns would become noncompact or slender with respect to local buckling. ASCE 41 accounts for the significant reduction in ductility for non-compact sections. The multi-story failure mechanism typically was in the stories with non-compact columns. As noted above the Pino Suarez building was due in large part to local buckling of thin-walled built-up box shaped columns [3].

Column splice fracture was noted in the previous sections as a potential deficiency. This failure mode was not observed in the 1994 Northridge Earthquake and has received relatively little attention by the U.S. engineering community. However, column splice failures in the 1995 Kobe earthquake contributed to several building collapses [4]. In addition, research in the late 1980s demonstrated that failure of partial penetration column splices is sudden and non-ductile [5]. NIST GCR 17-917-46v2 [6] provides a method to assess partial penetration column splices that evaluates three failure mechanisms – yielding of the base metal, rupture of the base metal over the thickness of the weld, or fracture of the weld. The capacity of the partial penetration splice is a function of the base metal strength and the weld toughness, with the level of toughness being the most important factor. Welds assessed using this procedure typically having capacities of 50% of F_y or less, leading to many column splices being flagged as susceptible to brittle fracture. Nonlinear models including fiber hinges to simulate column splice fracture indicated once a fracture occurred in one splice on a floor it

generally propagated to other columns on the floor, leading to a multi-story mechanism.

Ground Motion Sensitivity

Another interesting observation is how sensitive these building models were to different ground motion records. Reviewing the individual record performance of the building in Figure 4 shows a performance range from Immediate Occupancy to collapse, highlighting how different the maximum story drifts can be for structures that rely on brittle failure responses. Much less dispersion would be expected for more ductile responding structures. What is not shown in the plot is how sensitive the building can be to a 10% change in the scale factor. Figure 5 shows that when the performance of the building involves significant nonlinearity, slight changes in the ground motion intensity can cause a significant difference in the predicted building performance as demonstrated by the significant changes in drift from different ground motions.

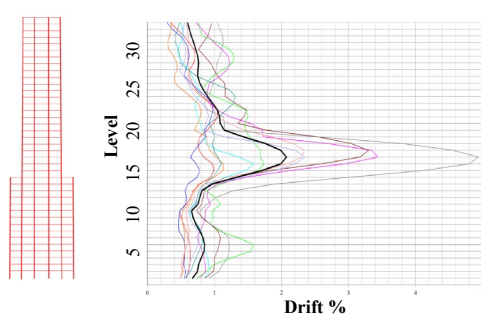


Fig. 4 - Variation in Drift Demands

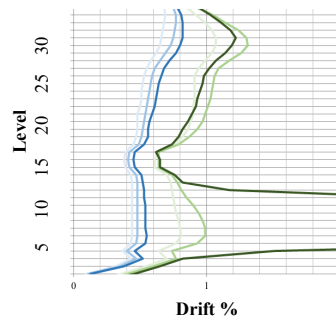


Fig. 5 – Drift Sensitivity to Ground Motions

Modeling Parameter Sensitivity

The pre-Northridge connection modeling approach can have a significant effect on the predicted seismic performance of the building. ASCE 41-17 provides a symmetric lumped plasticity model that is based on research from the SAC project. NIST GCR 17-917-45 [7] provides parameters for an updated lumped plasticity model and NIST GCR 17-917-46v2 [6] provides parameters for developing a fiber model that incorporates fracture of the flange welds and the couple between the flange in bearing with the bolts in the shear tab. The reasons the fiber model produces a less conservative result are that it accounts for the unsymmetrical nature of the pre-Northridge connection due to the influence of the concrete slab and because the residual strength based on the couple between the flange bearing in compression and the bolts in the shear tab provides a positive post yield slope prior to bolt fracture, as opposed to the horizontal residual strength region of the ASCE 41 curve.

Figure 6 shows the drift plots for a building modeled with lumped plasticity models, fiber models, and retrofit connections. There are two reasons the fiber model produces a less conservative result. First, it accounts for the unsymmetrical nature of the pre-Northridge connection. The influence of the concrete slab and the improved performance of the top flange weld when compared to the bottom flange often lead to a stronger negative moment capacity. In the case of this building, only the bottom flange fractures in the existing building's response. Therefore, a lumped plasticity hinge assuming the same fracture for the top and bottom flange reduces the frame stiffness by twice what an unsymmetrical hinge does. The second reason is the residual strength based on the couple between the flange bearing in compression and the bolts in the shear tab provides a second positive post yield slope until bolts begin to fracture, as opposed to the horizontal residual strength region of the ASCE 41 curve, as can be seen in the upper part of Figure 6.

The fiber hinge model requires knowledge of the weld toughness. The equations contained within NIST GCR 17-917-46v2 require similar input parameters based on Charpy V-notch toughness as those for the column splice fracture equations. While AISC 342 will permit the assumption of 10 ft.-lbs. for column splice welds, that toughness may be too optimistic for the bottom flange of pre-Northridge connections.

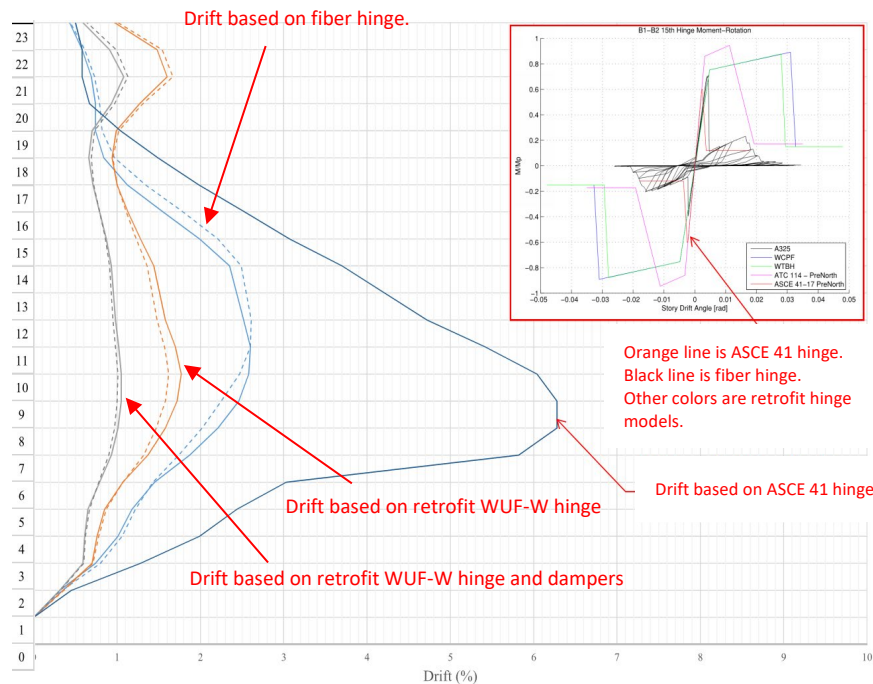


Fig. 6: Effects of Connection Modeling Parameters on Drift Response

Summary and Conclusions

Steel moment frame building design has evolved over the last century due to changes in construction technology and advances in analysis procedures and lessons from research testing and analysis. The most important conclusion that can be drawn from these evaluations is that there is no “archetype” welded steel moment frame. Most of the buildings are predicted to perform well in moderate earthquake intensities, up to the BSE-1E (225-year hazard) intensity. Well-designed buildings also might be expected to perform well in the BSE-1N. While none of the buildings met the ASCE 41 Collapse Prevention performance level for the BSE-2E (975-year hazard) or the BSE-2N (ASCE 7 MCE_R), the probability of collapse at that intensity varied significantly. It is at these larger hazards that individual building characteristics matter the most. The potential deficiencies involved and the predicted response of the building to those deficiencies can vary significantly by adjusting factors such as material properties, analysis techniques (linear vs. nonlinear), local connection modeling procedures for beam-column connections and column splices, and ground motions. The brittle nature of the response of these buildings (fracture of connections and splices) has a large amount of inherent randomness that makes it difficult to establish a high level of confidence in the predictive capacity of our present models. The response predicted by the models discussed in the paper are based on available information and can certainly be improved with better understanding of response of elements of the model, specifically the column-splices.

Acknowledgments

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COMPARING THE PERFORMANCE OUTCOMES OF A REINFORCED CONCRETE BUILDING DESIGNED TO CODIFIED NZ-JAPAN-AND CHILEAN DESIGN STRATEGIES

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Abstract

The observed differences between the recent performance of modern reinforced concrete buildings around the world has triggered the need to investigate design philosophies in various countries to determine the most effective approach for resilient building design. This work presents an analysis of a concrete frame building designed using seismic design specifications in three countries: New Zealand, Japan, and Chile. The objective of the work was to (1) compare the design practices in the three countries and (2) compare the seismic performance of the structures when subjected to earthquakes scaled to various seismic intensities. The seismic performance of the structures was evaluated using nonlinear dynamic time history analysis and results were compared in terms of design base shear, ductility, floor acceleration, and drift. The study reveals that the design practices in the three countries are similar in terms of ductility and detailing requirements with differences in strength and stiffness requirements. The results also show the difference between the seismic performance of the moment frames designed following the requirements of each country is significantly influenced by the level of seismic activity, with little difference between performance at low seismic demands and larger differences between performance at high seismic demands.

Introduction

In the last several decades, modern code-compliant buildings worldwide have faced various earthquakes, including the 1994 Northridge, 1995 Kobe, 2010 Darfield, 2011 Christchurch, 2013 Cook Straight, and 2016 Kaikoura earthquakes (Hare et al. 2012); (Holden et al. 2013); (Bradley et al. 2017). These events have prompted reflection within the structural engineering community regarding the most effective approaches to design buildings for seismic resilience.

Chilean and Japanese buildings have consistently exhibited close-to-operational performance following earthquakes, underscoring the resilience of reinforced concrete buildings in those countries (Lagos et al. 2012); (Massone 2013); (Sarrafzadeh, et al., 2017). Relative to New Zealand, concrete buildings in Chile and Japan have consistently demonstrated better performance and exhibited less damage. For instance, observations of the performance of reinforced concrete (RC) buildings after the 2016 Kumamoto earthquake in Japan showed buildings designed to Japanese specifications resulted in less damage and downtime compared to buildings following the 2010/2011 Christchurch earthquakes in New Zealand even though the earthquakes were of comparable magnitude (~Mw 6.2) (Sarrafzadeh, et al., 2017). A similar comparison can be made for Chile by looking at the 2010 Maule earthquake (Mw 8.8), where four buildings collapsed and only 40 buildings required demolition, whereas the 2010/2011 Christchurch earthquakes resulted in the demolition of 133 concrete buildings in the Christchurch CBD (Lagos et al. 2020); (Kam et al. 2011).

Based on the difference in damage and associated recovery time between Kumamoto, Maule and Christchurch, there is an interest in quantifying and comparing the performance of structures designed to

earthquake standards in different countries to help inform changes that can improve the resilience of concrete structures in New Zealand. However, previous comparisons of international design standards have demonstrated that differences in construction practices and methods for establishing seismic demands make it difficult to directly compare the performance of buildings designed according to different international standards. That is the objective of this work. Here, a four-storey case study building located in Wellington, NZ was designed using New Zealand material properties, but with scaled seismic demands and design requirements based upon New Zealand, Japanese and Chilean standards. The goals of this work are to identify the differences between the seismic requirements in the three countries, determine the factors that drive the design of reinforced concrete (RC) moment frames, and compare the impact on peak inter-storey drift and floor acceleration when designing to the three standards.

Background

Previous Code Comparisons. Prior research has highlighted the significant variation in seismic demands used for building design across different countries due to differences in site soil classification, parameters defining seismic hazard, and the resulting design response spectra (Khose et al. 2012). This fundamental difference in how seismic demands are established globally poses a considerable challenge in determining specific design strategies that result in better or worse building performance. Fenwick et al. (2002) compared the resulting seismic demands, stiffness, drift, and ductility of reinforced concrete moment frames designed to New Zealand, United States (US), and European standards. The findings indicated that attempts to compare the resulting designs were misleading due to the interaction and cumulative differences between the codes as the design advances through the calculation of seismic base shear, design actions, deflections, and ultimate building performance.

Moreover, a study conducted by Hampshire et al. (2013) focused on ductile reinforced concrete buildings designed according to US, European, Italian, and Brazilian standards. This research arrived at a similar conclusion, underscoring the challenge in comparing resulting designs because of disparities in the design spectra from each country. Specifically, variations in the shape of the elastic design spectra, without considering response modification factors, led to discrepancies exceeding 100% in some instances. To address the limitations noted in previous research that compared international design standards, the moment frames in this study were designed using the NZS 1170.5 elastic response spectrum (NZS 2004) as the starting point for seismic demands.

New Zealand Seismic Design. New Zealand structural design relies on member ductility and inelastic behaviour to dissipate energy allowing members to be designed for seismic forces lower than the forces corresponding to an elastic response. Seismic design follows the New Zealand Standard for Structural Design Actions part 5 (NZS 1170.5) for earthquake actions (NZS 2004) and the New Zealand Concrete Structures Standard (NZS 3101) for concrete structures (NZS 2006). Loads are combined following load resistance factor design (LRFD) with temporary (live) loads reduced when combined with permanent (dead) and seismic loads.

A probabilistic seismic hazard model is used to generate an elastic response spectrum based on average accelerations and 5% equivalent viscous damping. Elastic accelerations are then scaled by a structural performance factor (S_p) and ductility factor (k_d) to give the design accelerations corresponding to a 1/500-year return period earthquake at the ultimate limit state (ULS). Seismic base shear demands may be calculated using the equivalent static method for most regular structures with a ratcheting index less than 1.5.

For moment frame buildings, beams and columns are sized to meet a minimum 2.5% inter-storey drift limit at ULS with elastic drifts amplified to account for inelastic and p-delta effects. Reinforcement is designed following a capacity design approach based on beam overstrength moments to ensure ductile

behaviour and an appropriate hierarchy of strength (e.g. preventing joint failure or brittle shear failure). Additionally, at ULS the code limits the inelastic rotation of potential plastic hinge regions in beams, columns, and walls using an assumed effective plastic hinge length to calculate curvature. The resulting curvature is limited by a factor (K_d) based on the ductility of the structure. At the serviceability limit state, corresponding to a 1/25-year earthquake return period, inter-storey drifts are limited by the minimum drift capacity of individual building components to limit damage and maintain the ability for the building to be used as originally intended without needing any repairs.

Japanese Seismic Design. The Architectural Institute of Japan (AIJ) standards are used for seismic design of concrete structures in Japan (AIJ 2019) and are based on the government requirements of the Building Standard Law (BLS). The main differences between the seismic codes from New Zealand and Japan are in predicting seismic hazard, using the hazard to calculate seismic demands, and building performance objectives. These differences stem from general design principles that focus on strength and stiffness versus energy dissipation and ductility. Specifically, the procedure for seismic design in Japan for buildings under 60 m tall follows an allowable stress design with two verification levels. In the first level (Level 1), allowable stress is checked against elastic demands induced from a moderate seismic event comparable to a design limit state with a 1/500-year return period in New Zealand (Narapu et al., 2017). Allowable stress limits depend on loading and material and are equal to yield stress (f_y) for steel and two-thirds compressive stress ($\frac{2}{3} f'_c$) for concrete under combined gravity and earthquake loading. In the second level of design (Level 2) member demands are compared to capacity following a pushover analysis with checks on beam-column hinge development for a desired strong-column-weak-beam mechanism and a maximum allowable drift under Level 1 demands equal to 0.5% for frames.

Chilean Seismic Design. Reinforced concrete structures in Chile are designed according to the requirements set by NCh430 which uses the requirements in ACI 318 (ACI 2019) with some modifications specified by amendment DS60 (INN 2008). Seismic forces and drift limit requirements are set by NCh433 with amendments outlined in DS61 (INN 2009). Although the code only explicitly mandates a life safety performance, Chilean design practice generally results in buildings with immediate occupancy performance levels by following two main principles that differ from New Zealand. The first assumes every building will experience at least one large earthquake (M_w 8+) during its lifetime (Lagos et al. 2012). This is reflected by an elastic response spectrum that generates significantly larger acceleration demands and corresponding seismic base shears compared to New Zealand. Seismic forces in Chile are again increased by a 1.4 load combination factor to reach an MCE (maximum considered earthquake with a 2500-yr return period) design level when combined with dead and live loads. In addition, Chilean buildings have a very strict drift limit of 0.2% drift limit compared to a 2.5% drift limit in New Zealand. However, it is worth noting that this drift limit is checked using gross section properties.

Methodology

Case Study Building. An RC moment frame and shear wall building designed and tested at the E-Defense shake table facility in Japan in 2010 was selected as the case study building for this work. The structure was a full-scale 4-storey building with two-bay perimeter seismic moment frames in the longitudinal direction and shear walls in the transverse direction, shown in Figure 1 (Nagae et al. 2015). Only the moment frame direction was considered for this assessment; details on the shear walls and gravity members are not included. The case study building, and subsequent re-designs were designed as a typical office building. Note that this structure was selected as the base case study building due to the availability of experimental data that was used to validate the numerical techniques described in the following sections.

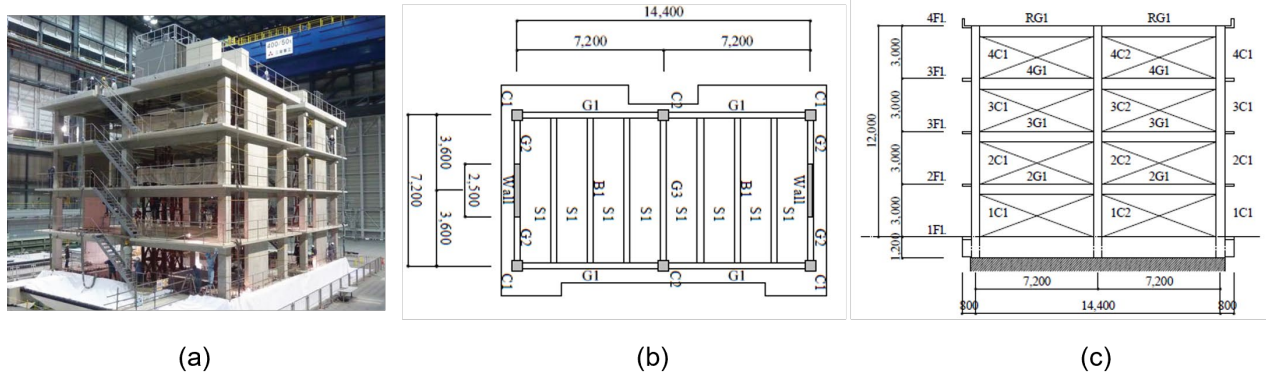


Figure 1. Case study building; (a) photo of building at E-Defense, (b) plan view, (c) moment frame elevation (modified from Nagae et al., 2015).

Moment Frame Designs. A single moment frame from the case study building was re-designed using New Zealand material properties and seismic hazards for a building located in New Zealand but with scaled seismic demands and design requirements following New Zealand, Japanese and Chilean standards. Elastic base shear demands were determined using the NZS1170.5 elastic response spectrum for a building located in Wellington, NZ with site class C soil conditions. As mentioned previously, all designs utilize New Zealand material properties equal to 30 MPa for concrete compressive strength (f'_c) and 500 MPa and 300 MPa for yield strength (f_y) of beam and column reinforcement, respectively. Table 1 summarizes the New Zealand, Japanese and Chilean moment frame designs in terms of design base shear, controlling load combination, section size, and longitudinal reinforcing.

Figure 2 shows the sum of column-to-beam moment strength ratios at each joint with the New Zealand ratios in black on the left side of the joint and the Japanese (in red) or Chilean (in green) design ratios on the right side of the joint; a ratio greater than 1.0 indicates columns are stronger than beams. The ratios are different for the outer joints on the right and left sides of the frame because the T-beams have different positive and negative moment capacities, and these values will reverse with reversed earthquake loading. The New Zealand design has mostly lower column-to-beam moment strength ratios compared to the Japanese and Chilean designs due to weaker column sections but relatively similar strength beam sections. In general, however, the ratios for all designs are similar as each country utilizes capacity design procedures with some form of weak beam-strong column requirement.

Table 1. Summary of moment frame designs

Design Feature	New Zealand Design	Japanese Design	Chilean Design
Elastic base shear (kN)	998	1699*	1421*
Reduction factor (equivalent ductility, μ)	5.7 (4)	5 (3.5)	7 (5)
Design base shear (kN)	175	340*	203*
Load combination	D + 0.3L + E	D + L + E	1.2D + L + E
Period, T_1 (s)	0.86	0.54	0.68
Beam size (b x h) [mm]	300 x 500	300 x 600	300 x 525
Beam reinforcement (no. top/bot)	(5/5) D20	(3/6) D20	(6/6) D20
Column size (b x h) [mm]	500 x 500	500 x 500	525 x 525
Column reinforcement (total no.)	(16) HD16	(10) HD20	(14) HD20

* Base shear demand uses gross section properties

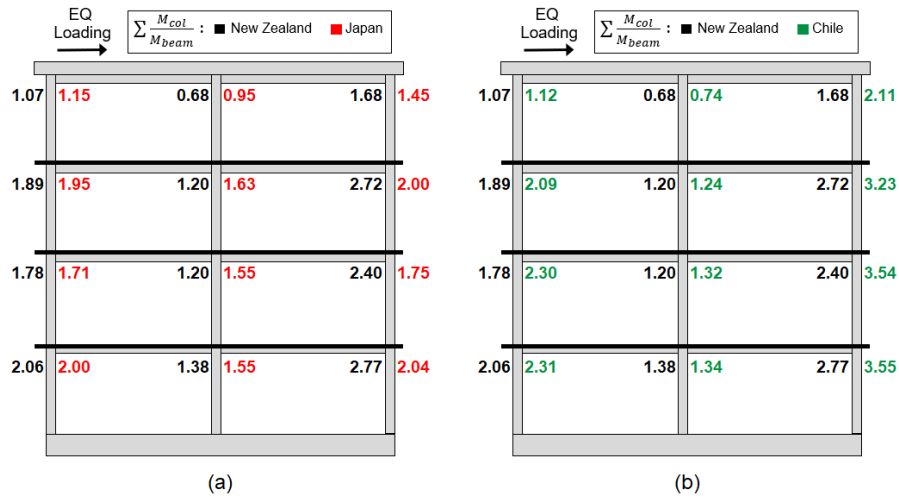


Figure 2. Column-to-beam moment strength ratios for (a) New Zealand and Japanese moment frame designs and (b) New Zealand and Chilean moment frame designs.

Structural Modelling and Analysis. To quantify and compare the performance of the three designs, a probabilistic seismic hazard analysis was performed using nonlinear two-dimensional (2D) models developed in the opensource structural analysis software OpenSeespy (Zhu et al. 2018). A lumped moment rotation approach was used because previous research has shown it can effectively capture inelastic drift, storey acceleration and collapse (Haselton et al. 2007). Beams and columns were modelled as elastic with an effective stiffness equal to 0.4 times gross moment of inertia, and nonlinear moment rotation hinges at member ends with hinge behavior based on a peak oriented hysteric response defined using the parameters specified in ASCE 41-17 Tables 10-7 and 10-8 for beams and columns respectively (ASCE 2017).

A site specific multiple stripe analysis (MSA) was performed for Wellington at a shear wave velocity to 30 m depth (V_{s30}) of 250 m/s using the National Seismic Hazard Model (NSHM), a New Zealand-specific rupture forecast model (Gerstenberger et al. 2022). Ground motions were selected and scaled to match the target spectrums for each intensity measure using the PEER NGA-West2 ground motion database (Ancheta et al. 2013) with computed weighted mean squared error (MSE) scaling. The resulting seismic hazard curve for each intensity measure and an example ground motion suite for intensity measure 3, IM3, with a 10% probability of exceedance in 50 years (e.g., a 500-year return period corresponding to ULS in Wellington) is shown in Figure 3.

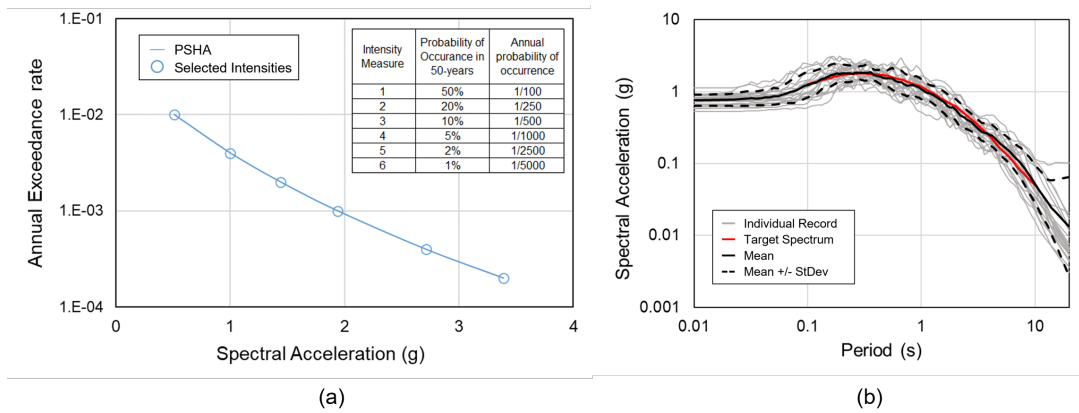


Figure 3. Multiple Stripe Analysis procedure; (a) probabilistic seismic hazard (PSHA) curve for Wellington $S_a(0.7s)$ subsoil class C and (b) ground motion suite for intensity measure IM3.

Results

Figure 4 shows the MSA results for the New Zealand, Japanese, and Chilean designed moment frames analysed for each ground motion at increasing intensity measures. Each point on the MSA plot represents the peak interstorey drift from non-linear time history analysis at the corresponding spectral acceleration intensity at the fundamental period of the structure $S_a(T_1)$. Each point represents the peak value from a single ground motion scaled to increasing intensity measures from $S_a(0.70s) = 0.5g$ to $S_a(0.70s) = 3.4g$. The vertical dashed line represents collapse at 10% drift.

The mean interstorey drift and total acceleration results from MSA are found by taking the average values from all ground motions at a single intensity measure at each floor. Results for intensity measures IM1, IM3, and IM5 are shown in Figure 5. As expected, the stiffer Japanese and Chilean buildings resulted in lower drifts and larger accelerations compared to the New Zealand moment frame, with little difference between values at lower IMs and larger differences at higher IMs.

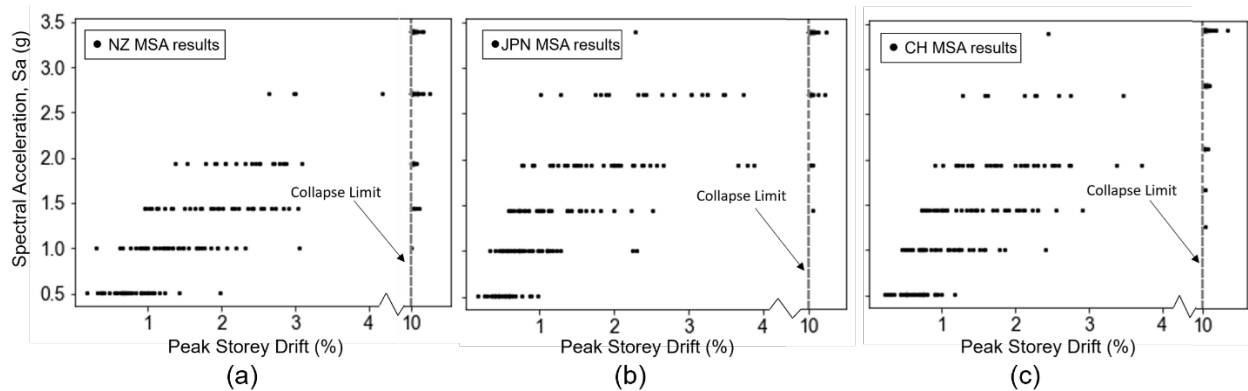


Figure 4. MSA Results for (a) New Zealand (b) Japanese and (c) Chilean design moment frames.

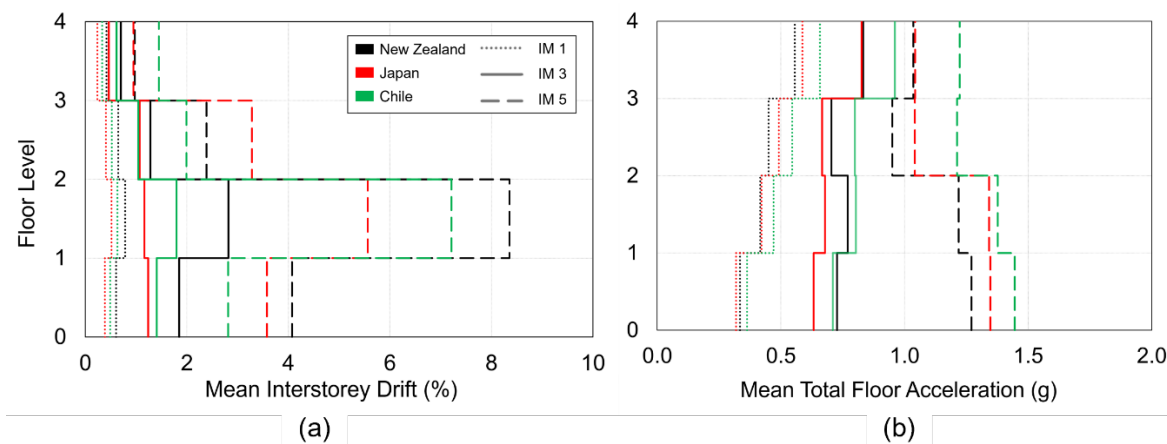


Figure 5. MSA results; (a) Mean interstorey drift per floor and (b) mean total acceleration per floor

Conclusions

Based on the case study discussed here, it was found that:

- New Zealand and Chile follow similar seismic design procedures with slight differences in strength and stiffness requirements. The Japanese design approach is unique in that it includes a two-stage design process and does not explicitly require that elements are capacity protected to resist beam overstrength.
- For low-rise RC buildings located in high seismic regions in New Zealand, moment frame design following New Zealand requirements is controlled by strength and minimum detailing, while the same moment frames following Chilean and Japanese requirements are controlled by a 0.2% drift limit and Level 1 allowable stress limits, respectively.
- Despite having similar design base shears, moment frames designed to Chilean specifications will have larger section sizes relative to New Zealand resulting in reduced peak interstorey drifts, but similar peak floor accelerations.
- Moment frames designed following Japanese specifications will have the largest beam sections resulting in the lowest peak interstorey drifts, but, again, similar floor accelerations compared to New Zealand and Chilean designed frames.

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ELASTIC RESPONSE SPECTRA – THE USE OF AMBIENT EXCITATION AND ITS DECONSTRUCTION FOR STRUCTURAL ASSESSMENT

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Abstract

Elastic response spectra are commonly used in codes of practice as a guide to the expected response of structures when they are excited by seismic activity. While this allows the calculation of the response of a structure given a known amount of energy dissipation, these curves have been averaged over a relatively short time. This averaging process depends on the collection of response data and by accumulating information about soil conditions and about the structure itself the possibility of determining the expected response becomes viable.

There are problems with this approach which are independent of the averaging process included in the excitation spectrum. These are the non-linear behavior of the structure, the effect of soil/structure interaction, and the ductile behavior of the structure at large amplitudes of response. For the structure the non-linearities include the variation of frequency and damping with increasing amplitude of response. The soil/structure interaction has little effect on the structural response at small amplitudes of response. These effects have made the estimation of such nonlinear behavior difficult to measure during seismic activity.

At small amplitudes of response, the non-linear characteristics of the structure are still non-linear (as demonstrated here with full-scale data) and allow the estimation of frequency and damping non-linearities as well as eliminating the effects of soil/structure interaction. This allows the reduction of variance errors and leads to the use of spectral information with a similar form to those normally used in design and with a far superior precision.

Introduction – Elastic Response Spectra and their uses in Seismic engineering

Elastic response spectra (ERS) are produced from the theoretical considerations of a series of single degree of freedom systems that are subjected to earthquakes of different magnitudes and the curves are a representation of the maximum magnitude experienced by these systems as a function of the period of the oscillations. The elastic response spectra are then used to produce a design response spectrum in which the response of a structure can be calculated. The design response spectrum is modified by the use of a series of parameters that are specific to the site being analyzed. (Newmark, 1959; Chopra, 2007).

There are several problems with this approach. Whilst the different soil parameters are well established others such as the non-linear damping and frequencies of the structure being considered are less well considered in seismic literature. Additionally, the difficulty of moving data sets forward are severely disrupted by the infrequency of the occurrence of earthquakes. The resulting variance in spectra produced from single or groups of earthquakes still prone to wide variance errors. This makes the precise assessment of a design response spectrum subject to some errors in calculating structural response that can cause much larger structural response than would otherwise be assumed. The non-linear structural damping is a particular example that is considered in more detail in this paper.

The difficulties have been discussed in detail by (Bradley et al, 2017) in which the authors conclude that ‘Elastic Response Spectra Normalisation of the response spectra at soil sites by those at reference rock sites illustrated spectral amplifications which are narrowband in nature, and exceed (in some instances significantly) those prescribed by the seismic design standard, NZS1170.5:2004’.

The basic problem with ERS’s is that they are dependent on the use of a series of single degree of freedom systems (with different natural frequencies (periods of oscillation)) with a constant value of damping and their theoretical response based on the soil conditions at the site being analysed. The objective of ERS is to combine the spectrum of input forces with that of the structural spectral response (Biot, 1934; Housner, 1941). The methodology relies on the accurate analysis of spectra from sites that are similar to the one being analysed theoretically or from measurement at a similar site using earthquake induced data.

Because the time of the incoming excitation is short then conventional spectral analysis cannot be used since the variance errors associated with a random process (such as wind, earthquake or flooding) are so large that the design response spectrum becomes randomized. The resulting spectrum used in codes either by averaging a series of ERS’s from different earthquakes representative of the site being assessed or by averaging the values from a single event at or near the site. The problem gets larger for more intense earthquakes because their occurrence is less frequent, and their duration is still quite short (just a few minutes when a requirement of up to three hours is required for a variance error of 10%). (Jeary and Winney;1972)

The problem then gets larger because the damping ratio of a structure is non-linear. Additionally, the damping starts at a small value and rises to a larger value when the amplitude of response gets larger (Jeary, 1986). In turn this means that the loss of vibration energy in the structure is dissipated more slowly than it is at greater response amplitudes. The implication of this is that the response increases disproportionately quickly when the structure’s response is at a small amplitude of response. At larger response amplitudes the damping ratio rises and the amount of energy dissipated increases. The overall effect of this is that the structure spends a longer time at larger response amplitudes than would be expected from that of a single degree of freedom response. (Jeary, 2003).

1. Non-linear damping and fracture mechanics

The classical version of describing damping as viscous damping has, for decades, established a misunderstanding of the behavior of structures that exhibit non-linear characteristics. This is because there is no mechanical equivalent that results in a viscous model of damping, and viscous dampers are not common in structures. The embarrassing predictions of a mathematical model of damping are that the natural frequency will become a higher frequency at larger amplitudes of response. The friction model gives a reduction in frequency at greater amplitudes, but a single element is unrealistic. A stick slip model was introduced (Wyatt, 1977) in which a large number of such elements was shown to produce a holistic model that exhibits equivalent viscous damping. This type of model also models a realistic version of the materials used in structures and allows the mature field of fracture mechanics to be used in assessing damping.

2. The difficulties of establishing non-linear structural parameters from limited data

When the amplitude of response increases for a structure that has non-linear characteristics, the frequency of the resonance decreases and the quantity of energy dissipation (measured as damping) increases. With classical spectrum analysis (Bendat and Piersol) the analysis bandwidth is chosen to resolve the bandwidth of the response. However, if the frequency of resonance changes, then the bandwidth of the response will appear to be wider (which is resolved as a larger damping ratio) (Jeary, 1981).

If there are copious amounts of data available then the analysis bandwidth can be reduced and the resulting spectrum will reflect changes in the level of response at changing amplitudes.

3. Small amplitude response and the extraction of non-linear characteristics

The so called ‘linear-elastic range’ allows the use of many engineering short cuts to be used in everyday practice. These include the use of Hooke’s law and Young’s modulus of elasticity. Most modern engineering practice could not exist without reference to these two concepts. However, if the concept of energy dissipation includes the stick-slip mechanism then it is impossible to use Hooke’s law and Young’s modulus except on the larger scale when the amalgamation of many stick slip mechanisms results in an equivalent viscous damping model and allows analysts to use these concepts. As an added bonus the mechanism of stick slip allows the resolution of some mathematical predictions that are at variance with theory (Clough and Penzien, 2002).

In the late 1960’s NASA had a problem with fatigue on the original space shuttle and arranged an algorithm that could be used to produce an early alarm when fatigue damage became significant (Cole, 1968). The algorithm was modified in the 1980’s to allow the establishment of the measurement of damping as a function of amplitude for structures in general (Jeary, 1981).

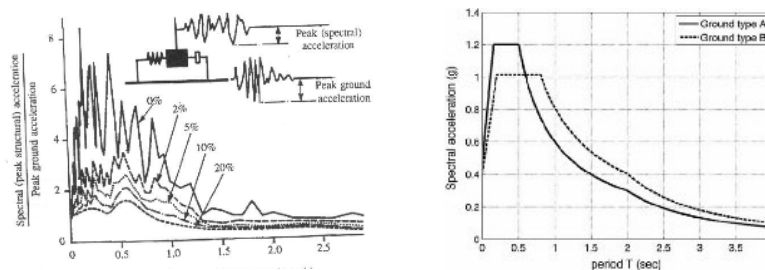


Fig. 1. Early measurements of ground motion and smoothed design curves

4. The effect of a precise knowledge on response estimation - the under-estimation of response

Measurements from the response of three buildings (Rabiespour et al 2024) showed that the response of these buildings was sometimes much larger than that predicted using the spectral approach. The authors conclude that ‘history shows structural collapses encourage

researchers/engineers to use spectral analysis, but it seems there is no clear consensus on how it should be done’.

The authors show that the response spectrum, in assuming design using elastic theory, severely underestimate the response of a building when ductility is considered.

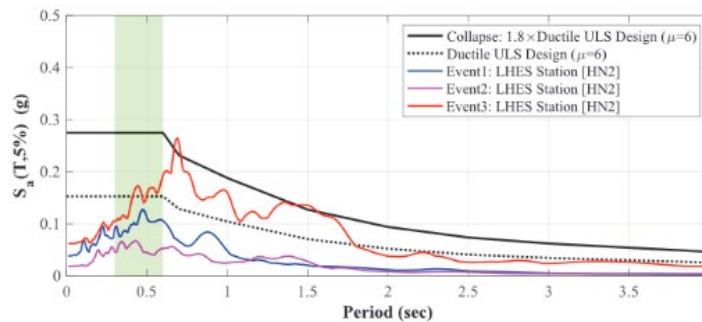


Fig 2. Spectral response measurements of the BNZ Building compared with NZ code predictions

Figure 2 shows that the BNZ building was subjected to forces that were larger than the predictions of NZ11705:2004 (Importance level: 2; Site subsoil class: D; Ductility factor: 6 (Siddiqui et al., 2019)). The situation gets even worse when non-linear damping is considered. Figure 2 shows the use of a 5% of critical hysteretic damping value being used in NZ1170. When the model adopted from site measurements (Wyatt, 1981) then this value is shown to be much larger than actually happens in practice (Tamura et al, 2000). Eventually the BNZ building was demolished because of severe structural and non structural damage. (Rabiespour, et al, 2024).

The conclusion is that the response of structures to earthquake action must be measured and the measurements should be input to current codes of practice.

5. induced vibration testing and the precise measurement of frequency and damping compared with extraction from ambient data

A series of induced vibrations tests was conducted by the UK’s Building Research Establishment on buildings and dams. The four unit, synchronized vibrator system was capable of delivering one tonne at one cycle per second and was accurate to 0.001 Hz.. The system was used on forty different structures.

Table 1. Induced vibration tests at South Stoneham House

South Stoneham House Induced Vibration Tests						
Force	Period	Amplitude	Amplitude	Damping	Modal Mass	Stiffness
N p-p	Secs	%g	mm p-p	% critical	x10 e6 Kg.	MN/m
14582	0.766	1.836	0.432	2.40	1.563	105.4
11766	0.766	1.491	0.357	2.34	1.595	107.2
9086	0.760	1.164	0.250	2.29	1.618	110.4
6226	0.760	0.818	0.141	2.28	1.576	107.6
3398	0.757	0.455	0.046	2.23	1.582	109.0

The induced vibration tests conducted by the Building Research Establishment in the UK allowed the precise measurement of non-linear characteristics that can be seen in the table above. It was also possible to conduct tests of human volunteers to measure their sensitivity to vibration.

The table above shows that even in the linear elastic zone the natural frequency and the damping values change as the amplitude changes. Nevertheless, the calculation of the modal mass of the structure remains unchanged throughout the measured amplitudes, when the mass is calculated by using the measured parameters (force, displacement, frequency and damping). This shows that the basic equation is robust and its use to measure displacement per unit force (using Young's Modulus for the entire building) is justified as is the current approach using spectra.

$$\frac{X_r}{F_r} = \frac{1}{8\pi^2 f_r \zeta_r M_r} \quad (1)$$

Where for mode r, X is the displacement caused by a force F acting on a structure having a fundamental natural frequency of f and an associated damping ratio of ζ .

6. Information obtainable from the SDRC curve with precise linear zone measurements

The linear elastic zone is the part that obeys Hook's law with a rate of rise termed the Young's Modulus of elasticity. After the elastic limit is reached the area dominated by ductility takes over. If the system is affected by an event such as a storm, earthquake or flood then damage may occur and if the same structure is tested again then the force per deformation characteristic will be changed to that shown in red in fig 3 below.

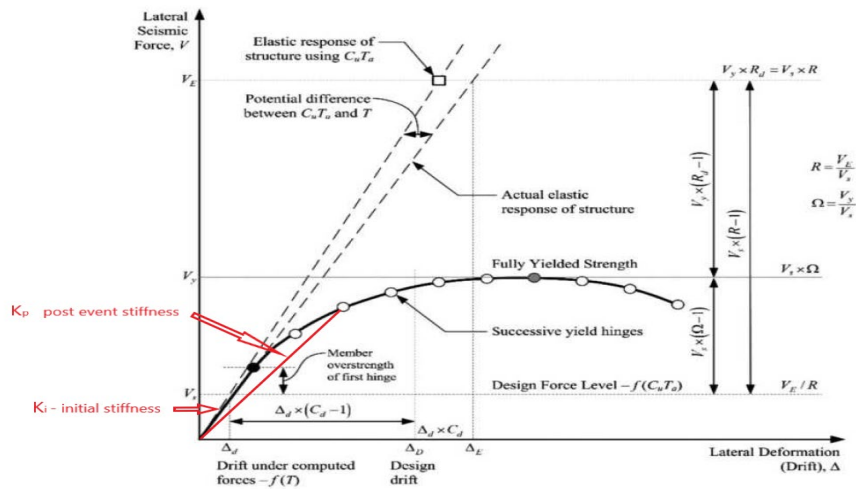


Fig 3: SDRC curve showing displacement against force for a typical structure

The change represented by the red line in figure two reflects a reduction in the natural frequency of the fundamental mode of vibration. Accompanying this is an increase in the damping associated with the fundamental mode of vibration. This is caused by a mechanistic correlation between the frequency and the damping, when a loss of stiffness (and hence a lowering of frequency) when micro-lesions lengthen thereby increasing the damping value through friction.

7. Benefits to seismic engineers for accurate estimation of structural response

The effect of using an unvarying value of damping leads to a structure with non-linear damping spending a longer time at larger amplitudes of response. As such the amount of damage is greater than is expected by a strict following of current code requirements.

In the Kaikoura earthquake on the South Island of New Zealand as many as 30% of structures in Wellington were unexpectedly damaged despite being designed to the requirements of the New Zealand earthquake code.

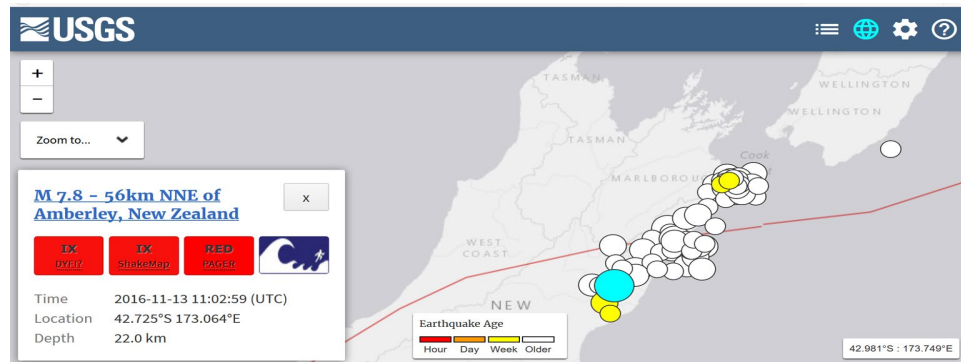


Fig 4: Kaikoura earthquake (blue) and aftershocks (white) approaching Wellington

Recent advice suggests that there are many unknowns in assessing the maximum likely response to an earthquake, particularly the soil condition, in addition to the uncertainty associated with the damping value, but more importantly with the lack of any consideration of the non-linear damping characteristics when considering design.

Conclusion

The use of spectral techniques which are based on linear elastic response spectra for the design in earthquake prone areas are the cause of the underestimation of lateral seismically induced forces for two main reasons. These are that the neglect of ductility in the elastic response spectra is a dubious methodology and the use of damping values that represent the response of a building using linear elastic approximations for actual behavior and not conservative. When this is coupled with the non-linear damping behavior of real structures, then buildings spend a longer time at large response amplitude than are expected using spectral elastic response spectra. The solution is to make a series of measurements of the real response of structures urgently.

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PRACTICAL TECHNIQUES FOR ELASTIC SECONDARY STIFFNESS DESIGN TO PROVIDE SEISMIC STABILITY AND ENHANCE SEISMIC RESILIENCE

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Abstract

Although ductility is a foundational attribute of seismic design, it is not sufficient to provide seismic stability. Instead, persistent positive tangent stiffness that obviates the destabilizing effects of gravity is the essential attribute that prevents earthquake-induced collapse of a structure even at large inelastic drifts. Many ductile seismic force-resisting systems (SFRS) develop distributed yielding of fuse elements that leads to gradual softening, maintaining positive stiffness over a significant drift range. This region of positive secondary (post-yield) stiffness provides favorable performance, although this benefit is not recognized in current standards and is thus not typically considered in design. Secondary stiffness varies between different SFRS and even within different realizations of a particular SFRS. Thus, instead of allowing the post-yield stiffness of a system to be an uncontrolled property, a more rational and reliable approach is to directly design an elastic secondary stiffness system as a companion to the primary SFRS. The secondary system is designed to have sufficient stiffness to offset the negative geometric stiffness effects while having sufficient flexibility (considering its strength) to remain elastic to large drifts. In addition to providing reliable seismic stability, the persistent elastic restoring force of the secondary stiffness system has the benefit of bringing the structure back toward its initial plumb position during dynamic earthquake response, which reduces transient and residual drifts. This paper describes a framework for designing elastic secondary stiffness systems to improve seismic performance and seismic resilience. Archetype building designs are presented and their performance is assessed with nonlinear analysis.

Introduction

The stability of a building against earthquake-induced sidesway collapse depends on the seismic force-resisting system (SFRS) equilibrating not only the effect of horizontal seismic accelerations, but also the destabilizing effect of the vertical gravity loads acting at horizontal drifts caused by those accelerations (the “P-Delta effect”). The P-Delta effect is equivalent to an additional overturning moment at each story caused by the gravity load (P) times a moment arm equal to the story drift (Δ). Figure 1 shows a schematic diagram of seismic base shear versus lateral drift for an idealized structure with story height of h . The P-Delta effect is shown as a negative stiffness line with slope $K_{P\Delta} = P/h$. The elastic structural stiffness without the P-Delta reduction is shown as a line with slope K_{e1} , and with the P-Delta reduction as a line with slope K_{e2} . Thus, K_{e2} is the effective elastic stiffness of the system in the presence of the vertical load. In the elastic range, the P-Delta effect can be considered as a reduction in effective stiffness, and both the resistance and the P-Delta effect increase linearly with drift. This permits a straightforward determination of the elastic P-Delta amplification based on the effective stiffness (K_{e2}) and equilibrium at the second-order drift. Using the stability coefficient $\theta = K_{P\Delta}/K_{e1}$, the associated amplifier $B_2 = 1/(1-\theta)$ can be used to evaluate the elastic P-Delta effect. Current code provisions in the United States (ASCE 2022) require consideration of the elastic P-Delta effect in seismic design.

However, seismic response at the Design Earthquake (DE) involves significant inelastic ductile behavior. In the inelastic range, the P-Delta effect continues to increase linearly with displacement whereas the

resistance does not. Thus, the P-Delta effect becomes proportionally much larger in the inelastic range than it is in the elastic range. Furthermore, an amplifier such as B_2 , or a more rigorous elastic analysis formulation that directly considers geometric stiffness, does not accurately capture the inelastic P-Delta effect. Figure 1 shows inelastic base shear versus drift curves, with and without the P-Delta reduction in effective strength. The figure shows that in the current design method, both required strength and estimated inelastic drift are amplified by the elastic second-order effect, represented by B_2 . The inelastic drift also utilizes the deflection amplification factor C_d , which represents the expected drift of a system designed for forces reduced by the response modification coefficient R , which accounts for ductility and overstrength. The P-Delta effect consumes a larger proportion of the inelastic lateral resistance at larger drifts, ultimately leading to negative effective stiffness as the SFRS reaches its peak strength. This condition can lead to a significant increase in drift demands (Gupta and Krawinkler 2000), making P-Delta an appreciable concern for seismic stability.

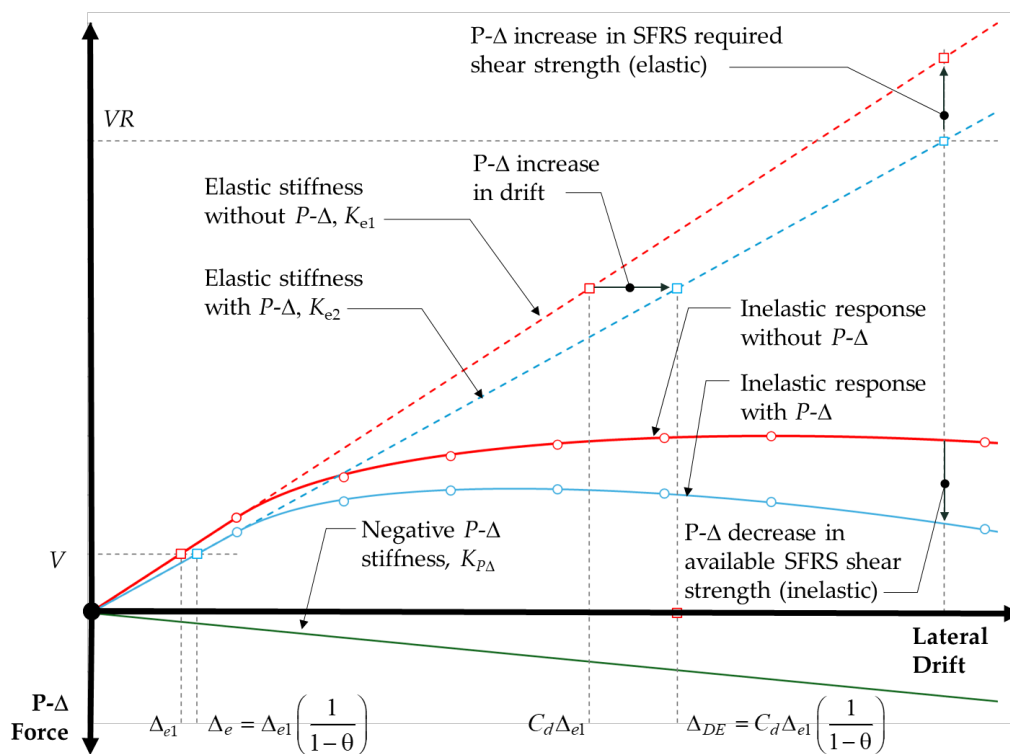


Figure 1. Seismic base shear versus lateral drift.

Since a portion of the SFRS lateral resistance is consumed in resisting the P-Delta effect, and thus the strength available to resist applied seismic loads is reduced, SFRS properties must be adjusted to compensate for the strength loss at inelastic lateral drift. Provisions in American Society of Civil Engineers (ASCE) 7-22 (ASCE 2022) lead to slightly larger design strength, based on the P-Delta effect at the elastic drift Δ_{e1} shown in Figure 1. However, this approach is not consistent with the expected inelastic drift at the Design Earthquake, Δ_{DE} , and it does not appreciably influence member proportioning or seismic response (Mathur et al. 2012). Other elements of current provisions, such as controlled strength loss in ductile systems at large inelastic drift, indirectly seek to address the P-Delta effect in the inelastic drift range. However, to more directly address the increased influence of the P-Delta effect in the inelastic drift range, a strength-based approach would need to consider a reduced effective stiffness. Such an enhanced strength-based approach – compared to current provisions in ASCE 7-22 – would improve seismic performance but would not directly address the most fundamental parameter that

influences seismic stability, namely, persistent secondary (post-yield) tangent stiffness (Fahnestock 2022). This paper does not examine an enhanced strength-based approach for considering the P-Delta effect, but rather presents an approach for designing elastic secondary stiffness to improve seismic performance and seismic resilience.

Secondary Stiffness

Background. The importance of secondary stiffness has been known for decades. For example, MacRae (1994) identified the ratio of post-yield stiffness to elastic stiffness as the major parameter that controls dynamic stability, and Gupta and Krawinkler (2000) demonstrated the deleterious effects of global negative stiffness after significant inelastic response. Although secondary tangent stiffness has not been implemented in design provisions as a parameter that is explicitly proportioned, ductile SFRS typically develop distributed yielding that leads to gradual softening and a significant region of positive post-yield stiffness. This post-yield stiffness plays an essential role in favorable seismic performance, yet it varies between different SFRS and even within different realizations of a particular SFRS. In fact, code-compliant ductile SFRS can be configured in a way that minimizes post-yield stiffness and leads to performance that is lower than desired (Zaruma and Fahnestock 2018). Instead of allowing the post-yield stiffness of a system to be an uncontrolled parameter that varies based on other design decisions, a more direct and reliable approach is to design an elastic secondary stiffness system as a companion to the primary SFRS. The secondary stiffness system is designed to be flexible so that it can remain elastic at large drifts, but it is also proportioned to be stiff enough to combat the negative geometric stiffness effects. A recent investigation of a designed elastic secondary stiffness frame (SSF) system using diagonal braces in the context of a steel-framed building demonstrated robust results (Hariri et al. 2024). The present study is exploring an elastic SSF system using flexural response in a steel-framed building.

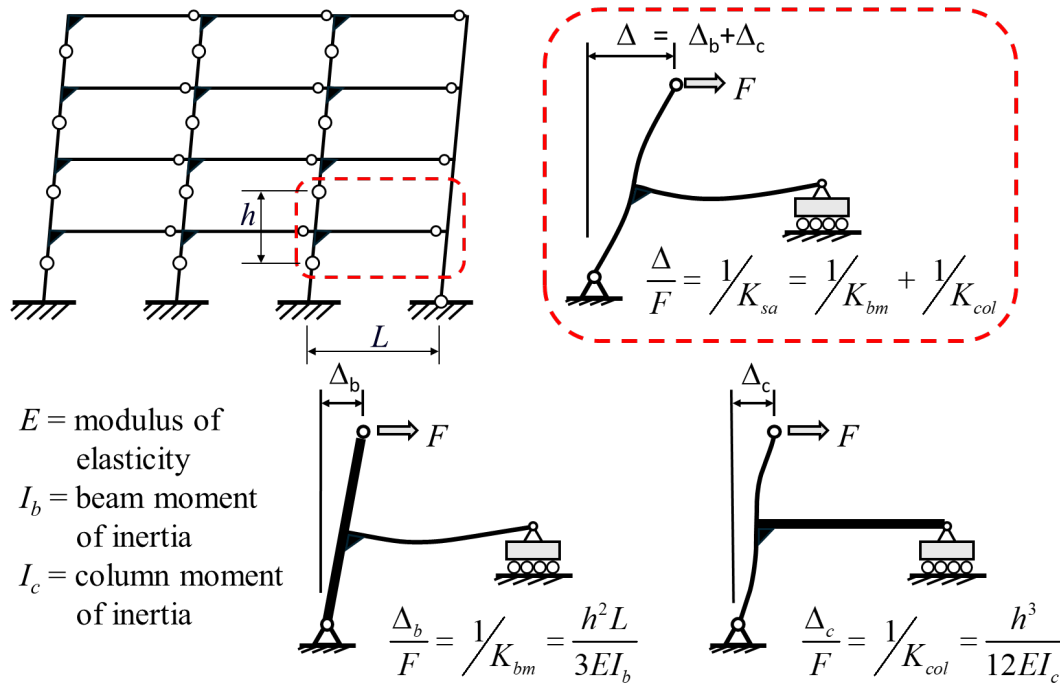


Figure 2. Schematic gravity frame and subassembly for designing secondary stiffness properties.

Implementation. Since steel-framed buildings typically have a relatively small number of SFRS bays and significantly more gravity-only bays, the gravity framing can be leveraged for elastic SSF design. Figure 2 illustrates a schematic multi-bay multi-story gravity frame where the beam-column connections are pinned (gravity shear connections), represented by a small open circle. To mobilize this gravity framing as an elastic SSF, the left end of every beam is changed from pinned to fixed (moment connections), as represented by a filled triangle. Assuming inflection points at mid-height of every column (represented by a large open circle) and the right-side column providing negligible stiffness, a subassembly is used for designing the SSF properties. The approximate lateral stiffness of the subassembly, K_{sa} , comprises beam stiffness, K_{bm} , and column stiffness, K_{col} , components as defined in Figure 2, with $K_{sa} = K_{bm}K_{col}/(K_{bm}+K_{col})$. The total number of subassemblies in a story, n , must have cumulative stiffness nK_{sa} that exceeds the destabilizing effect of gravity in the story, P_{story}/h . This requirement establishes the minimum stiffness for the SSF. Since the SSF must remain elastic up to a large drift, a maximum stiffness requirement is established for the subassembly using the estimated drift at the Maximum Considered Earthquake, Δ_{MCE} . Thus, K_{sa} must not exceed $M_{pb}/h\Delta_{MCE}$ and $2M'_{pc}/h\Delta_{MCE}$ where M_{pb} is the beam plastic moment capacity and M'_{pc} is the column plastic moment capacity adjusted for axial-flexural interaction. The minimum frame stiffness depends on the number of frames used (n), while the maximum stiffness depends on the members selected. Using more frames to achieve the same cumulative system stiffness (nK_{sa}) permits selection of lighter members and may facilitate meeting the maximum-stiffness requirement.

Theme Building

As a case study for application of the SSF concept in steel moment frames, a 12-story office building (Figure 3), was adapted from a previous 16-story design (Harris and Speicher 2015). This structure employs two three-bay Special Moment Frames (SMFs) in the east-west direction and two two-bay Special Concentrically Braced Frames (SCBFs) in the north-south direction. This study considers only the east-west direction. All interior bays are gravity framed, and these bays are strategically used to provide the SSF, which is named the secondary-stiffness moment frame (SSMF). Since this SSMF is designed to remain elastic, it does not require special detailing or capacity-based proportioning like the primary SMF. As shown in Figure 3, two three-bay SSMFs are used for this case study. The building is located in Los Angeles, California (33.821° N, 117.818° W) and designed using the ASCE 7-22 equivalent lateral force (ELF) procedure along with the appropriate steel standards as follows: American Institute of Steel Construction (AISC) 360-22 (AISC 2022b), AISC 341-16 (AISC 2016), and AISC 358-22 (AISC 2022a).

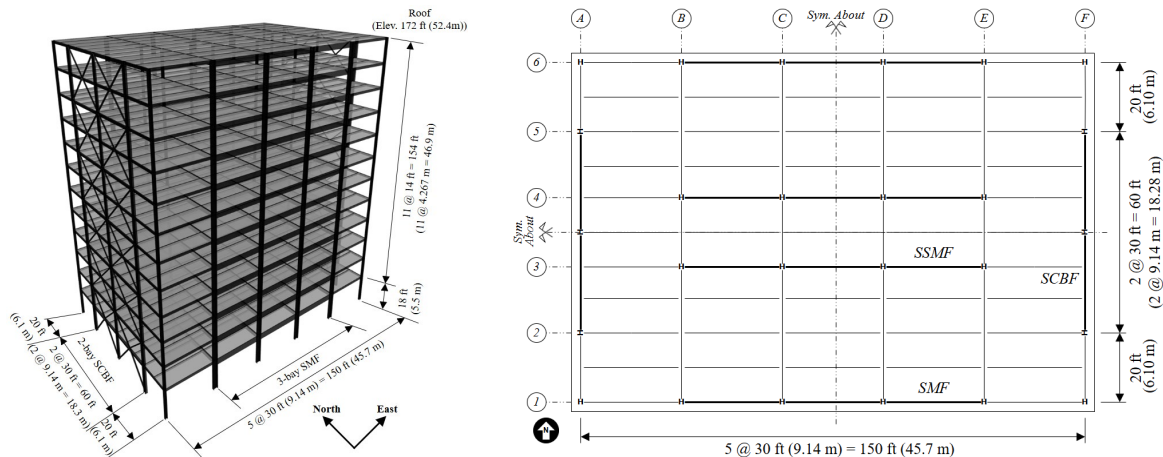


Figure 3. 12-story theme building: isometric view and floor plan.

Numerical Modeling Methodology

Analyses were executed using the OpenSees platform (McKenna et al. 2010), including elastic analyses for structural proportioning and nonlinear inelastic static and dynamic analyses for performance assessment. All models are two-dimensional based on structural symmetry and a rigid diaphragm assumption. Nonlinear models follow established procedures (NIST 2017, Speicher et al. 2020), where all beams and columns in the SMFs and SSMFs are modeled as elastic elements with nonlinear rotational springs at the ends, and nonlinear panel zone behavior is represented at all beam-column joints. Modal damping of 2 % is used, supplemented by 0.1 % stiffness-proportional damping to mitigate higher modes of vibration. A leaning column captures the destabilizing effects of gravity, and it has flexural stiffness and strength properties associated with the summation of all columns tributary to the SFRS.

Seismic Design Case Study

The design matrix includes three cases, all of which satisfy a 2 % story drift limit per ASCE 7-22. The first case, referred to as Baseline SMF (two three-bay SMFs, on Column Lines 1 and 6), is proportioned using first-order elastic analysis and thus does not account for the P-Delta effect. The second case, called Current Practice SMF (two three-bay SMFs, on Column Lines 1 and 6), is proportioned using second-order elastic analysis, which accounts for the P-Delta effect at a small elastic drift as required in current provisions, although this is not a robust approach as discussed above. In both of these cases, gravity-only framing is on Column Lines 3 and 4. The third case, referred to as Baseline SMF plus SSMF, employs an enhanced SSMF gravity frame that is proportioned as described above and replaces the gravity-only framing on Columns Lines 3 and 4 as illustrated in Figure 3. For the SMF, A992 wide flange sections with $F_y = 50$ ksi (340 MPa) are used. In the SSMF, A913 Grade 65 wide flange sections with $F_y = 65$ ksi (448 MPa) are used as a strategy to help meet the minimum and maximum stiffness requirements.

Table 1. Section Sizes for Frame Designs.

Story	<i>Baseline SMF</i>			<i>Current Practice SMF</i>			<i>SSMF</i>	
	<i>Beam</i>	<i>Int. Column</i>	<i>Ext. Column</i>	<i>Beam</i>	<i>Int. Column</i>	<i>Ext. Column</i>	<i>Beam</i>	<i>Column</i>
12	W24X55	W14X132	W14X132	W24X55	W14X159	W14X132	W16X26	W14X26
11	W24X84	W14X132	W14X132	W27X94	W14X159	W14X132	W18X35	W14X26
10	W27X94	W14X233	W14X159	W27X94	W14X283	W14X193	W21X44	W14X53
9	W30X124	W14X233	W14X159	W33X141	W14X283	W14X193	W21X50	W14X53
8	W30X124	W14X283	W14X211	W33X141	W14X311	W14X211	W24X55	W14X74
7	W33X141	W14X283	W14X211	W33X152	W14X311	W14X211	W24X62	W14X74
6	W33X141	W14X311	W14X233	W33X152	W14X342	W14X257	W24X68	W14X99
5	W33X141	W14X311	W14X233	W33X169	W14X342	W14X257	W24X76	W14X99
4	W33X141	W14X311	W14X233	W33X169	W14X370	W14X283	W24X84	W14X132
3	W33X141	W14X311	W14X233	W33X169	W14X370	W14X283	W24X94	W14X132
2	W33X141	W14X342	W14X283	W33X169	W14X398	W14X342	W24X103	W14X176
1	W33X141	W14X342	W14X283	W33X169	W14X398	W14X342	W27X129	W14X176

Steel weight comparisons for a single frame (a three-bay moment frame, one half of the SFRS in one direction) provide a reasonable estimate of relative cost for the three design cases considered here. The Baseline SMF weighs 297 kips (1320 kN) and the Current Practice SMF weighs 340 kips (1510 kN).

Thus, current design provisions lead to approximately a 15 % SMF steel weight increase due to the P-Delta effect. The SSMF is an enhanced version of a three-bay line of gravity framing (e.g. Column Line 3 from Column Lines B to E), as shown in Figure 3. As a gravity-only frame (A992 steel), the steel weight for this line of framing is 116 kips (516 kN), and when it is enhanced to serve as the SSMF (A913 Grade 65 steel), the steel weight is 140 kips (623 kN). Assuming a 10 % premium for the higher strength steel in the SSMF, the equivalent A992 SSMF frame weight for cost comparison is 154 kips (685 kN). Thus, to obtain a comparable steel weight that is associated with SFRS cost, the increase in weight from gravity-only to SSMF, 38 kips (169 kN), is added to the Baseline SMF to obtain 335 kips (1490 kN). Thus, the Baseline SMF plus SSMF is slightly more economical than the Current Practice SMF, and as will be shown below, it has significantly better performance.

Nonlinear Assessment

Static Response. To evaluate inelastic behavior of the three design cases, displacement-controlled nonlinear static analyses were conducted (Figure 4). The Baseline case was assessed with no gravity load on the leaning column. Thus, it serves as a reference case where the P-Delta effect is not included in proportioning or assessment, and it can be viewed as a target for a design method to achieve perfect correction for the P-Delta effect. Figure 4 indicates Current Practice case does not adequately compensate for the P-Delta effect, as it does not reach the same strength levels and drift ratios as the Baseline case. The Baseline case reaches maximum base shear around 2.7 % roof drift ratio, whereas Current Practice reaches maximum base shear around 1.4 % roof drift ratio. The maximum base shear in Current Practice compared to Baseline is around 90 %. In contrast, the SSMF case, which is the Baseline SMF design plus the SSMF, provides significant secondary stiffness and counteracts the P-Delta effect, reaching maximum base shear around 3.2 % roof drift ratio. The peak base shear for SSMF is around 130 % of Baseline. Figure 4 reveals that the Baseline SMF exhibits an extended region of positive secondary stiffness extending across a range of approximately 2 % roof drift ratio. The Current Practice SMF is dominated by the P-Delta effect and exhibits a limited region of positive secondary stiffness extending across a range of approximately 0.5 % roof drift ratio. As anticipated, the case where secondary stiffness was explicitly designed exhibits the most prominent region of positive secondary stiffness extending across a range of more than 2 % roof drift ratio.

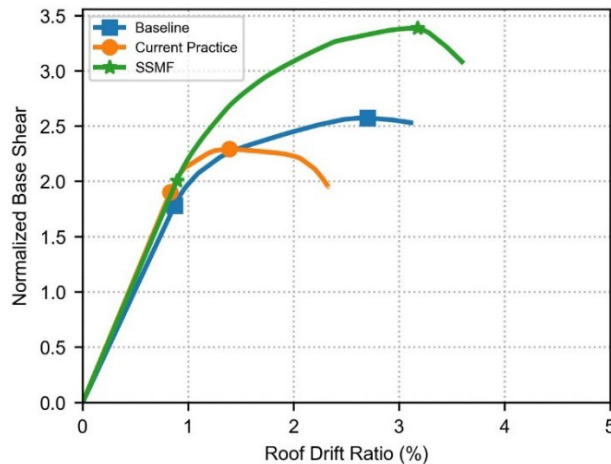


Figure 4. Normalized monotonic static pushover curves.

Dynamic Response. For nonlinear response history analysis, the Federal Emergency Management Agency (FEMA) P695 (FEMA 2009) far-field record set, which comprises 22 seismic records (44 individual components) was normalized according to FEMA P695 guidelines and scaled to the risk-targeted Maximum Considered Earthquake (MCE_R) response spectrum at the fundamental period of the SFRS. As a case study, the system responses to the acceleration record Imperial Valley, El Centro Array #11 (IMPVALL/H-E11140) are illustrated in Figure 5. Although all three cases have similar maximum roof drift, Current Practice, which is characterized by the smallest secondary stiffness region, develops significant residual roof drift. The reason for this is further illustrated in Figure 6, which focuses on first story drift. SSMF has a similar response to Baseline, but Current Practice develops a very large first story drift that exceeds the 7.5 % threshold and is considered to indicate collapse in this study. Near the end of the record the response is trending toward complete sideways collapse.

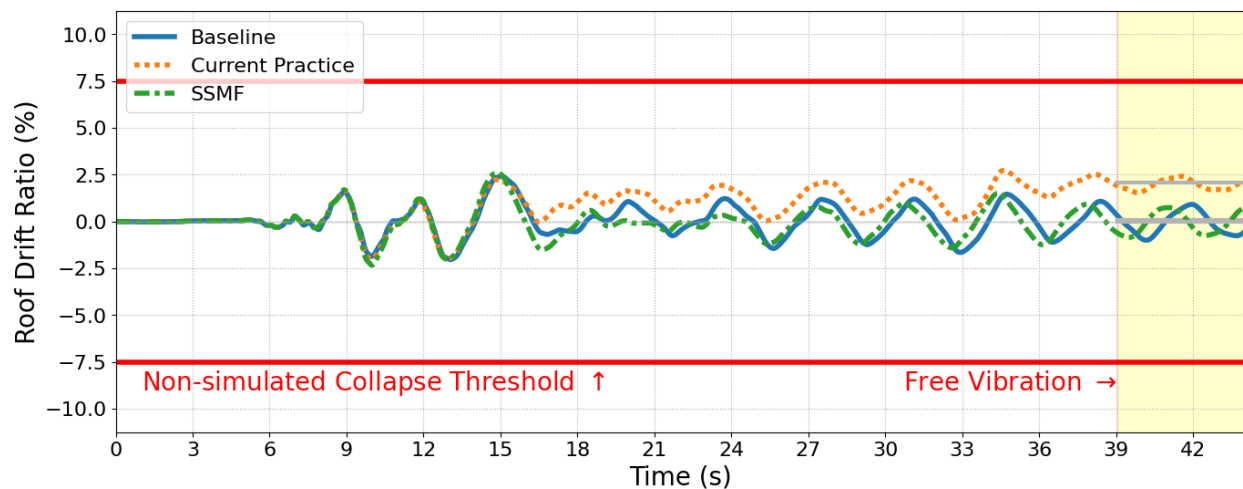


Figure 5. Roof drift ratio response history for SMFs subjected to Imperial Valley, El Centro Array #11 (IMPVALL/H-E11140) scaled to MCE_R .

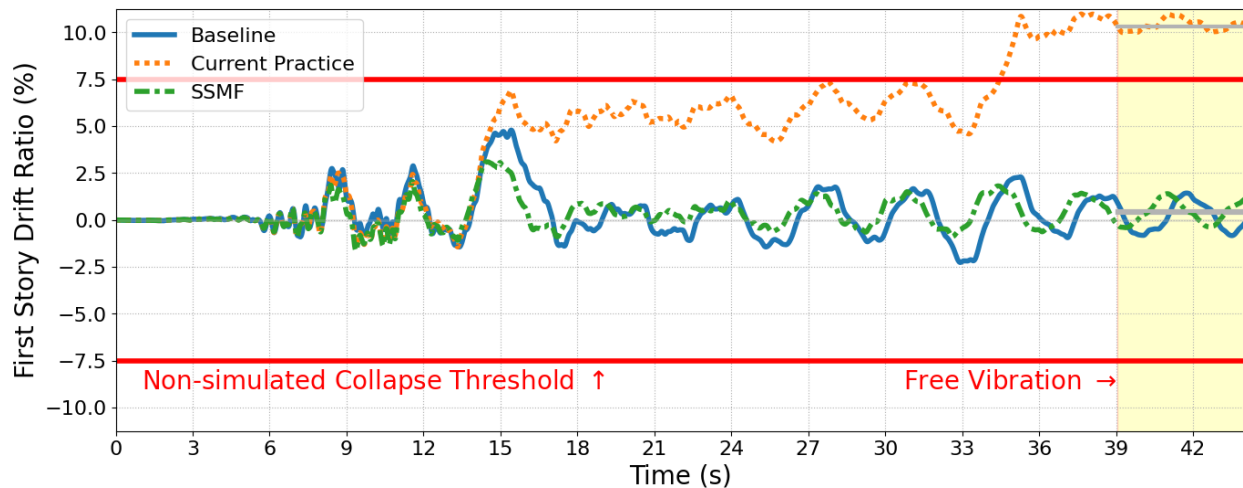


Figure 6. First story drift ratio response history for SMFs subjected to Imperial Valley, El Centro Array #11 (IMPVALL/H-E11140) scaled to MCE_R .

Summary

This paper presents an approach for designing elastic secondary stiffness moment frames that counteract the P-Delta effect and provide seismic stability. Case study designs illustrate the performance of current practice special moment frames (SMF) and SMFs enhanced with elastic secondary stiffness moment frames (SSMF). Evaluated statically, the Current Practice SMF has minimal intrinsic secondary stiffness whereas the combined SMF (proportioned ignoring the P-Delta effect) and SSMF has an extensive region of secondary stiffness. The Current Practice SMF does not effectively counteract the P-Delta effect, whereas the combined SMF and SSMF does. Dynamic response indicates that the combined SMF and SSMF can reduce maximum and residual drift compared to the Current Practice SMF, and that it performs better than the hypothetical case without the P-Delta effect. This initial study illustrates the promise of using elastic secondary-stiffness moment frames as an economical and simple approach for providing seismic stability and enhancing seismic resilience. Future research is needed to comprehensively quantify the influence of secondary-stiffness moment frames on seismic performance of a broader range of primary SFRS and to develop an implementation strategy for design provisions.

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MITIGATION OF SEISMIC COLLAPSE RISK OF RC FRAME BUILDINGS BY REDUCING DESIGN DRIFT

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Abstract

This paper numerically investigates the effect of the maximum inter-storey drift permitted in designing ductile RC frame buildings on their probability of collapse during earthquakes. For this purpose, seismic collapse probability of a range of RC moment resisting frame buildings is assessed at the design basis earthquake (DBE) and the maximum considered earthquake (MCE) levels by conducting more than a thousand nonlinear response history analyses (NRHA) using a suite of 40 seismic ground motion records. The NRHA results consistently demonstrate that the design inter-storey drift limit and anti-buckling detailing of transverse reinforcement in the potential plastic regions of critical frame members strongly affect the collapse probability of RC frame buildings. Probability of building collapse at MCE is found to around 40%, despite following ductile detailing practice, if buildings are designed to 2.5% inter-storey drift limit, as permitted by the incumbent NZ standards. Moreover, collapse probability of RC frame buildings is found to increase exponentially with an increase in design plastic drift limit (θ_p) and the extent of ductile detailing quantified by the buckling length to the bar-diameter ratio (denoted as L_{sr}), which is an indicator of the level of ductile detailing. Based on NRHA results of 13 case study buildings, an easy-to-use equation relating their seismic collapse probability at MCE with θ_p and L_{sr} is established. The proposed equation provides a rapid, yet reasonably reliable method to estimate seismic collapse probability of RC frame buildings without having to conduct cumbersome NRHA and can be used as a performance-based seismic design tool for RC frame buildings to restrict probability of collapse at MCE within an acceptable limit.

Introduction

Building collapse is defined as the state at which one or several gravity load-carrying components suffer severe damage and deterioration, to an extent which renders the building irrecoverable. In addition, building collapse may also be triggered by structural instability due to P- Δ actions exceeding the residual resistance of the deteriorated columns; thereby prompting sidesway mode of failure. Note that these definitions of structural collapse do not refer to complete building collapse. Rather, collapse is interpreted and defined as the onset of the damage/failure of structural components (or system) to the extent that the building is no longer repairable and safe to use following an earthquake. Hence, it is important to note that buildings do not need to fall into debris to be identified as “collapsed”, and severely damaged buildings that are subsequently demolished after an earthquake are also categorized as “collapsed”.

Collapse probability is a critical parameter in evaluating seismic performance of buildings as it governs life-safety of a building and contributes significantly to the building’s expected seismic loss/risk. Hence, understanding the causes of seismic collapse and their mitigation measures is the key to enhance seismic performance of buildings. Resistance of a building against collapse can be enhanced by a combination of; (i) not letting the building to be exposed to excessive deformation (by designing it to be stiffer); and (ii) detailing the key components of the building to avoid failure when exposed to the expected level of deformation (by avoiding shear failure and providing ductile detailing). To qualify this hypothesis, this paper presents an extensive numerical investigation conducted to understand if/how collapse probability of RC frame buildings is affected by the drift limit applied in designing the buildings and the robustness of the transverse reinforcement detailing in the frame members, which is characterized by the ratio of the

potential bar buckling length L_b (which is a multiple of the stirrup spacing) and the bar diameter d_b , i.e. $L_{sr}=L_b/d_b$, which is commonly accepted/used to quantify the buckling-resistance of RC members.

While providing ductile detailing renders a building able to remain upright despite severe damage (caused by large inelastic deformation), it is intuitive that reducing building drift by designing stiffer buildings leads to less structural and non-structural damage and subsequently less loss from repair costs and downtime (Dhakal 2024). In addition, drift limit has also been identified in previous research (Koopae and Dhakal 2024) as a key parameter that predominantly influences seismic collapse probability of RC frame buildings. This research confirms that stiffer and well-detailed buildings are significantly less likely to collapse in a major earthquake and establishes an interrelationship between seismic collapse probability and design plastic drift limit (θ_p) for different levels of ductile detailing (i.e. L_{sr}).

Overview of the Numerical Investigation

A set of 13 RC frame buildings representing low-medium height building stock in seismic regions were designed as per the current design approach (characterised by capacity design and ductile detailing) practiced in New Zealand. Nonlinear response history analyses (NRHA) were conducted on all these buildings in OpenSees using fiber-element-based structural model, which enables simulation of both vertical and sidesway modes of collapse. Previous studies investigating collapse (Koopae et al., 2015) have shown that RC frames may lose vertical load carrying capacity due to (i) crushing of confined core concrete, which will eventually cause buckling of reinforcing steel bars; (ii) buckling of reinforcing bars due to cracked concrete not engaged in compression during cyclic loading (i.e. cyclic strength and stiffness degradation); and (iii) rupture of reinforcing steel bars due to low cycle fatigue. It is hence imperative that collapse analyses are conducted using models that are able to capture these failure mechanisms. For that purpose, Popovics (1973) model was used for concrete fibers and the response of steel fibers were simulated using the *reinforcing steel* model which duly considers the effect of fatigue and buckling of reinforcing bars based on the Dhakal and Maekawa bar-buckling model (2002a, 2002b).

Buckling induced strength degradation of compression bars, which controls the inelastic response (and flexural collapse) of ductile RC members, is solely characterised by the potential buckling length to the bar diameter ratio (Dhakal and Maekawa, 2002c); denoted as L_{sr} in the steel constitutive model used herein. Note that buckling length of longitudinal bars can vary from one to several times tie spacing depending on the geometrical and mechanical properties of lateral ties and longitudinal bars (Dhakal and Maekawa 2002c). The 13 case study buildings were analysed by assigning $L_{sr}=5.0, 7.5$ and 10 for the reinforcing bars in beams and columns to represent different configurations of transverse reinforcement resulting in varying levels of anti-buckling resistance. These values of L_{sr} were chosen to represent the range of transverse reinforcement spacing for fully ductile ($s \leq 6d_b$), limited-ductile ($s \leq 8d_b$) and nominally-ductile ($s \leq 10d_b$) detailing allowed by the NZ concrete standard (NZS3101, 2006).

For collapse prediction, NRHA were conducted using a suite of 20 pairs of orthogonal ground motions scaled to DBE (10% probability of exceedance in 50 years) and MCE (2% probability of exceedance in 50 years) levels of seismic intensity. The details of the case study building design and the ground motion selection and scaling methods are explained in detail in Koopae and Dhakal (2024). At each of these two hazard levels (DBE and MCE), probability of collapse was calculated as the ratio of the number of GMs resulting in structural collapse to the total number of GMs (i.e. 40).

Results of the Parametric Numerical Analysis

Effect of Design Drifts. Collapse analysis results for the case study buildings with varying L_{sr} and design inter-storey drifts are shown in Table 1 and Table 2 for the DBE and MCE hazard levels, respectively. In these tables, each row presents analysis results for a new case study building which is

created by varying section sizes of structural components to achieve an intended design drift. An empty cell in the tables indicates that collapse analysis was not carried out to save computation time, since that case is unlikely to provide any additional useful information on top of those provided by the other cases.

In Tables 1 and 2, design inter-storey drifts (θ_{des}) comprise of two components: elastic drift (θ_e) and plastic drift (θ_p). As collapse of buildings takes place due to excessive plastic deformation in the plastic hinge regions of the structural components, it is prudent to relate the maximum plastic drift (rather than the total drift) to the collapse probability. Plastic drift for case study buildings can be approximately estimated as;

$$\theta_p = \theta_e(\mu \times k_{dm} - 1) \quad (1)$$

where θ_e is the maximum linear inter-storey drift calculated from an equivalent static analysis (ESA), μ is the ductility and k_{dm} is a coefficient to account for building height as per NZS1170.5 (2004).

Table 1: Collapse probability of case study buildings at DBE (500-year return period)

<i>Building ID</i>	<i>Max. θ_{des}</i>	<i>Max. θ_p</i>	<i>$P(C DBE)_{Lsr=5}$</i>	<i>$P(C DBE)_{Lsr=7.5}$</i>	<i>$P(C DBE)_{Lsr=10}$</i>
10-Storey, 4-bay Case Study Buildings					
Base Model	0.028	0.0233	0.125	0.375	0.80
2	0.024	0.0200	0	0.075	0.4
3	0.021	0.0175	0	0.025	-
4	0.016	0.0133	0	0	-
5	0.0146	0.0122	0	0	-
6	0.011	0.0092	0	0	0.225
10-Storey, 6-bay Case Study Buildings					
7	0.026	0.0217	0.1	0.3	-
8	0.023	0.0192	0	0.075	-

Note that the base (i.e., the first) model and models 7 and 9 are shown to have the design inter-storey drifts (estimated by equivalent static analysis, ESA) greater than the 2.5% limit prescribed in the NZ standards. It is important to note here that the allowable drift limit in NZ standards is the highest among the contemporary codes, which also indicates that there is greater reliance on good detailing and ductility to prevent structural failure/collapse. Ductile buildings are expected to respond deep into the inelastic response phase at DBE (i.e., close to the target drift, which can be as high as 2.5%) and the stiffness of the system would have substantially degraded at that stage. Hence, when the shaking intensity is further increased to MCE (i.e., $1.8 \times DBE$), the maximum drift demand is bound to increase further by significantly more than 1.8 times, which can often lead to collapse due to excessive plastic rotations in the plastic hinges (despite ductile detailing).

From Table 1, it is evident that for most of the case study buildings, collapse at DBE is avoided (i.e. zero collapse probability) if the members are provided with ductile detailing; i.e. $L_{sr} \leq 6.0$, except for when the design drift is larger than 2.5%. However, if not detailed well enough to ensure full ductility (i.e. $L_{sr} > 6$), an RC frame building is not guaranteed to avoid collapse at DBE if the building is designed to reach a

target drift of more than 2%. It is hence important that RC buildings adopt ductile detailing to avoid collapse at design level shakings.

Table 2: Collapse probability of case study buildings at MCE (2500-year return period)

<i>Building ID</i>	<i>T (Sec)</i>	<i>Max. θ_{des}</i>	<i>Max. θ_p</i>	<i>$P(C MCE)_{Lsr=5}$</i>	<i>$P(C MCE)_{Lsr=7.5}$</i>	<i>$P(C MCE)_{Lsr=10}$</i>
10-Storey, 4-bay Case Study Buildings						
Base Model	1.76	0.028	0.0233	0.675	0.90	0.975
2	1.62	0.024	0.0200	0.4	0.625	0.8
3	1.47	0.021	0.0175	0.35	0.525	0.8
4	1.16	0.016	0.0133	0.175	0.425	0.675
5	1.12	0.0146	0.0122	0.15	0.375	0.65
6	0.83	0.011	0.0092	0.1	0.275	0.625
10-Storey, 6-bay Case Study Buildings						
7	1.75	0.026	0.0217	0.55	0.95	1.0
8	1.6	0.023	0.0192	0.325	0.65	0.85
12-Storey, 4-bay Case Study Buildings						
9	1.84	0.0265	0.0221	0.525	-	-
10	1.35	0.0162	0.0135	0.30	0.60	-
11	1.22	0.0141	0.0118	0.25	-	-
5-Storey, 3-bay Case Study Buildings						
12	0.85	0.016	0.0127	0.15	0.275	0.55
13	0.87	0.0178	0.0142	0.20	0.325	0.625

Here, greater emphasis is placed on the collapse probability at MCE hazard level, as it provides a wide range of data to establish interrelationship between the design drift and collapse probability. As can be seen in the first two rows of Table 2, collapse probability of buildings designed in accordance with the NZ standards (NZS1170.5, 2004) can become high at MCE despite adhering to capacity design principles and applying ductile detailing ($L_{sr} \leq 6.0$); if the design drift is close to the maximum allowable (i.e. 2.5%) inter-storey drift limit allowed in the standard.

For example, the collapse probability of the code-confirming building model 2 (i.e. designed to limit the maximum inter-storey drift of 2.4% with ductile detailing) is found to be 40% at MCE. Although the predicted collapse probability of 40% at MCE appears quite high at first glance, it is not inconsistent with the observation from 2011 Christchurch earthquake that had ground shakings comparable to 2500 return period level hazard for Christchurch and forced more than 60% of multi-storey concrete buildings to be demolished (Kim et al. 2017). Note that the same building had 0% probability of collapse at DBE level

of intensity (see the second row in Table 1), which is again consistent with the observation from 2010 Darfield earthquake that produced ground shakings of relatively lesser intensity and did not cause collapse of any modern concrete buildings (Kam et al. 2010). This indicates that the design approach of implicitly assuming that a building that meets the collapse prevention and life-safety requirements at DBE will also satisfy the collapse prevention at MCE cannot be justified.

Case study buildings 3 to 6 have been designed by gradually reducing the design inter-storey drifts. As expected, analysis results for these buildings demonstrate that designing stiffer buildings (which drift less) result in considerably lower collapse probability compared to flexible buildings. Findings from these building models suggest that a consistent reduction of collapse probability can be achieved by reducing the maximum design inter-storey drifts. This is consistent with the findings of Sarrafzadeh et al. (2017), who reported significantly smaller proportion of severely damaged concrete buildings (leading to demolition) in the 2016 Kumamoto earthquake compared to that in 2011 Christchurch earthquake despite comparable shaking intensity and design seismicity in the two cities. The stark difference between the concrete buildings' performances in the two earthquakes/cities was claimed to be mainly due to the requirement to design stiffer buildings with drift limit of 1.2% in Japan compared to much more flexible buildings drifting up to 2.5% in New Zealand.

To further investigate this outcome, building models 7 and 8 were designed in accordance with NZS1170.5 but with a different layout compared to the base building (6-bays vs 4-bays). Collapse analysis of these building further confirms that buildings designed to respond close to the maximum allowable drift limits of NZS1170.5 are likely to have high collapse probability at MCE. Comparing the collapse probabilities of building models 7 and 8 shows the positive effect of reducing the maximum design drift. To investigate the collapse probability of buildings with different heights, NRHA are also carried out for 12-storey and 5-storey buildings designed for varying drift limits. Results of the 5-storey (i.e. models 12 and 13) and 12-storey (i.e. models 9-11) building variants further confirm that by reducing the maximum design inter-storey drifts, collapse probability will reduce.

Effect of Anti-buckling Detailing. Collapse analysis results shown in Table 1 and Table 2 also manifest the effect of transverse reinforcement detailing in the potential plastic hinge regions of the frame members on the collapse probability of the building. It is evident from these tables that typical buildings detailed for limited or nominal ductility (i.e., $L_{sr} \geq 7.5$) have noticeably higher collapse probabilities compared to ductile buildings designed with $L_{sr} \leq 6.0$ particularly at MCE hazard level.

Results shown in Table 2 also demonstrate that reduction of design inter-storey drifts does not always compensate for the lack of good anti-buckling detailing of structural components. This is specifically evident from results of building 6 which is designed for a very low design inter-storey drift. For this building, collapse probability at MCE (10% for $L_{sr}=5$) almost triples (to 27.5%) when the frame members are detailed for limited ductility (i.e., $L_{sr}=7.5$). Similarly, when ductile detailing is not followed (i.e. $L_{sr}=10$) collapse probability at MCE increases to 62.5%, which is unacceptable by all means. Therefore, it is fair to suggest that to achieve acceptably lower collapse probability at MCE, ensuring appropriate anti-buckling detailing of structural components is critically important.

Synthesizing the outcome of all (more than a thousand) collapse analyses undertaken to obtain the results shown in Tables 1 and 2, a generic expression shown below in Equation (2) has been proposed to estimate the collapse probability at MCE of RC frame buildings designed to a plastic drift limit and detailed for different levels of ductility (resulting in different L_{sr}).

$$P(C|MCE) = 0.07[e^{(100 \times \theta_p)} - 1] + 1.1 \times [1 - \langle 1 - 100\theta_p \rangle] \times \langle 1 - 6/L_{sr} \rangle \quad (2)$$

In Equation (2), θ_p is the maximum plastic inter-storey drift permitted at the DBE which shall be computed based on Equation (1), and the $\langle \rangle$ brackets represent step (i.e. singularity) functions, and the

term becomes zero when the value of the parameter inside these brackets is negative. Needless to mention, the prediction of Equation (2) is bound by a minimum limit of zero and maximum limit of one, as collapse probability cannot be outside this range.

Equation (2) implies that, when the plastic drifts are converted to design inter-storey drifts, buildings designed with maximum inter-storey drifts higher than about 0.033 (i.e. $\theta_p \sim 0.0275$) will result in 100% collapse probability at MCE. This may induce an error if this equation is used for buildings with higher design drifts, but 0.033 is already much higher than the design drift limit prescribed by all codes in the world. Also, this will be inconsequential in design as $P(C|MCE)$ will typically be restricted to much lower values for design purposes. Thereby, precise estimation of collapse probability at high levels of probabilities is not a matter of high interest and importance.

Equation (2) also enables quick estimation of seismic collapse probability of ductile RC frame buildings at MCE without having to conduct a large number of rigorous nonlinear dynamic analysis. Figure 1 plots the collapse probability at MCE level of earthquakes against the design plastic inter-storey drift predicted by Equation (2) for the three values of L_{sr} used in the case study RC frame buildings. Shown in the figure are also the collapse probabilities of all 13 case study buildings considered in the NRHA reported earlier. As can be seen in the figure, accuracy of Equation (2) is reasonably good for all values of θ_p and L_{sr} .

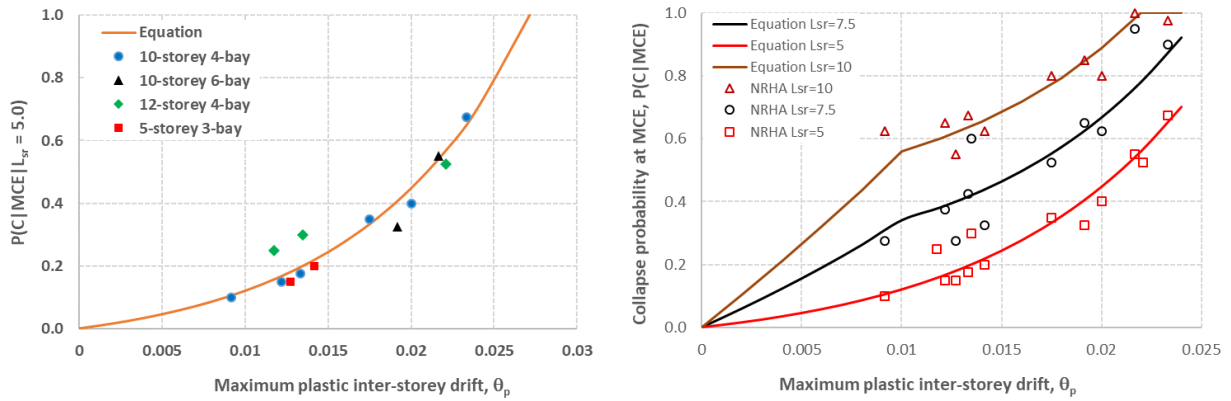


Figure 1. Effect of (a) design plastic drift; and (b) level of ductile detailing (i.e. L_{sr}) on the collapse probability at MCE predicted by NRHA and the proposed equations for the case study buildings.

Design of RC frame buildings based on a target collapse probability

Results of case study buildings reported in the previous sections demonstrated that collapse probability of RC frame buildings is strongly related to L_{sr} and design inter-storey drifts. They also illustrated that, in general, if RC frames are detailed such that L_{sr} is higher than or equal to 7.5, the resulting collapse probability is generally high. Therefore, for seismic design of new buildings, it is reasonable to suggest the plastic hinge regions of the frame members be detailed to ensure $L_{sr} \leq 6$. The limit of 6 is used here because the maximum tie spacing to bar diameter ratio for ductile detailing is specified as 6 in most concrete design codes, and the buckling-induced deterioration of rebar compression stress is practically acceptable if the slenderness ratio does not exceed 6 (Dhakal and Maekawa 2002a).

If the acceptable/target collapse probability at MCE; i.e. $P(C|MCE|L_{sr} \leq 6)_T$, is known, Equation (2) can be re-arranged to calculate the maximum design plastic hinge rotation $\theta_{p,a}$ for ductile-detailed (i.e., $L_{sr} \leq 6$) RC frame buildings as:

$$\theta_{p,a} = \ln \left[\frac{P(C|MCE|L_{sr} \leq 6)_T}{0.07} + 1 \right] / 100 \quad (3)$$

Once $\theta_{p,a}$ is calculated for the given target collapse probability at MCE, the maximum allowable design inter-storey drift (θ_{des}) and consequently the maximum allowable elastic inter-storey drift ($\theta_{e,a}$) at ULS (i.e., DBE), can be estimated using Equation (1).

Using Equation (3), a simplified performance-based seismic design procedure to limit collapse probability of RC frame buildings can be formulated. As an example, if 20% probability of collapse at MCE (defined based on NZS1170.5 for 2500-year return period event, i.e., $1.8 \times \text{DBE}$) is assumed as the design objective, the maximum allowable plastic rotation limit should be; $\theta_{p,a} = \ln \left[\frac{P(C|MCE)_T}{0.07} + 1 \right] / 100 = 0.0135$. Consequently, the maximum allowable elastic inter-storey drift for a mid-rise ductile building ($k_{dm}=1.5$) designed with $\mu=4$ becomes; $\theta_{e,a} = \frac{\theta_{p,a}}{\mu \cdot k_{dm} - 1} = 0.0027 \text{ radians}$. This indicates that the design inter-storey drift will have to be limited to $\sim 1.6\%$ to meet the 20% collapse probability target. Similar calculation suggests that to restrict the collapse probability at MCE to less than 10%, the maximum design drift should be restricted to 1.05%. However, it should be noted that above calculations assume that detailing of structural components comply with the requirements of $L_{sr} \leq 6$, meaning that buckling length of longitudinal reinforcing cannot be greater than 6 times of the diameter of longitudinal bars.

The recommendation of designing stiffer (hence stronger) buildings may make designers and building owners apprehensive about the increase in the cost. Nevertheless, previous research on effect of design ductility on construction cost of low and medium rise RC frame buildings in NZ (Dhakal and Aninthaneni 2021) has reported that increasing building strength by four times (which naturally makes the building significantly stiffer) increases the total cost of the building by only about 7%. Similarly, a study based on construction costs in the United States has claimed that construction costs will increase only by about 1% if the material costs for a lateral load resisting components in a building is doubled to increase the building's strength and stiffness (Porter 2020). As the total project cost increase is modest and it yields significant gain in seismic performance via reduction of the expected damage/loss and collapse probability in a major earthquake, it seems desirable and financially feasible to design concrete frame buildings to be stiffer so that it incurs limited inelastic response (i.e. no more than 1% plastic drift) in a design level earthquake.

Conclusions

In this study, 13 RC moment resisting frame buildings with different heights (5-12 storeys) and footprints (3-6 bays) were designed to reach different maximum inter-storey drifts at ULS, and three variants were generated for each case study building by detailing the frame members as per the full, limited, and nominal ductility provisions in NZ concrete standards (resulting in L_{sr} values 5, 7.5, and 10, respectively). Using a suite of 40 seismic ground motion records, more than a thousand NRHA were conducted on these building models to assess their probability of collapse at DBE and MCE levels of seismic shakings.

The NRHA results consistently demonstrated that the design drift limit and the anti-buckling detailing (represented by the L_{sr} values) of the critical frame members strongly influence the collapse probability of RC frame buildings. The results also showed that while RC frames buildings designed to reach 2.5% (or less) inter-storey drifts and detailed for full ductility ($L_{sr} \leq 6$) can avoid collapse at DBE, collapse at DBE cannot be ruled out if RC frames are designed for a drift limit of more than 2% and not detailed for full ductility (i.e. $L_{sr} > 6$). At MCE, collapse probability is found to consistently exceed 10% unless the building is designed to incur no more than 1% plastic drift and the frame members conform to ductile detailing requirements.

To minimize the likelihood of building collapse in a major earthquake, it is hence recommended that plastic drift limits used in designing concrete buildings is restricted to $<1\%$ and the frame members are reinforced to satisfy the ductile detailing provisions (i.e. resulting in $L_{sr} \leq 6$). Designing and constructing

stiffer (hence stronger) buildings may incur modest cost increase (e.g. less than 7% for a 4-fold strength increase), but the significant gains in seismic performance it provides (in the form of less damage/loss and significantly small probability of collapse in major earthquakes) easily outweigh the modest cost increase.

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DESIGNING TO LOWER DRIFT LIMITS: THE IMPACTS ON CONSTRUCTION COSTS AND NON-STRUCTURAL COMPONENTS

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Abstract

There is clear experimental, numerical, and field evidence indicating that stiffer structures experience less deformation and damage during earthquakes. The evidence suggests that reducing the drift limit specified in design provisions leads to a) less structural damage, b) less damage to non-structural components such as facades, partitions, and finishes, and c) faster recovery. Common arguments against designing stiffer structures or reducing the drift limit refer to 1) increases in construction cost, and 2) damage to “acceleration-sensitive” non-structural components. Nevertheless, there is scarce quantitative evidence to support either argument. This study quantifies the increase in cost associated with reducing the design drift limit for reinforced concrete (RC) structures and examines field information on the post-earthquake functionality of Chilean hospitals with frame and wall structures. A total of 64 idealized, hypothetical RC structures were designed for two estimates of seismic hazard for Wellington, New Zealand. The structures, ranging from 5 to 20 storeys tall, were designed to drift limits ranging from 2.5% to 1.0%. Using current estimates of material and labour costs, the cost of each structure was estimated. The results suggest that reducing the design drift limit from 2.5% to 1.0% increases the total building cost by 1 to 2%. The estimates also suggest that walls are more cost-effective in reducing drift demands. To complement this information, tests are being conducted to study the behaviour of “acceleration-sensitive” non-structural components under floor motions from stiff and flexible structures. The tests will add to field evidence that is being compiled from records of earthquake damage and loss of function occurring in hospitals in Chile. A preliminary examination of this evidence confirms the idea that loss of function is more frequent in hospital buildings with frame –instead of wall- structures. The records also suggest that damage to ceilings and equipment is not more frequent in wall structures.

Introduction

Evidence from earthquake reconnaissance in the last 100 years, dating back to the Great Kanto Earthquake in 1923 (Naito, 1923), has repeatedly illustrated that stiffer structures deform less and sustain less damage (Pujol et al. 2024). Several numerical (Browning, 1998; Koopae, 2022; and Pledger et al. 2024a) and experimental studies (Cecen, 1979, & Algan, 1982) have drawn the same conclusions. It is relevant to note that proportioning structures to increase stiffness (i.e. by making columns and beams larger, or walls longer) does not always lead to a reduction in drift capacity. Evidence from quasi-static tests of RC columns and walls highlight that the effect of reducing aspect ratio -commonly recognized to reduce drift capacity- can be offset by reducing the axial load ratio and/or providing more transverse reinforcement (Pledger et al. 2024b). Of course, if increases in aspect ratio remain a concern under specific circumstances, increases in stiffness can be achieved by increasing the number instead of the sizes of elements or frames resisting lateral demands.

A simple method to achieve stiffer structures that deform less is to reduce the drift limit specified in design provisions. Reducing the design drift limit leads to smaller drift demands, a lower probability of collapse (as long as drift capacity is not reduced as stiffness is increased), and less damage to non-

structural elements such as facades, partitions, and finishes. Another way to produce stiffer structures that deform less is to specify minima for the cross-sectional areas of elements in the system(s) resisting lateral demands. For instance, in Japan, buildings with fewer than 7 storeys are required to satisfy the following expression (BSL, 2016).

$$2.5 \text{ MPa } A_w + 0.7 \text{ MPa } A_c > W \div \sqrt{f'_c \div 18 \text{ MPa}} \quad (1)$$

Where, A_w is the summation of cross-sectional wall areas in a principal floor plan, A_c is the summation of cross-sectional column areas, and W is the total building weight. Equation 1 applies to the first storey of the structure. Normalizing terms in Equation 1 and assuming $f'_c = 30 \text{ MPa}$, the expression above can be rewritten as:

$$WI + 0.55 \text{ CI} > 0.3\% \quad (2)$$

where WI is the area of walls in one principal floor plan divided by the total floor area of the structure, and CI is one half of the sum of column areas divided by the total floor area. That expression is similar to the assessment limit proposed by Shah (2017) based on work by Hassan (1997) and field evidence:

$$WI + 0.5 \text{ CI} > 0.25\% \quad (3)$$

That two very different approaches and traditions have arrived at nearly the same conclusion is remarkable. The idea should be considered, if nothing else, as a preliminary check on the proportions of new buildings in highly seismic areas. To introduce the idea, the check could be used to require peer review for structures not meeting it. Nevertheless, it must be admitted that the attention of the engineering profession has been directed towards detailed analyses and away from pragmatic proportioning “rules.” Reversing that trend may be difficult and, for that reason, it may be easier to achieve drift control by simply reducing the design drift limit.

Two arguments against reducing the design drift limit are still common. The first of these is cost, and the perception that reducing the design drift limit leads to an increase in the cost of the structure so large that construction becomes uneconomical. There is little quantitative evidence to support this, and results from previous studies (Garcia et al. 1996, & Pledger et al. 2024) suggest that the increase in building cost related to reducing the drift limit is far from being prohibitive. A drift limit of 1.5% was adopted in Colombia in the 1998 version of the code (NSR-98) after Garcia et al. (1996) demonstrated the cost-effectiveness of using RC walls, and their good performance in the 1985 Chile Earthquake (Riddell et al., 1987). The second argument against reducing the design drift limit of a structure is that a tighter limit will lead to increased acceleration demands, causing more damage to contents and non-structural components. Evidence not supporting that claim is presented in a later section. Ongoing experimental tests examine this idea further by comparing the behaviour of non-structural components under floor motions from stiff and flexible structures.

The following section explores the effect of reducing the design drift limit on building cost. A total of 64 hypothetical RC structures were designed to satisfy strength, stiffness, and ductility requirements specified in the New Zealand seismic design guidelines. The buildings ranged from 5 to 20 storeys in height, and the design drift limit ranged from 2.5% (NZ code maximum) to 1.0%. The structures were designed to two different intensity levels, one being nearly twice the other (consistent with seismic hazard

estimates available before and after a review of the NZ hazard maps that was published in 2024). Half of the structures were RC frames and half were RC wall structures.

Design of archetypal RC structures

Sixty-four structures were designed for a site in Wellington, New Zealand. Relative increases in approximate building cost associated with changes to a) design drift limit, b) lateral load resisting system, c) number of storeys, and d) the seismic hazard, were estimated. Structures were designed to drift limits of 2.5%, 2.0%, 1.5%, and 1.0%. Structures with 5, 10, 15, and 20 storeys were assessed. The different structural systems considered in this study were RC moment resisting frames and RC walls. This study also explores how changes to the seismic hazard influence building cost by designing structures to design intensities recommended in 2022 (Gerstenberger et al. 2024) and design intensities recommended in 2010 (Stirling et al. 2012). Table 1 provides a summary of the design demands for the 2022 and 2010 seismic hazard. Figure 1 shows a comparison between spectral displacement and acceleration for the 2010 seismic hazard and the 2022 seismic hazard. For the 2010 seismic hazard, the displacement spectrum was altered to remove the plateau at $T = 3$ sec and it was assumed instead that the displacement spectrum scales linearly with period. Soil Class C was used for structures designed to the 2010 hazard, and soil class III was used for the 2022 seismic hazard corresponding to a value of $V_{s(30)}$ of 300 m/s.

Table 1: Design parameters for the 2010 and 2022 seismic hazards. Location - Wellington, NZ

2010 Seismic Hazard	
Hazard factor for Wellington, Z	0.4
PGA	0.53 g
Assumed return period	500yr
Distance to fault	>10 km
Soil Class, C, (Shallow soil)	C
2022 Seismic Hazard	
PGA	0.91 g
$S_{a,s}$ (Short-period spectral acceleration)	1.84 g
Distance to fault	>10 km
Site Class III	
T_c (Spectral-acceleration-plateau corner period)	0.5 s
T_d (Spectral-velocity-plateau corner period)	3.0 s

Figure 1 shows that the increase in spectral demands related to the mentioned changes in prescribed design seismic demands for Wellington lies between 50 and 75% for structures with periods shorter than 5.0 seconds. Here, structures were designed to the same design drift limits regardless of the assumed design intensity to allow estimation of the relative increase in cost associated with changes in the assumed hazard.

Typical floor heights were 3.6 m with a first-floor height of 4.0 m. Each frame had 6.0 m long bays in each plan direction. The assumed concrete strength (30 MPa) and yield strength of reinforcement were not varied (500 MPa) from structure to structure. The modulus of elasticity of concrete was assumed to be 25 GPa. Soil-structure interaction was ignored, and columns and walls were assumed to be fixed at the base. Unit weight (8 kPa) was kept constants for all structures. A ductility factor (μ) of 4 and a structural performance factor (S_p) of 0.7 was applied to all structures. These two factors, in NZ practice, translate into a total “force-reduction factor” of $4/0.7=5.7$. The modal response spectrum method outlined in NZS1170.5 (2004) was used for design. Displacement demands were estimated based on the mode-shape

and spectral displacement associated with the first three modes of the structure. Second-order analysis was performed to account for P-delta effects. Frame structures were designed to have a strong-column weak-beam mechanism such that $\Sigma M_c / \Sigma M_b$ was always greater than 1.5. Columns and beams had a constant longitudinal reinforcement ratio of 2.0%. For walls, the longitudinal reinforcement ratio was 3.0% in the boundary elements, and 1.0% in the web. A transverse reinforcement area ratio of 1.0% was used for columns, beams, and walls. All the buildings were designed to be ductile and not susceptible to failures related to shear or bond.

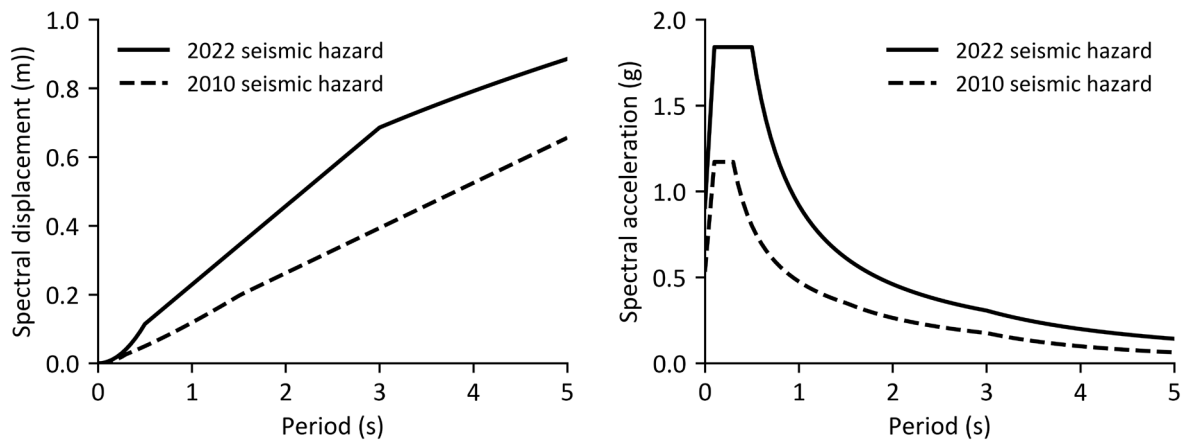


Figure 1: Spectral displacement and spectral acceleration for a site in Wellington based on the 2022 and 2010 seismic hazard.

Table 2 provides a summary of the properties of the proportioned buildings, including number of storeys, design drift limit, the fundamental period calculated for cracked sections, and whether the building was designed according to the 2010 or 2022 seismic hazard. The column and wall indices values are also listed in Table 2. Two trends are clear from Table 2, 1) A 40-50% reduction in fundamental period is observed as the structure is designed to the 2022 hazard as opposed to the 2010 hazard, and 2) fundamental period reduces with the design drift limit in an approximately linear fashion. The results from Table 2 also highlight that wall and frame structures designed to the same drift limit exhibited similar fundamental periods. When compared with the provisions outlined in the Japanese building code for RC structures with fewer than 7 storeys (Equation 1), structures designed to a 1.0% drift limit for the 2022 seismic hazard have values close to the minimum of $WI + 0.55 CI > 0.3\%$. This indicates that structures designed in Japan may be comparable to buildings designed to not exceed 1.0% drift in New Zealand today.

Cost estimation

Relative structural costs of the 64 RC buildings was estimated using the software package QV cost builder (<https://costbuilder.qv.co.nz/>). The used software is a digital New-Zealand-specific quantity surveying catalogue that provides current estimates of material, and labour costs for different structures. The structures were designed as office buildings. Table 3 provides a summary of costs for medium-rise office buildings in Wellington. The structure -consisting of foundations (substructure), the structural system (frame and or walls), and the flooring (i.e. cast in-situ slabs)- accounts for approximately 20% of the overall building cost. It was assumed that the flooring systems and foundations remained unchanged for all the buildings. Any increase in cost was related directly to changes in the size of structural elements. Reductions in building cost related to the reduced quantity of partitions or fire-rated walls in RC wall structures were ignored, offsetting potential ignored increases in foundation cost. The average values provided in Table 3 relate to office buildings between 6 and 15 storeys tall. Nevertheless, for the purpose of this study, these cost estimates were also used for the 5- and 20-storey structures as well.

Table 2: Summary of building fundamental periods for RC structures designed to different drift limits for the 2022 and 2010 seismic hazard models.

No. of storeys	Drift limit (%)	Frame structures				Wall structure period (s)					
		2010 SHM		2022 SHM		2010 SHM			2022 SHM		
		T (s)	CI (%)	T (s)	CI (%)	T (s)	CI (%)	WI (%)	T (s)	CI (%)	WI (%)
5	2.5	2.3	0.09	1.3	0.17	2.2	0.03	0.03	1.3	0.04	0.06
	2.0	1.8	0.11	1.0	0.20	1.8	0.03	0.05	1.0	0.04	0.07
	1.5	1.4	0.17	0.8	0.28	1.3	0.03	0.07	0.7	0.06	0.11
	1.0	1.1	0.25	0.5	0.35	1.0	0.06	0.08	0.5	0.08	0.25
10	2.5	3.1	0.06	1.9	0.09	3.6	0.01	0.02	2.1	0.04	0.07
	2.0	2.7	0.07	1.5	0.11	2.9	0.03	0.04	1.6	0.04	0.09
	1.5	2.1	0.11	1.1	0.14	2.1	0.03	0.06	1.2	0.07	0.15
	1.0	1.4	0.14	0.7	0.25	1.4	0.03	0.07	0.8	0.07	0.17
15	2.5	4.4	0.05	2.8	0.06	4.8	0.03	0.02	2.9	0.04	0.07
	2.0	3.5	0.06	2.1	0.09	4.2	0.03	0.03	2.4	0.03	0.07
	1.5	2.7	0.07	1.6	0.12	3.3	0.03	0.03	1.8	0.03	0.10
	1.0	2.0	0.09	1.1	0.17	2.0	0.02	0.10	1.3	0.04	0.14
20	2.5	5.8	0.04	3.9	0.06	6.3	0.03	0.02	3.9	0.02	0.08
	2.0	4.8	0.04	2.8	0.07	5.2	0.03	0.03	3.3	0.03	0.08
	1.5	3.5	0.06	2.1	0.08	4.0	0.03	0.03	2.4	0.03	0.08
	1.0	2.5	0.07	1.4	0.13	2.7	0.04	0.05	1.6	0.04	0.26

Table 3: Cost breakdown of base building costs for a site in Wellington for high-rise office buildings.

Element/Element Group	High Rise Offices. 6 to 15 storeys.	
	\$NZD/m ²	%
Substructure (foundations)	204	5.3
Lateral load resisting system (Frame and walls)	320	8.3
Upper Floors (flooring system)	281	7.2
Structure	805	20.9
External Fabric	492	12.8
Internal Finishing	666	17.3
Services	1,339	34.8
External Works & Sundries	29	0.7
Prelims & Contingency	519	13.5
Total	3,850	100

Table 4 provides a summary of the costs for formwork, reinforcement, and concrete, used to estimate the cost of the structural system. The unit costs provided in Table 4 account for 1) material costs, fixings and consumables such as screws, fasteners, and glue, 2) labor costs, 3) delivery and disposal of materials, and 5) any overheads or profits. These costs serve as indices and the writers acknowledge that actual costs may vary depending on many factors that are hard to account for including supply, demand, location, or inflation.

Table 4: Unit costs for construction of cast in-situ RC frame and wall structures accounting for labour and material.

		\$ for a site in Wellington
Description	Unit	(Q4 – 2024)
Formwork to sides of:		
Beams or lintels	m ²	241
Rectangular columns	m ²	208
Walls	m ²	196
Reinforcement		
25mm diameter	tonne	4,040
32mm diameter	tonne	3,980
10mm diameter (stirrups)	tonne	6,600
Standard Ready-Mixed Concrete delivered to site		
25 MPa, 19mm aggregate	m ³	328

The unit costs in Table 4 are used to estimate the relative structure cost for each of the 64 buildings. Figure 2 depicts the structure cost per square meter of floor area for each combination of structural system, design drift limit, and seismic hazard. The values in Figure 2 correspond to the average costs for the 5, 10, 15 and 20 storey structures. The blue bars in the figure represent the structural cost for RC wall buildings. Bars that are hatched refer to structures designed to the 2010 seismic hazard, while unhatched bars are for structures designed to the 2022 seismic hazard. Grey bars illustrate the structural cost for RC frames.

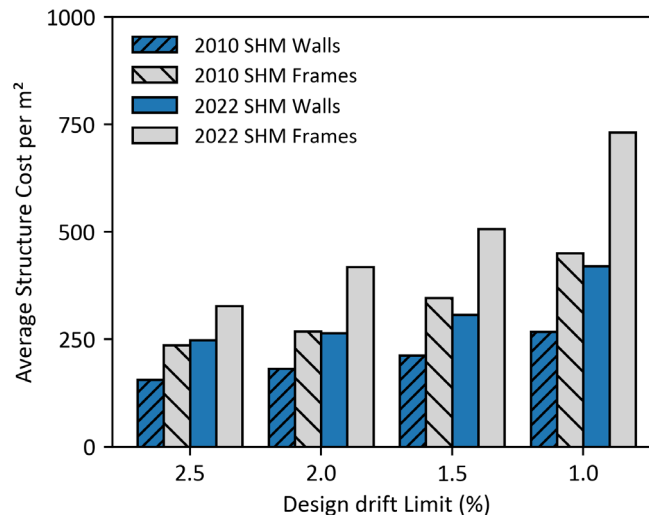


Figure 2: Average structural system cost per square meter for frame and wall buildings designed to different drift limits for the 2022 and 2010 seismic hazard.

Figure 2 depicts a general increase in cost as structures are designed to lower drift limits. In all cases, wall structures cost less than frames structures designed to the same seismic hazard and drift limit. The results from Figure 2 suggest that it is more cost efficient to design the structural system with walls as opposed to a frame. For the cost of a frame designed to a 2.5% drift limit, a wall structure approximately twice as stiff can be designed to meet a drift limit of 1.0% to 1.5%. Keeping all other variables constant, the lateral-load resisting system costs approximately 50% more when designing to the 2022 seismic hazard, with demands nearly twice the demands from the 2010 seismic hazard. However, wall structures

designed for the 2022 seismic hazard were no more expensive than frame structures designed for the 2010 hazard, emphasizing the cost-effectiveness of walls.

As shown in Table 3, the lateral-load-resisting system accounts for approximately 8% of the overall building cost. Figure 3 plots the increases in building cost relative to the cost of a frame structure designed to a 2.5% design drift limit for the 2010 seismic hazard and normalized by the overall building cost of the structure (\$3850 NZD per m²). The results illustrate the percentage change in the building cost relative to a code minimum (2.5% drift limit) frame structure. Shifting from a 2.5% drift limit to a 1.0% drift limit for a RC frame structure designed to the 2010 hazard leads to an increase in building cost close to 5% (grey hatched bars). The results in Figure 3 illustrate that it is cheaper to design a wall structure for the same hazard and drift limit. In fact, it can be cheaper to design a wall structure to the 2022 hazard (blue bars) than it is to design a frame to the 2010 hazard (grey hatched bars).

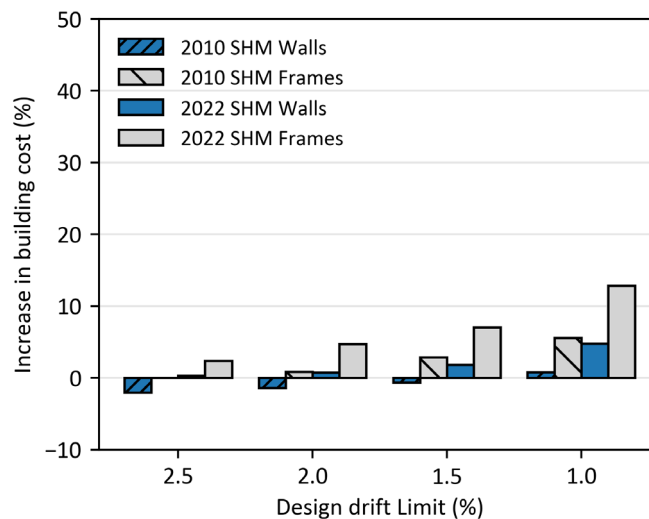


Figure 3: Increase in building cost (%) related to the design drift limit, different structural systems, and changes to the seismic hazard. Results are normalized by the building cost of a RC frame designed to a 2.5% drift limit for the 2010 seismic hazard.

Performance of non-structural components in past earthquakes.

There are numerous reports describing the performance of non-structural components in past earthquakes (Dhakal, 2010; Braga et al., 2011; & Barid and Ferner, 2017). Nevertheless, few studies systematically detail the type of structural system, building age, the period of the structure, or other building information alongside the damage reports of non-structural damage. This makes it difficult to relate non-structural damage to the behavior of the structure.

Reports from the Chilean Department of Risk Management in Emergencies and Disasters detail structural and non-structural damage to 84 Chilean hospitals in the last 50 years. Of these 84 buildings, 31 reports included details of the structural system (RC frame or walls) as well as comments of damage to the structure and/or non-structural components (Figure 4). The surveyed buildings were between 1 and 8 storeys tall.

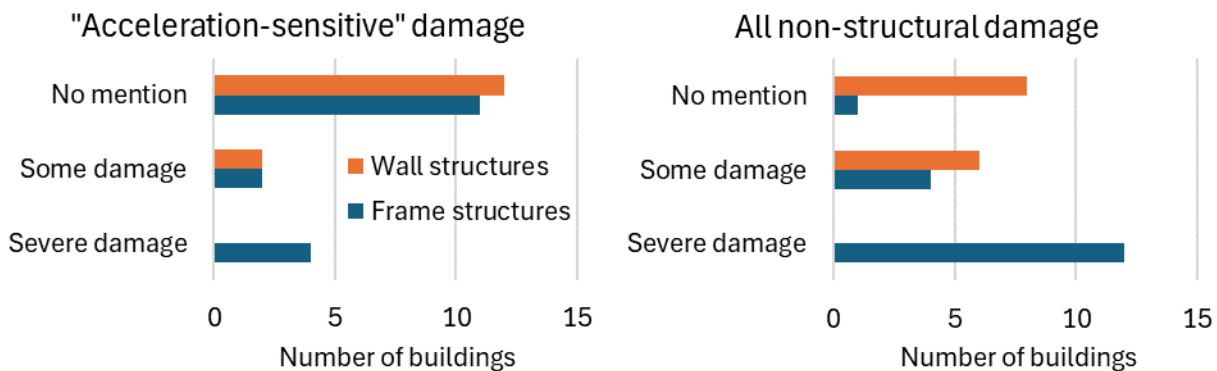


Figure 4: Non-structural damage in hospitals in Chile after large earthquakes. a) reported damage to ceilings, piping, and hospital beds and equipment (acceleration sensitive components) b) reported non-structural damage.

The median year of construction was 1985, with the oldest hospital being constructed in 1940, and the newest in 2010. Damage to non-structural components in hospitals surveyed in Chile was largely concentrated in RC frame structures as opposed to the hospitals with RC walls (Figure 4). Notably, only two instances of damage to ceilings or other “acceleration-sensitive” components such as hospital beds, services, piping, or HVAC were reported in the 14 RC wall structures. Meanwhile, 6 out of 17 RC frame structures had reports of damage to “acceleration-sensitive” components and 16 out of 17 had some mention of non-structural damage. These observations appear to contradict the idea that stiffer structures always experience more damage to non-structural components. The writers acknowledge the small sample size of these preliminary findings and are currently working on compiling more data from past earthquakes.

Conclusions

Observations from past earthquakes and results from numerical studies have clearly demonstrated the benefits of reducing design drift limits. This study explored the increase in initial cost associated with making structures stiffer and reducing the design drift limit.

- In the hypothetical cases considered, and for the unit prices assumed and listed in Tables 3 and 4, reducing the drift limit from 2.5% to 1.0% lead to relative increases in the building cost of 1 to 2% depending on the hazard level considered and assuming walls would be used instead of frames to meet more stringent drift limits.
- For a similar cost, wall structures can be designed to a drift limit half that of a frame structure.

It is more cost efficient to minimize drift by using walls, than by increasing the size of the frame members. It is unclear if reducing the drift limit translates to more seismic loss and damage to ceilings and services deemed to be “acceleration sensitive”. The field evidence presented does not support that notion. And ongoing experiments are expected to provide additional information on the matter.

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RECENT ADVANCEMENTS IN DESIGN AND FABRICATION OF ECCENTRICALLY BRACED FRAMES FOR ENHANCED DUCTILITY, STABILITY, AND RESILIENCE

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Abstract

Steel Eccentrically Braced Frames (EBFs) have high lateral stiffness, owing to their diagonal braces, and high energy dissipation capacity through inelastic deformations of the segment of the floor beam, referred to as the yielding link. As such, EBFs facilitate a design where not only peak global deformations can be mitigated effectively, but also the design forces can be reduced substantially. Despite these advantages, conventional EBFs have several limitations that affect their performance in terms of ductility, global and local stability, and their overall resilience. In addition, conventional EBFs lead to challenges in design, detailing, fabrication, and repairs after earthquakes. This paper provides an overview of a series of research projects aimed at improving the performance, design, and fabrication of EBFs, to ultimately result in more resilient EBFs. A new generation of replaceable yielding links, referred to as cast steel replaceable modular yielding links, were designed and experimentally validated. The proposed links remove the need for strict detailing requirements and demonstrate increased rotation capacities of up to 0.21 radians, while eliminating any instability failure modes such as local buckling or lateral torsional buckling. Further, they can be replaced rapidly after seismic events, increasing the overall resilience of the system. Recognizing that stable yielding systems are prone to global seismic instabilities and large residual deformations, a strongback system, with replaceable elements was developed. The performance of the proposed strongback system was also validated through pseudo-dynamic hybrid simulations, which was shown to effectively enhance global stability and mitigate the residual deformations in EBFs.

Introduction

Background on EBFs. Steel Eccentrically Braced Frames (EBFs) were first proposed as a seismic force resisting system (SFRS) in Japan (Fujimoto et al., 1972) and were then studied extensively in the United States (Popov et al., 1976; Roeder and Popov, 1976; Malley and Popov, 1984; Kasai and Popov, 1986; Engelhardt and Popov, 1989; Okazaki and Engelhardt, 2005; 2007). The characteristics of EBFs allow the system to have an energy dissipation capacity comparable to special moment resisting frames (SMRFs), while effectively controlling the lateral drifts similar to special concentrically braced frames (SCBFs). Although various cross sections have been proposed to be used as the yielding link in EBFs such as built-up section (Itani et al., 2003), box sections (Berman and Bruneau, 2007), and double channels (Mansour et al., 2011), yielding links are primarily fabricated from wide flange sections (W-Sections).

Limitations of EBFs. Despite the advantages they provide, there are several aspects in the performance of EBFs that can be improved. Yielding links in EBFs must be detailed properly with web stiffeners to ensure stable yielding of the web. Furthermore, yielding links are susceptible to both local buckling in flanges, in the case of flexural links, and global lateral torsional buckling for both shear and flexural links. For mitigating lateral torsional buckling, lateral supports are often required. In addition, designing yielding links requires iterations in the design for two reasons: (1) it is challenging to prevent the yielding of the beam outside of the link as the link element is a part of a beam, and (2) finding a balanced design between the lateral drifts and link plastic rotations is not a trivial process. In addition, the repair process in EBFs following an earthquake is challenging, as the damaged yielding component is integral with the floor beam.

In order to address some of these challenges, replaceable links were proposed around a decade ago (Mansour et al., 2011), which decouple the yielding link from the EBF beam allowing for a more convenient post-earthquake repair process. In addition, using replaceable links allows for an easier design process as preventing the beam outside of the link from yielding is easier. These design advantages led to a wide interest in replaceable links particularly in the rebuilding of Christchurch (Bruneau and MacRae, 2017). However, replaceable links require stringent detailing requirements and are susceptible to the same failure mechanisms as conventional links, and therefore, their design rotation is limited to 0.08 radians.

Lastly, EBFs like other stable yielding systems could be susceptible to global seismic instability and formation of soft-story mechanism under significant second-order P-Delta effects, which could lead to large peak deformations, large residual deformations, and even collapse in extreme cases.

Paper Layout. This paper provides a brief overview of recent research by the authors aimed at addressing the above-mentioned limitations in steel EBFs to improve their performance and resilience, and to simplify their design and construction. In the first section, previous research on the experimental validation of cast steel replaceable yielding links is briefly presented, including reversed-cyclic tests and pseudo dynamic hybrid simulations on EBFs using cast steel links. In the sections to follow, the development of a strongback system is briefly reviewed, which was developed to enhance the global seismic stability of steel EBFs and mitigate their residual deformations, which is crucial for seamless repairs after major earthquake and overall resilience of the system. In the ends concluding remarks are presented.

Cast Steel Replaceable Yielding Links in EBFs

Background. The use of steel casting in structural engineering has increased in the last few decades given the advantages it offers. The freedom of geometry that is offered by steel castings allows engineers to propose complex shapes for an improved structural performance, which are not feasible with welded fabrication. Use of steel casting for fabrication of yielding links in EBFs was first proposed by Tan and Christopoulos (2016), where finite element analyses were performed in order to find the optimal geometric designs. Cast steel links with hollow box regions were proposed, where both depth and width of the hollow box regions tapered along the length. This design approach was used to achieve simultaneous flexural yielding under a double curvature moment diagram.

Experimental Validation. A detailed design of optimized cast steel links was provided by Mortazavi et al. (2023a). A box section was chosen for CMLs to prevent them from lateral torsional buckling, without the need for stiffeners. In addition, a strict flange compactness limit was considered in the design to avoid local buckling limit states. In the initial design approach, CMLs were designed to feature a constant yielding length. Figure 1 depicts a CML100, which consists of: (1) the yielding regions comprised of a tapered hollow box such that it undergoes simultaneous yielding, (2) the elastic segment for transferring moment, shear, and axial forces, (3) the transition fillets, and (4) the integral cast endplates. In the naming convention of CMLs, the acronym CML is followed by a number, which is the nominal plastic shear capacity (V_p) of the link in kips. Four link sizes were experimentally validated in the study ranging from smallest to largest sizes. Experimental setups including a component test setup and a frame test setup, as well as selected results are shown in Figure 2. While the CMLs experienced no axial loads in the component test setup, the CMLs tested in the frame setup experienced an axial load of $P_{link} = 1.1V_{link}$, where P_{Link} and V_{Link} are the link axial and shear forces, respectively. Therefore, these experiments provided means for understanding the response of yielding links in EBFs under the effects of axial loads as well (Mortazavi et al., 2024a). While small to medium size links demonstrated large rotation capacities of 0.19 – 0.21 radians, the largest link size had a more moderate capacity of 0.11 radians. This was primarily because the CMLs were designed to have a constant yielding length, which was shown to have an indirect correlation with the link ductility.

In the second design approach, cast steel links were designed to feature varying yielding lengths across different sizes, such that a constant ductility was maintained across different link sizes Mortazavi et al.

(2023b). CMLs were designed such that links with a larger shear capacity feature a longer yielding length, while smaller capacity links feature a shorter length. This was done to target a more consistent ductility capacity across different link sizes. Finite element analyses (FEAs) and a simple fracture estimation approach were directly considered in the design process to ensure that the yielding length of the links is selected such that their ductility capacity matched the targeted link rotation capacity. Two link sizes were tested in a subsequent experimental program, in which responses were shown to match the ductility targeted in the design. Additional details on the recently developed cast steel links and their experimental validation can be found in Mortazavi et al. (2023b).

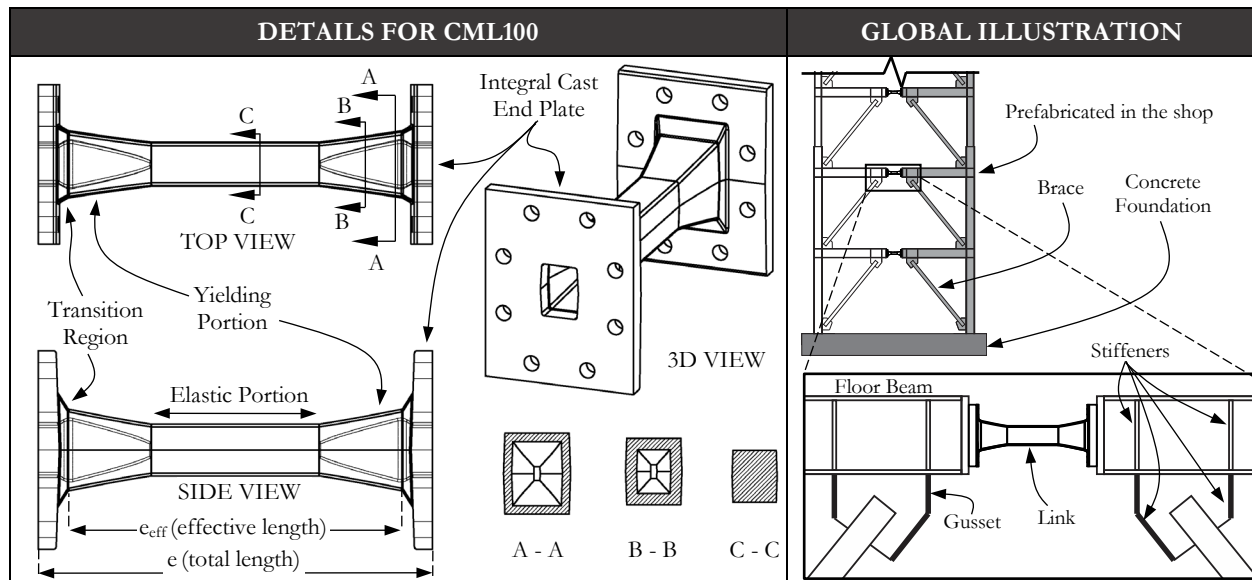
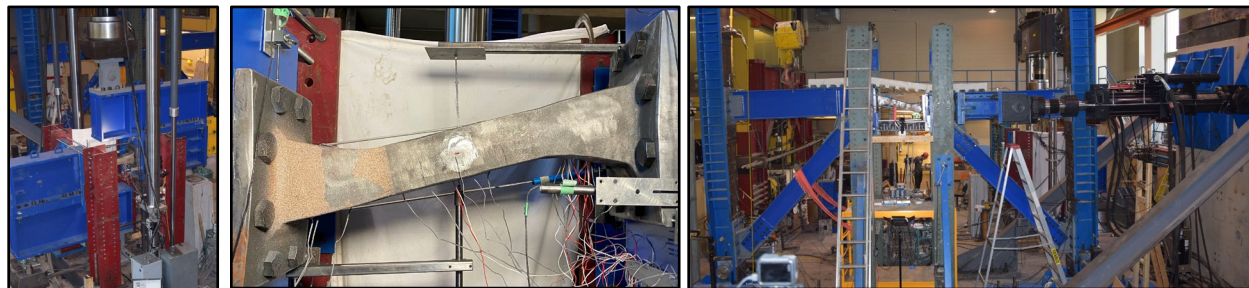
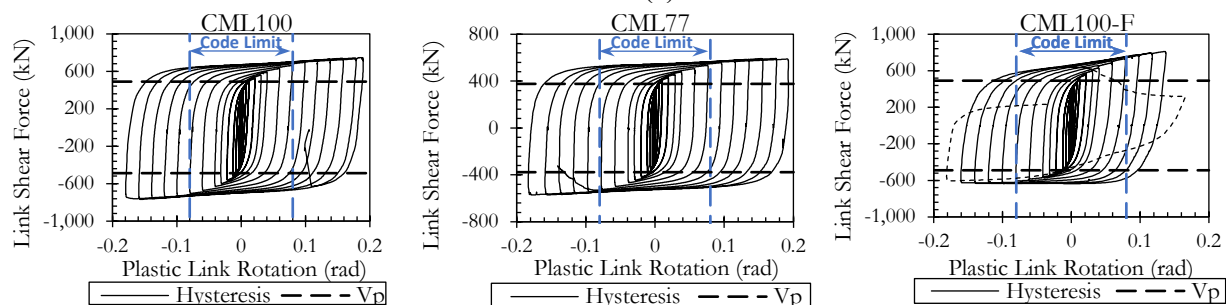


Figure 1. Overview of Cast Steel Replaceable Modular Yielding Links.



(a)



(b)

Figure 2. Experimental Program: (a) Experimental Setups, (b) Hysteretic Responses

Pseudo-dynamic Hybrid Simulations. In a subsequent study, pseudo-dynamic hybrid simulations were performed on two archetype buildings, designed with EBFs equipped with CMLs located in downtown Los Angeles (Mortazavi et al, 2023c). The final design of the EBFs that were studied in the hybrid simulations are shown in Figure 3. The purpose of these experiments was to understand the global response of EBFs with CMLs under real earthquake excitations. The first set of hybrid simulations were performed on the four-story archetype EBF where the response of the first story CML was physically captured in the component test setup, while the rest of the system was modeled numerically in OpenSees (McKenna et al., 2000). The communication framework between the physical substructure and the numerical model was facilitated using the UT-SIM framework (Huang et al., 2020; Mortazavi et al., 2017). The second hybrid simulation was performed on a two-story reference structure, which was designed such that the top floor CML would experience the same level of axial load as that present in the frame test setup. Therefore, the CML located on the second floor was physically tested in the frame setup while the rest of the system was modelled numerically in OpenSees. Selected results from the hybrid simulations are shown in Figure 4. The hybrid simulations demonstrated that CMLs provide stable hysteretic response and energy dissipation under real earthquake ground motions. Results from the hybrid simulations were also used for the development and calibration of numerical models for CMLs. One important observation from the hybrid simulations was significant residual deformations after earthquakes, especially notable residual link rotations, which could hinder the replacement of the yielding link after an earthquake and elongate the recovery time for the building.

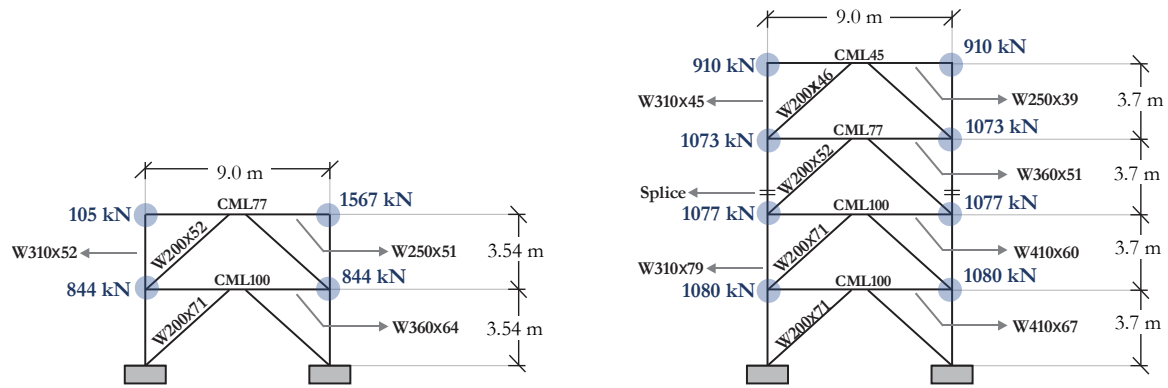


Figure 3. Reference EBFs Studied in Hybrid Simulations.

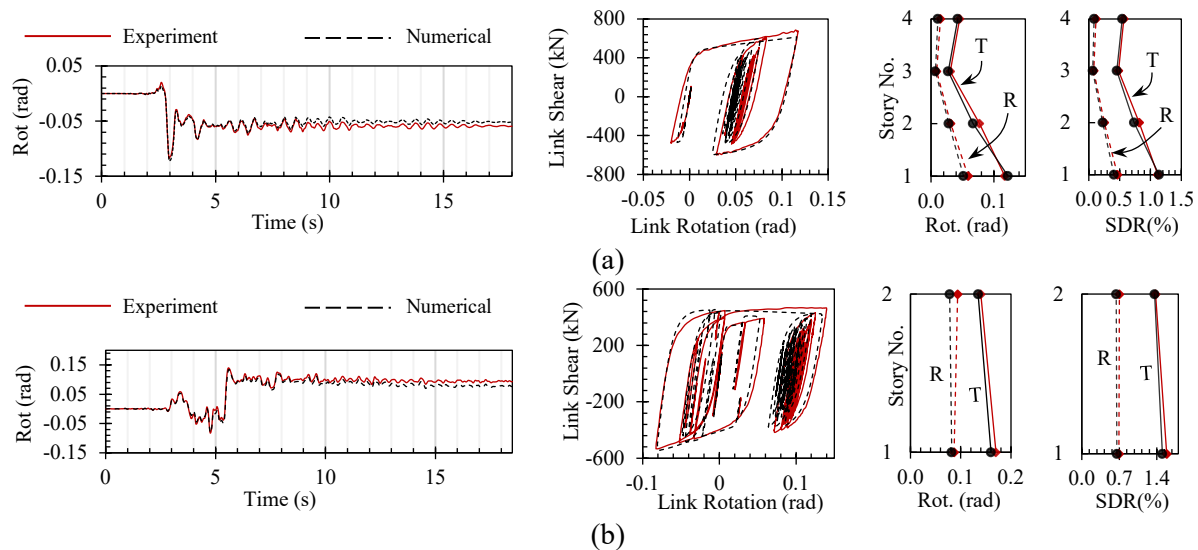


Figure 4. Selected Results from the Hybrid Simulations: (a) Four-story Structure, (b) Two-story Structure.

Design Guidelines and Codification. The experimental program was complemented by a comprehensive numerical study on building structures, ranging from 2- to 12-stories, designed in downtown Los Angeles (Mortazavi et al., 2024b). The buildings were designed with EBFs with conventional links, replaceable links fabricated from W-Section links, and CMLs. The results demonstrated two main findings. Firstly, it was shown that the global response of EBFs designed with CMLs is generally comparable to EBFs designed with W-Section yielding links, while EBFs with CMLs have larger link rotation capacity. Secondly, it was shown that the response of EBFs is accompanied by large residual deformations, especially significant residual link rotations. In the case of replaceable links which were developed with the aim of getting replaced after major earthquakes, this excessive residual link rotation can hinder the link replacement process and at times make it impossible. This aspect was addressed in the next phase of the project, which is discussed in the next section. The findings from the experimental programs discussed above and the subsequent numerical studies were eventually used to develop guidelines for the design of EBFs with CMLs, which were adopted by CSA-S16:24 (Mortazavi et al., 2024c).

Strongback System for Enhanced Stability and Resilience

Concept Development. Recognizing that the response of stable yielding systems is susceptible to large residual and peak deformations (Erochko et al., 2011; Fahnstock et al., 2007; Mojiri et al., 2021) which was also shown in the numerical studies, which can also get magnified by P-Delta effects, a low-cost strongback system was developed to enhance the global seismic stability of EBFs and to mitigate large residual deformations in the system to facilitate seamless replacement of the yielding links (Mortazavi et al., 2024d). The proposed strongback system is shown in Figure 5. In the proposed system, two hollow structural steel (HSS) beams are designed to work in parallel with the yielding link to provide a restoring force during the response of the system. The HSS members are designed to have a longer length such that they experience much smaller rotation compared to the yielding link. Further, they can be sized to provide a specific post-yield stiffness for CMLs at each floor level. The HSS beams are connected to the floor beam using the detail shown in Figure 5 (b) and (c). The end connection is detailed with slotted holes such that the end connection can transfer both shear and bending moments, but not any axial loads. This ensures that the HSS beams do not impose any axial restraints on the CMLs.

Unlike previous strongback system, the proposed system features replaceable elements which can get replaced after an earthquake if needed. Furthermore, the HSS members can be sized independently for each floor, which can lead to an optimized design. The HSS beams can have a low stiffness (i.e., less than 5% of the seismic force resisting system) which ensures that the overall base shear is not increased significantly. For tall EBF buildings, the HSS beams can be sized to counter the P-Delta effects and provide a minimal level of post yield stiffness. The HSS members are connected to the beam outside of the link, close to the inflection point in the moment diagram, which ensures that the beam is not overloaded. To increase the range of their elastic response, they can be selected from high-strength HSS grades (Ma et al., 2017). Additional details about the mechanics of the proposed system are provided by Mortazavi et al. (2024d).

Concept Validation Tests. The proposed strongback system was adopted for the reference structures shown in Figure 3. HSS beams were sized for each floor such that they provide an additional post-yield stiffness ratio of 4%. The design of the EBF members was not changed to evaluate the effect of the HSS beams on the response of the capacity protected elements. The hybrid simulations on the reference structures were repeated, where the response of the CML was captured physically while the rest of the structure was modelled numerically. The HSS beams were incorporated into the OpenSees numerical model, given that they primarily remain elastic, and that their response is predictable. Selected results from the hybrid simulations are shown in Figure 6, which demonstrates that the proposed strongback system is effective in mitigating both peak and residual deformations. Additionally, it was observed that adding the HSS beams did not affect the response of capacity protected elements.

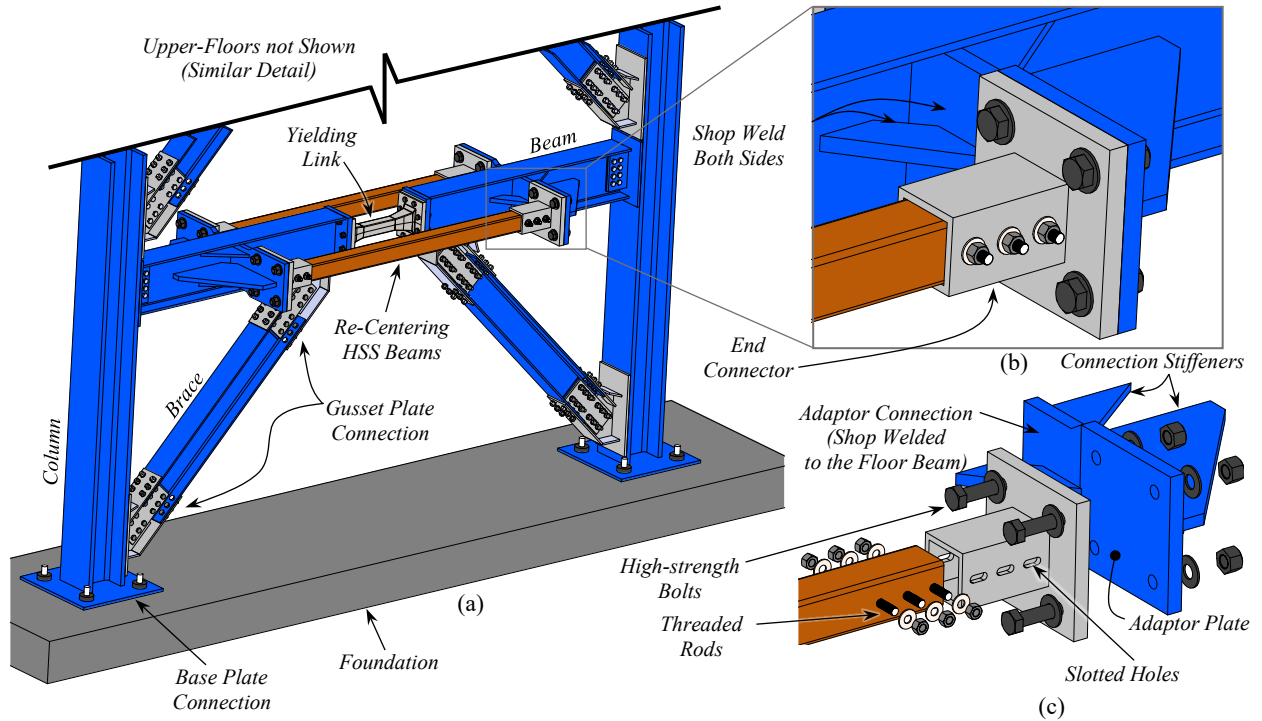


Figure 5. Proposed Re-Centering System: (a) EBF View, (b) Connection Detail, (c) Connection Parts.

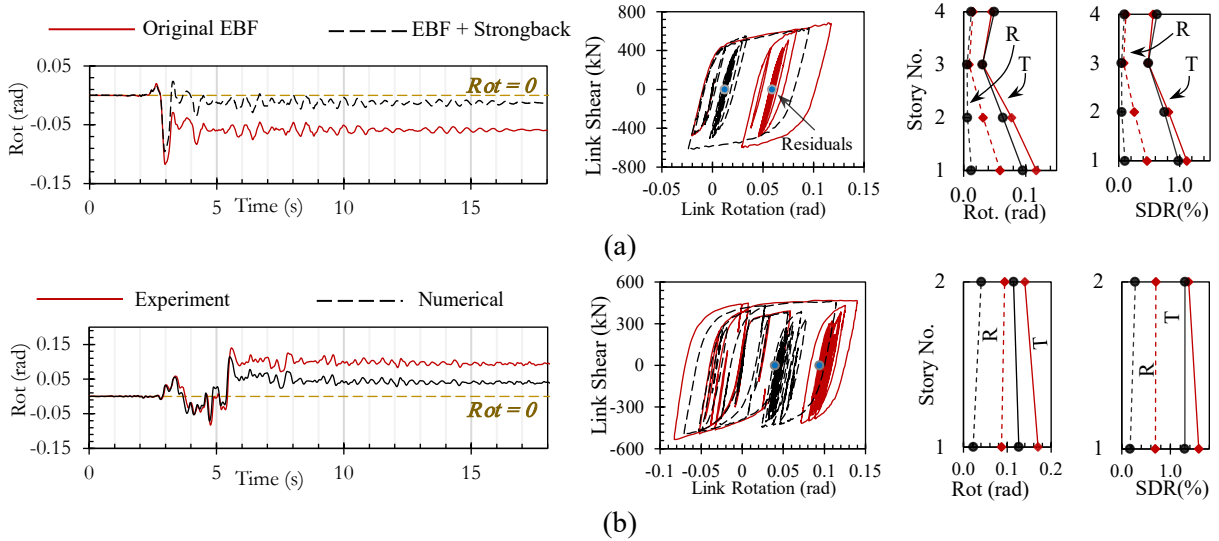


Figure 6. Results from the Hybrid Simulations: (a) Four-story Structure, (b) Two-story Structure.

Summary and Conclusions

This paper provides an overview of a series of studies aimed at addressing some of the shortcomings of steel EBFs not only to improve their performance, stability, and resilience, but also to simplify their design, construction, and reparability after major seismic events. The findings from the study can be summarized in the following.

- Cast replaceable modular yielding links demonstrate a highly improved performance compared to conventional links, reaching link rotations up to 0.21 radians. In addition, the design and fabrication of CMLs is significantly simplified as there is no need for designing the link stiffeners and endplates

with strict detailing requirements. Additionally, CMLs are designed to eliminate undesirable stability failure modes such as local buckling and lateral torsional buckling. CMLs are designed to get connected to the floor beams through bolted endplate connections. This facilitates modular construction and allows for replacing the damage links after major earthquakes.

- The increased rotation capacity of CMLs eliminates/reduces the design iterations that are typically required for controlling the link rotation as part of the design of EBFs.
- Results from pseudo-dynamic hybrid simulations and subsequent numerical studies demonstrated that the global response with CMLs is generally comparable with EBFs designed with other yielding links.
- Both the hybrid simulations and the numerical studies showed that EBFs are susceptible to large residual deformations, which could also magnify due to large P-Delta effects. This could hinder the link replacement process and elongate the recovery time of EBFs after major seismic events.
- A strongback system was developed to enhance the global seismic stability of EBFs and mitigate their residual deformations, with the aim to expedite the repair process and improve their resilience. The proposed strongback system was adopted for two reference structures and was shown to effectively mitigate both peak and residual deformations in EBFs.

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RECENT DEVELOPMENTS FOR SEISMIC DESIGN, RETROFIT, AND REPAIR OF CONCENTRICALLY BRACED FRAMES

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Abstract

Centrically braced frames (CBFs) are widely used in steel building construction to resist earthquake loads due to their relatively high strength and stiffness and architectural adaptability. In regions with high seismic risk in the US, a wide variety of CBFs exist including CBFs built up to the 1990s with potentially nonductile behavior; special CBFs designed for high ductility; or, in low-rise infrastructure only, ordinary CBFs designed for moderate ductility. These systems have been the subject of substantial research over the past decade that has significantly advanced evaluation and design standards. This presentation will discuss recent and ongoing developments in the American Institute of Steel Construction (AISC) *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341) and *Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings* (ANSI/AISC 342) through the lens of the fundamental experimental and computational research that has shaped these documents. Major highlights include the relaxation of the required strength of beams intersected by chevron-configuration braces, introduction of new modeling parameters and acceptance criteria for braces and gusset plates, and revision of brace width-to-thickness ratio limits. Finally, with a view toward enhanced resiliency of CBFs, new developments for repair-oriented design of CBFs are discussed.

Introduction

Centrically braced frames (CBFs) are mainstays of steel lateral-force-resisting systems in US construction. The primary resistance of CBFs is developed through tension and compression of the diagonal braces, which are often hollow structural section (HSS) or wide-flange (W) shapes connected to the framing members with gusset plates that approximate pinned boundary conditions for out-of-plane buckling. Braces are proportioned and oriented to satisfy architectural and structural requirements, resulting in configurations including chevron, multi-story-X, single diagonal, and single-story-X configurations. A wide array of brace-end and beam-to-column connections are also used in practice. Although brace and connection configurations appear to be simple design decisions, they can result in the development of different plastic mechanisms, yielding hierarchies, and failure modes that significantly affect seismic performance. These variations render it challenging to generalize the inelastic behavior of CBFs.

Despite the complex inelastic response of these systems, substantial research has been conducted in the past several decades to advance the state of the practice. This research includes subassembly testing to investigate seismic behavior of new and existing systems (Lehman et al. 2008; Roeder et al. 2011; Sen et al. 2017; Simpson and Mahin 2018; Uriz and Mahin 2004), component testing to establish inelastic response of buckling braces (Fell et al. 2009; Lee 1988; Lehman et al. 2024; Tremblay et al. 2008), and computational research to develop new methods for nonlinear analysis and seismic performance evaluation (Hsiao et al. 2012; Karamanci and Lignos 2014; Sen et al. 2019; Tirca and Chen 2014). The design of CBFs in the US continues to evolve to respond to the needs of design practice that is becoming increasingly complex and cognizant of seismic resiliency and infrastructure sustainability objectives. New seismic design provisions for CBFs in the American Institute of Steel Construction (AISC) standards, ANSI/AISC 341-22 (AISC 2022a) and ANSI/AISC 342-22 (AISC 2022b), have respectively focused on improved design for special CBFs (SCBFs) in the chevron configuration and the introduction of new modeling parameters and

acceptance criteria for braces and gusset plates for evaluation of existing buildings. The rationale behind these new provisions and ongoing research on seismic design of CBFs in the US are discussed herein.

Concentrically Braced Frames in US Practice

In current US practice, three (3) categories of CBF systems are specified in new construction based on the American Society of Civil Engineers (ASCE) standard, ASCE/SEI 7-22, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 2022). These categories have varying intended ductility capacity and corresponding design requirements specified in ANSI/AISC 341-23, *Seismic Provisions for Structural Steel Buildings* (AISC 2022a). Table 1 summarizes the basic design parameters of these documents, where (1) special CBFs (SCBFs) are highly ductile systems with stringent capacity-based and ductile design requirements, (2) ordinary CBFs (OCBFs) are moderately ductile systems with overstrength-based design requirements, and (3) “ $R = 3$ ” CBFs are limited-ductility systems that are not specifically detailed for seismic resistance and only permitted at sites with low seismic risk. The required strength of these systems is based on the equal displacement rule, whereby providing greater inelastic deformation capacity permits a larger response modification factor, R , to reduce the design base shear.

Table 1. Basic Seismic Design Parameters for CBFs

<i>System</i>		<i>SCBF</i>	<i>OCBF</i>	<i>“$R = 3$”</i>
Response modification coefficient, R		6	3.25	3
Overstrength factor, Ω_0		2	2	3
Deflection amplification factor, C_d		5	3.25	3
Brace limiting width-to-thickness ratio		λ_{hd}	λ_{md}	NL
Beam/column limiting width-to-thickness ratio		λ_{hd}	NL	NL
Height limit (ft) based on Seismic Design Category (SDC)	SDC A/B/C	NL	NL	NL
	SDC D/E	160 ^a	35 ^a	NP
	SDC F	100 ^a	NP	NP

NL = No Limit; NP = Not Permitted

^aHeight limit can be increased in some circumstances.

A defining feature of SCBFs is that the design requirements enforce a specific yielding and failure hierarchy to achieve the required deformation capacity. In SCBFs, the braces are expected to buckle in compression and yield in tension while sustaining large inelastic deformations to dissipate seismic energy. The intended failure mechanism is brace fracture, which is expected to occur due to low-cycle fatigue. Adequate fracture life of the brace is ensured through a stringent cross-sectional width-to-thickness ratio limit, λ_{hd} . This controlled response mechanism is enabled by capacity-based design to avoid buckling of the columns and fracture of the connections when the braces fully yield in tension and maintain either the expected or post-buckling compressive strength in compression. Ductile detailing requirements, including minimum weld toughness and gusset plate detailing to accommodate brace buckling, ensure inelastic deformations can be sustained by the connections.

In contrast to SCBFs, OCBFs and “ $R = 3$ ” CBFs do not have designated yielding or failure hierarchies, which results in complex and uncertain inelastic system behavior. OCBFs have some characteristics of ductile design that are essentially relaxations of SCBF requirements, including a less-stringent cross-sectional width-to-thickness ratio, λ_{md} , for the braces, minimum weld toughness (but not demand critical welds), and pseudo-capacity-based design using an overstrength factor, Ω_0 . This overstrength factor is essentially used to double the horizontal seismic load effects on the connections, beams, and columns in the braced frame; however, this does not ensure that these components can develop the strength of the brace in tension and compression. As such, brace buckling and yielding is not necessarily expected, and the failure mode of the system could be brace fracture, connection fracture, or column buckling. “ $R = 3$ ” systems do not have any of these features but are not used in regions with high seismic hazards (i.e., strength of the lateral-force-resisting system is likely governed by wind, resulting in significant inherent overstrength for earthquake loading). However, it is important to note that many existing CBFs that remain in service today in regions with high seismic risk that were constructed prior to the codification of SCBFs and OCBFs; as such, these systems resemble “ $R = 3$ ” systems, and their vulnerability has been well studied (Sen et al. 2016b; a; Simpson and Mahin 2018).

Developments for New Design

Design requirements for SCBFs and OCBFs used in new construction in the US remain under scrutiny to ensure they reflect recent research on their inelastic response and seismic performance and address design considerations that develop from practical experience in applying the AISC *Seismic Provisions* (AISC 2022a). New developments for these provisions address the ongoing need to ensure design flexibility without compromising safety, which is guided by the life-safety performance objective defined in ASCE/SEI 7-22 (ASCE 2022). This objective states that, for buildings categorized as Risk Category I or II, the conditional probability of collapse in the risk-adjusted maximum-considered earthquake (MCE_R) is less than or equal to 10%. Here, the MCE_R is based on spectral acceleration with a 2%/50-year probability of exceedance that is adjusted to target an absolute probability of collapse of 1%/50 years. Two developments—one recently implemented in ANSI/AISC 341-22 and one the subject of ongoing research—are discussed below.

Beam Yielding in the Chevron Configuration. Chevron-configuration CBFs are arranged in brace pairs within one bay such that the braces intersect at the beam midspan in the frame. This configuration is often architecturally desirable because it can accommodate openings (windows, doors, etc.) that would not generally be possible if the brace(s) span across the entire bay. However, early research on chevron-configuration CBFs showed that the beams sustain large demands due to the unbalanced forces between the tension and compression braces in the bay (Foutch et al. 1987; Khatib et al. 1988; Roeder 1989). These forces arise because of the difference in tensile and compressive strength of the brace, and this difference generally increases in magnitude after brace buckling, when compressive resistance degrades due to P- δ effects. These observations led to plastic-mechanism-based design requirements for beams in the chevron configuration, which were eventually implemented for both SCBFs and OCBFs. Specifically, the beam must be designed to develop the axial-flexural demands due to the unbalanced, post-buckling brace forces as shown in Fig. 1a.

This requirement had a substantial effect on SCBF design practice due to the need for relatively heavy, deep beams within the braced frame. However, a recent experimental investigation of older CBFs with weaker beams characteristic of pre-1988 construction showed that beam yielding due the unbalanced brace forces may provide adequate or even improved inelastic behavior (Sen et al. 2016a). This concept was then comprehensively studied in the context of SCBFs by Roeder et al. (2019, 2020) and Asada et al. (2020) with support from AISC. This research showed that a relaxation of the mechanism-based design requirements resulted in controlled beam yielding, lighter and shallower possible beam shapes, reduced

likelihood of brace fracture, lower drift demands, and lower collapse probability at the 2%/50-year hazard level. The result of this work is Section F2.3, Exception (d) of ANSI/AISC 341-22, which states that, for design of the beam only, the expected brace strength in tension can be capped at the magnitude of the expected strength in compression (AISC 2022a). This exception greatly reduces the axial-flexural demands in the post-brace-buckling mechanism analysis (Fig. 1b), which in turn encourages the development of a yielding beam mechanism within the yielding and failure hierarchy. It is noted that, with this approach, brace fracture remains the expected failure mode.

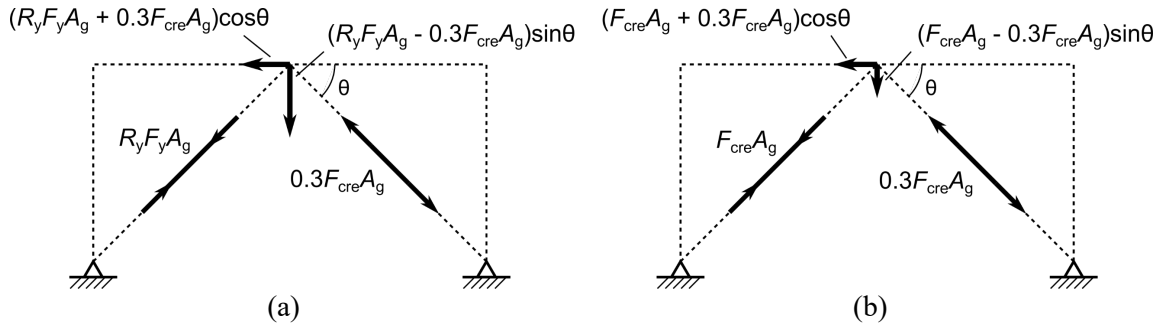


Figure 1. Chevron-configuration brace axial forces and beam resultant forces for post-brace-buckling load state based on (a) ANSI/AISC 341-16 and (b) ANSI/AISC 341-22 exception.

Brace Width-to-Thickness Ratios. Expressions for cross-sectional width-to-thickness ratio limits, λ_{hd} and λ_{md} , are defined for both SCBF and OCBF braces, respectively, to ensure that these members have adequate inelastic deformation capacity. The relationship between width-to-thickness ratio and fracture life has long been recognized (Goel 1992), but the precise rationale behind the established limits in Table D1.1 of ANSI/AISC 341-16 (AISC 2017) and its predecessors was not clear (Schafer et al. 2022). Further, substantial experimental research on braced frames and buckling braces over the past decade has generated significant data that were not available at the conception of these limits. New experimental and computational studies funded by AISC are underway to re-establish appropriate expressions for these width-to-thickness ratios with an emphasis on rectangular and round HSS braces. Notably, in this modernization effort, a collapse performance benchmark based on the life safety objective in ASCE/SEI 7-22 is used to provide system-level validation of the updated width-to-thickness ratio limits. This work is ongoing, and its findings are expected to be implemented in the forthcoming ANSI/AISC 341-27 standard.

Developments for Seismic Performance Evaluation and Retrofit

In US practice, seismic performance evaluation is guided by the ASCE/SEI 41-23 standard, *Seismic Evaluation and Retrofit of Existing Buildings* (ASCE 2023). In this most recent iteration of the standard, modeling parameters for steel structures are established by a new standard, ANSI/AISC 342-22. In view of the aforementioned large quantity of recent experimental research on braced frames of various vintages, ANSI/AISC 342-22 overhauls the modeling parameters and acceptance criteria for buckling braces and their connections. Notably, a new, general backbone curve is established for buckling braces (see Fig. 2a), and modeling parameters for rectangular HSS are established from regression analysis of 69 experiments with braces that sustained tearing or fracture due to low-cycle fatigue. Equation 1 shows the new expression for deformation capacity of rectangular HSS, d , which is a function of the normalized width-to-thickness ratio and global slenderness ratio. Figures 2b and 2c show how the new ANSI/AISC 342-22 and previous ASCE/SEI 41-17 (ASCE 2017) backbone curves compare to the experimentally measured hysteretic response of two braces. Similar modeling-parameter expressions for other shapes were reformulated from existing ASCE/SEI 41-17 values; updated modeling parameters for these shapes constitute an ongoing research and development need.

$$d = \left[3.0 \left(\frac{\lambda}{\lambda_{hd}} \right)^{-1.0} \left(\frac{L_c/r}{\sqrt{E/F_y}} \right)^{1.0} \right] \Delta_c \quad (1)$$

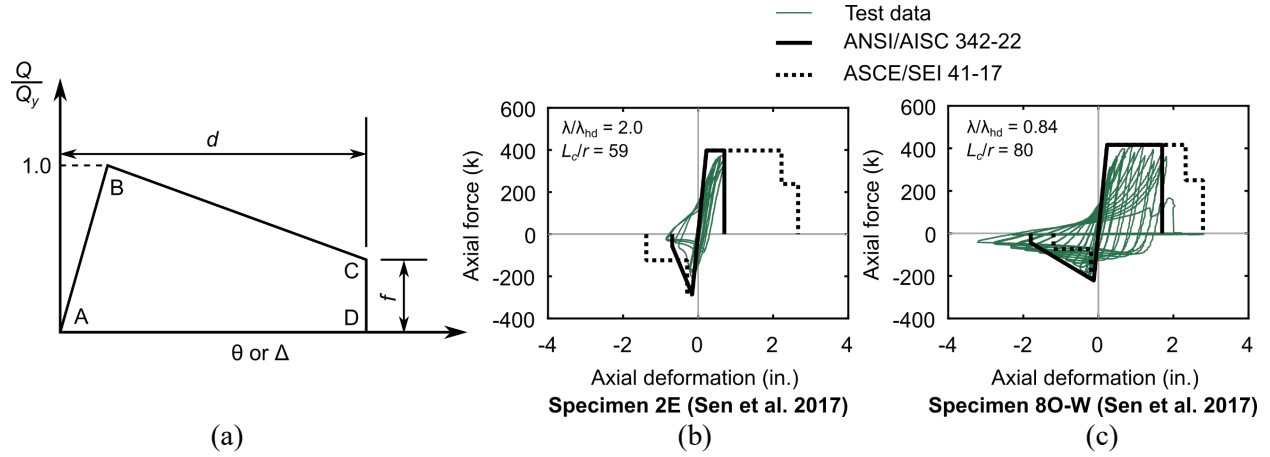


Figure 2. (a) ANSI/AISC 342-22 brace backbone curve; comparison of backbone curves with experimental data for a (b) nonductile and (c) ductile brace.

Research on CBFs constructed prior to about 1988 revealed that essentially all gusset plates in these vintage systems lack sufficient clearance to accommodate brace buckling and have gusset plate-to-beam or -column welds formed with relatively low-toughness filler metal that are insufficiently sized to develop the strength of the plate (Sen et al. 2016b). New provisions in ANSI/AISC 342-22 define the rotation capacity, d (rad), of these important interface welds based on the elliptical clearance-to-plate thickness ratio and weld demand-to-capacity ratio (Eq. 2).

$$d = 0.11 \left(\frac{L_{ell}}{t_p} \right)^{0.33} \left(\frac{f_{tUD}}{f_{tCE}} \right)^{-0.57} \quad (2)$$

Developments for Repair

Buildings with SCBFs, OCBFs, and other conventional seismic-force-resisting systems are expected to sustain damage in earthquakes to provide inelastic deformation capacity and dissipate seismic energy. For SCBFs, such damage may be in the form of residual brace and gusset plate deformation due to buckling and yielding, brace fracture, and yielding and local buckling of beams and columns. However, the development of this damage, which may necessitate significant capital for repair and downtime, conflicts with the growing call for the capacity for functional recovery of building infrastructure in the US. Ongoing experimental research funded by the National Science Foundation seeks to develop new strategies for repair-oriented design of SCBFs using segmented braces. In this approach, the midspan segment of a buckling brace is the designated region for damage and repair, and this midspan segment is designed to be replaced through a bolted end plate connection. Initial experimental testing and finite-element analysis of this concept (see Fig. 3) has shown that, at the precipice of brace fracture (2.5%-3.0% story drift), yielding in the beams, columns, and gusset plates is limited such that repair of the brace and retention of these components may be feasible. However, the segmented brace design may result in strain concentration within the yielding midspan segment that reduces brace's inelastic deformation capacity. Further research is needed to establish valid conditions for repair and design of the segment.

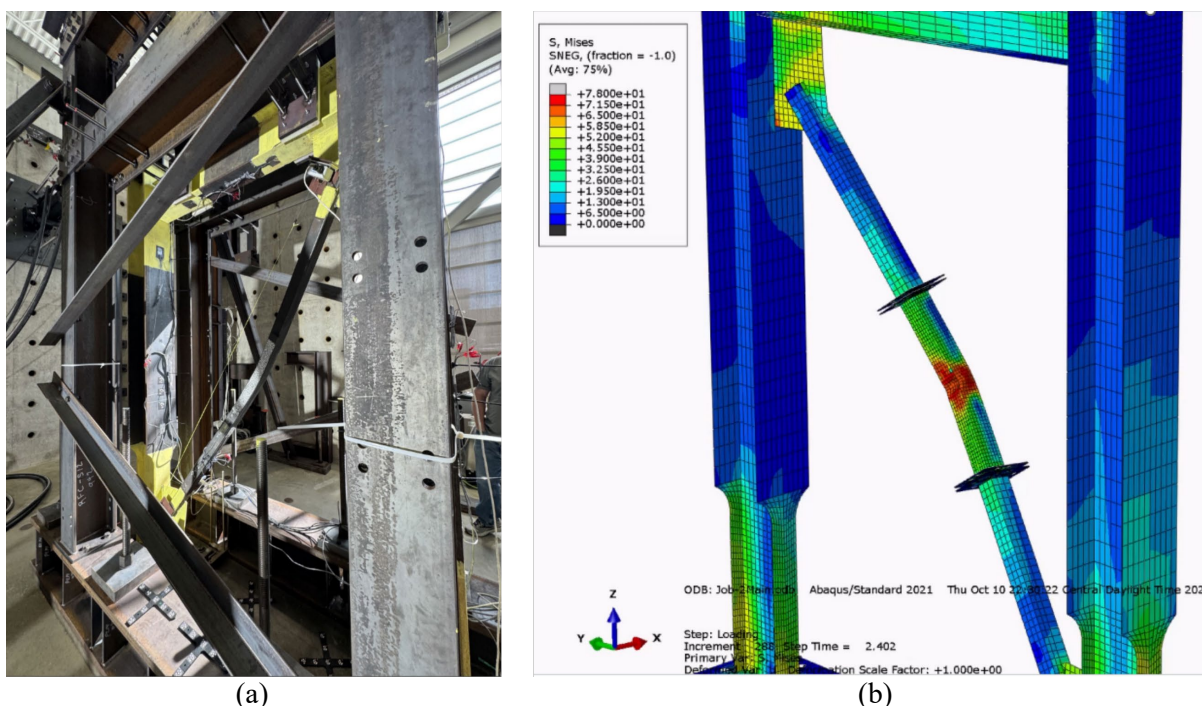


Figure 3. (a) Experimental testing of conventional SCBF at 2% target story drift and (b) finite-element analysis von Mises stress contours of SCBF with segmented brace.

Summary and Conclusions

Although the design of SCBFs and OCBFs has already changed significantly since the 1980s, their design provisions continue to evolve in response to novel research findings, practical concerns, and shifting societal expectations. Based on the latest edition of ANSI/AISC 341, SCBFs with chevron-configuration braces can be designed with lighter, shallower beams that permit controlled beam yielding, which has been shown to be beneficial for performance. In the new ANSI/AISC 342-22 standard, the brace backbone curve and modeling parameters have been revised from ASCE/SEI 41-17 to provide simulated response consistent with experimental data. In addition, new provisions for gusset-plate connection modeling have been added to simulate gusset plate interface weld fracture, which is a prevalent failure mode for vintage CBFs. Finally, ongoing work is underway to revise brace width-to-thickness ratio limits and develop repair-oriented design strategies for new CBFs using segmented braces. These changes and research directions emphasize that ANSI/AISC 341 and ANSI/AISC 342 are living standards that will continue to adapt to reflect consensus best practices.

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OVERVIEW OF A PROJECT ON RESILIENCE-BASED DESIGN OF BUILDINGS WITH ACCELERATED POST-EARTHQUAKE FUNCTIONAL RECOVERY

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Abstract

With urban populations rising, the demand for multi-story buildings in seismic-prone areas like California is increasing. However, recent earthquakes have revealed that even newly designed multi-story buildings can collapse, resulting in fatalities, financial losses, non-functional communities, and significant environmental impacts due to debris. The U.S. National Earthquake Hazards Reduction Program (NEHRP) emphasizes national earthquake resilience, underscoring the need for buildings to recover functionality quickly after strong earthquakes to enhance community resilience. FEMA identifies the development of "better-than-code" structural systems as a key research need for future NEHRP Provisions. Supported by the NIST/NSF Disaster Resilience Research Program (2024-2027), our research focuses on developing resilience-based designs for buildings to ensure accelerated post-earthquake functional recovery. Specifically, we investigate buildings with rocking inelastic base mechanisms and practical force-limiting deformable connections strategically placed in the seismic load path. These systems are preferred over conventional designs due to their potential to achieve faster and more reliable post-earthquake functionality, owing to their limited seismic responses and reduced sensitivity to the variability in strong earthquake ground motions. This paper and corresponding presentation will provide an overview of the basis of the proposed research project.

Introduction

Despite advancements in seismic engineering, the last two decades have witnessed a troubling increase in earthquake-related fatalities and damages (CRED 2022; Tsampras 2016). This rise can largely be attributed to higher population densities in seismically active regions and the complexities involved in modern urban structures, particularly multi-story building constructions. Such trends underscore a critical shortfall in traditional building designs, which, while intended to preserve life, often fail to promptly restore building functionality, resulting in significant economic losses and community disruption. Recognizing this gap, there is a pressing need to rethink current design strategies and adopt resilience-based objectives that not only aim to protect lives but also ensure rapid functional recovery after earthquake events that exceed the design intensity level.

This paper discusses the shortcomings of current seismic design practice on community resilience; and introduces a comprehensive approach incorporating advanced structural solutions to enhance post-earthquake functional recovery of buildings.

National Earthquake Resilience. Chen et al. (2023) discussed the need for national earthquake resilience which is one of the priorities of the National Earthquake Hazards Reduction Program (NEHRP) (Congressional Research Service 2021; FEMA 2020). Designing buildings to recover a functional level promptly after a strong seismic event is required to contribute to community resilience and, as a result, enhance national earthquake resilience. Individual buildings can be designed to return to a target functionality state after a given period if resilience-based design methods are used (FEMA 2020).

Designing buildings for higher performance can mitigate the damage in the structural and non-structural components expected in conventional building designs.

FEMA P-58 (FEMA 2018) provides a methodology for designing buildings using performance-based seismic design principles including performance- and resilience-based design objectives. These objectives can include the damage that buildings may experience and its consequences considering potential casualties, loss of use or occupancy, and repair/reconstruction costs. ATC 138-3 provides a methodology for assessing functional recovery time (ATC 138-3 2021). These methods are different from those included in the current seismic design provisions for conventional buildings, which are prescriptive and rely on ensuring life-safety performance levels (for Risk Category II buildings), and do not provide explicitly the means to design buildings for performance levels other than life safety (FEMA 2018; McAllister et al. 2022).

The FEMA/BSSC/PUC functional recovery planning committee addresses complex questions such as: “How can we reduce the risks of life and property from future earthquakes and increase the resilience of communities in the United States?” (FEMA/BSSC/PUC 2024). Addressing these important questions requires a clear definition of resilience metrics in the context of seismic design of buildings.

Design for Functional Recovery. The engineering community is shifting the seismic design criteria of buildings from life safety to pursuing more ambitious design objectives, such as immediate occupancy (Burton et al. 2016). Building occupancy is associated with a specific level of building functionality. So, it is necessary to implement methods to improve the functional recovery of buildings after a strong earthquake ground motion (Davis et al. 2022; FEMA P-2090 2021; Burton et al. 2016). Building resilience is rooted in the concept of building functionality (NASEM 2021); therefore, most resilience metrics are built on functional recovery curves that typically show the evolution of a building functionality metric over time (Bocchini et al. 2014). Functional recovery curves can be associated with each post-disaster damage state to determine the evolution of functionality over time and obtain a set of samples of restoration functions (Karamlou and Bocchini 2017b). This collection of samples can be used for the probabilistic assessment of resilience, which focuses not only on the expected (i.e., mean) recovery path but also on its variability (Karamlou and Bocchini 2017a).

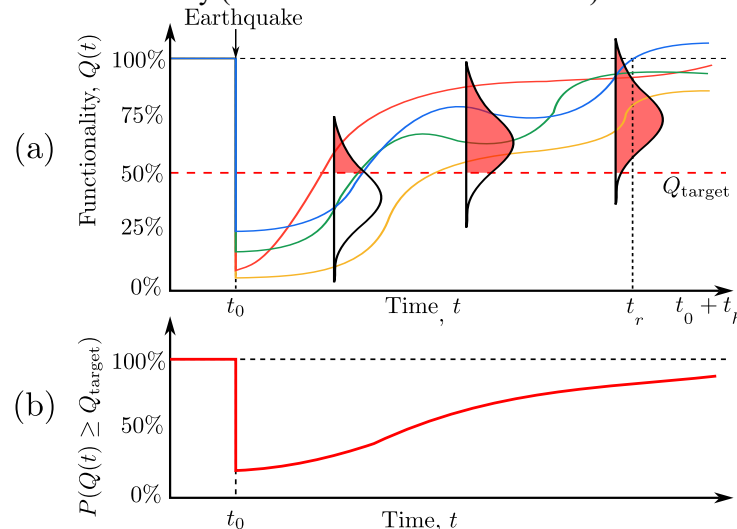


Figure 1. (a) Samples of functional recovery curves and (b) probability of exceeding a target functionality level.

Figure 1a shows samples of functional recovery curves where $Q(t)$ is a building functionality metric (e.g., percentage occupancy) and t is time. The repair actions will allow to safely reopen portions of the

building over time, and the functional recovery path will depend on the initial damage state and the effectiveness of the repair actions. The fact that the initial damage and the recovery curve are random yields a probability distribution for the functionality level at each point in time. One metric that is commonly used to capture this uncertainty is the probability of the functionality exceeding a certain target level (see Figure 1b) (Karamlou and Bocchini 2017a).

Metrics such as the one above can be used to design buildings for a target functional recovery. However, design for accelerated post-earthquake functional recovery for conventional buildings may be challenging considering the inherent performance requirements for structural and nonstructural components in conventional buildings (e.g., moment resisting frame systems, braced frame systems, bearing wall systems).

Challenges for Conventional Buildings. The current seismic design practice for conventional buildings focuses on practical and easy-to-implement methods that use design response spectra to represent the seismic demand. They can be classified as force-based design methods (Uang 1991; American Society of Civil Engineers 2022) that estimate the force demands of buildings using seismic design modification factors and design response spectra; displacement-based methods (Moehle 1992; M. Priestley et al. 1996; M. J. N. Priestley et al. 2007; Fardis 2018) that are performance-oriented and account for the nonlinear evolution of stiffness of an equivalent single degree of freedom system; or hybrid methods (Karavasilis et al. 2010; 2007; Tzimas et al. 2013; Tsampras 2011) that use performance-oriented displacement-based design principles framed in the context of force-based design. All these methods are approximate and include assumptions and limitations; however, they provide effective and efficient procedures to design buildings with different seismic force-resisting systems that meet mass and stiffness distribution, and height requirements.

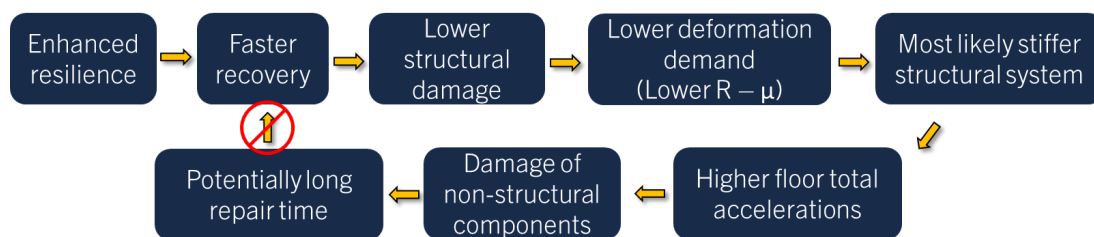


Figure 2. Flowchart of the trade-off between building seismic responses associated with structural damage and those associated with damage to non-structural components for conventional buildings

Although the current design methods for conventional buildings can provide reasonable seismic performance during an earthquake ground motion, researchers and engineers are trying to promote faster functional recovery after a strong earthquake ground motion to enhance the resilience of our community. However, there is a conflict in the ability to achieve faster functional recovery of conventional buildings after an earthquake since there is a trade-off between the building seismic responses associated with structural damage and those associated with damage to non-structural components. This trade-off can be explained using the flowchart shown in Figure 2. Consider for example a moment-resisting frame or a braced frame that inherently modifies the seismic response of buildings by allowing local inelastic response in deformation-controlled structural members. If this conventional building is designed using the current design methods to enhance the resilience of our community, it is required to aim for faster functional recovery after an earthquake. Therefore, the post-earthquake damage of the structural system needs to be below a certain limit. This implies a design for lower deformation demand that requires lower seismic response modification factors providing lower ductility demand. So, the building will most likely be stiffer, and as a result, higher floor accelerations are expected. The higher floor accelerations may produce increased damage to non-structural components and potentially longer repair time, which

prolongs the required post-earthquake recovery time and prevents the enhancement of seismic resilience. For this reason, there is a need to focus on “better-than-code” buildings to achieve resilience-based design objectives.

In addition, conventional buildings inherently tend to develop damage in locations that engineers typically do not design for. Consider for example a reinforced concrete (RC) wall building in which the earthquake-induced inertial forces are transferred through the floor diaphragms to the RC walls, which transfer these forces to the foundation system. The walls are designed to develop an inelastic flexural mechanism at the base of the walls. The inelastic flexural base mechanism region is carefully detailed to provide a ductile moment-rotation response at the base of the walls. However, other regions of the building such as the connections between floor diaphragms and the walls are not usually designed to take excessive deformation demands; therefore, damage is expected in these connections because of the kinematic compatibility between the connected components and the potential change in the force distribution over the height of the walls due to the higher-mode effects. This issue is present in more advanced building systems, as well. This was demonstrated by the experimental tests conducted by Sritharan et al. (Liu et al. 2018; Sritharan 2015). They tested a rocking wall system including a portion of the floor slabs, in which the wall was designed to rock at its base; therefore, the system developed a controlled rocking base mechanism instead of an inelastic flexural base mechanism typically associated with structural damage. The restoring moment at the base of the wall was controlled by post-tensioning and energy dissipation systems reducing or eliminating the structural damage at the wall base. However, excessive structural damage was observed in the connection between the floor diaphragm and the rocking wall. This structural damage that engineers do not design for, even for more advanced building systems, may result in increased uncertainty in the seismic response of buildings. Understanding that this challenge potentially leads to increased uncertainty in the seismic response and considering that there is variability in the characteristics of earthquake ground motions and variability in the evolution over time of dynamic properties of buildings, the level of confidence in the prediction of the seismic response of buildings through numerical earthquake simulation, and therefore the confidence in the resulting design of these buildings, is negatively affected.

To overcome the trade-off between the seismic responses related to structural and nonstructural damage aiming for resilience-based design objectives and mitigate or eliminate the damage in locations that engineers typically do not design for, it is recommended to use “better-than-code” building systems.

Buildings with Force-Limiting Connections. Researchers (Tsampras et al. 2016a; 2016b; 2017; 2018; Zhang et al. 2018) developed force-limiting deformable connections that consist of two components: (1) a limited-strength hysteretic component to limit the forces transferred through the connections, and (2) a bearing component to provide post-elastic stiffness to prevent excessive inelastic deformation in the connections and provide the out-of-plane stability to the seismic force-resisting system composed of planar structural elements. Figure 3 shows a schematic three-dimensional view, plan view, and connections between the gravity load-resisting system and the seismic force-resisting system of a RC rocking wall building with force-limiting connections. As observed in the plan view, including these force-limiting connections requires a gap between the slabs and the rocking walls (made through openings in the floor system) to allow relative motion between the two connected systems. These connections are designed to eliminate the damage expected due to the stress concentration in conventional monolithic floor-to-wall joints generated to provide kinematic compatibility between the connected structural components. The building also includes limited-strength hysteretic components in the rocking interface at the base.

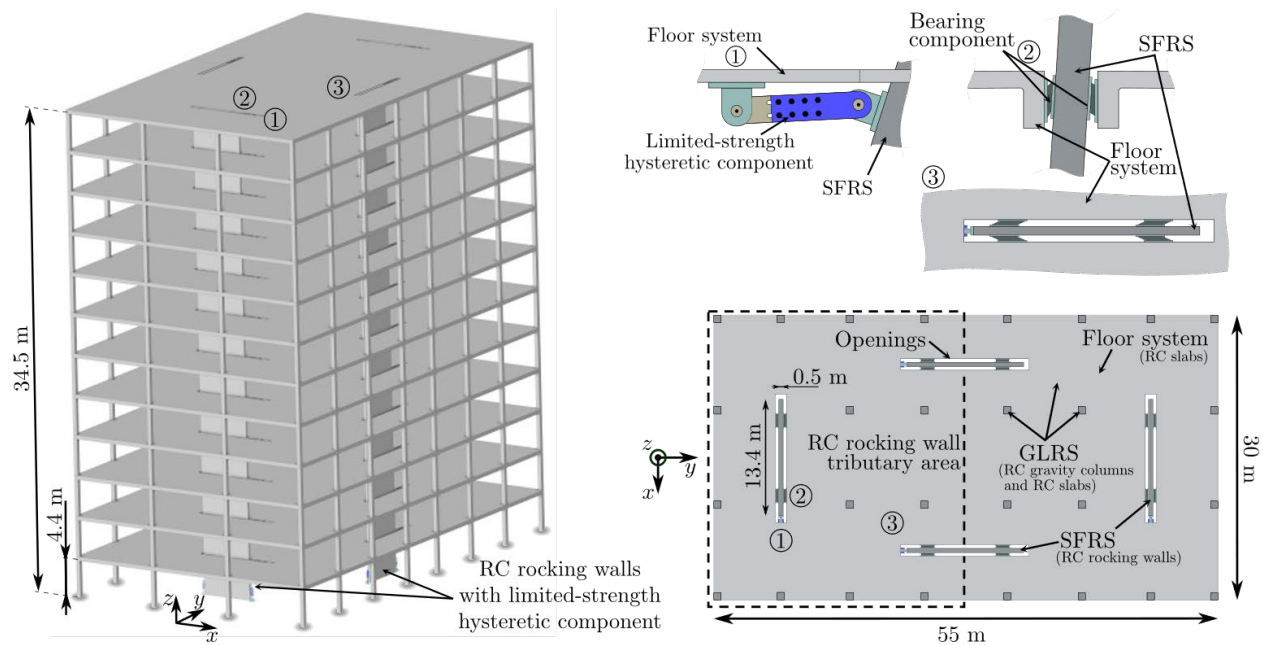


Figure 3. RC rocking wall building with force-limiting connections.

Tsampras and Sause (2022) proposed a force-based design method for the force-limiting connections. This method estimates the earthquake-induced inertial forces at the building floors and applies a reduction factor R_{DC} to control the ductility demand on the connections. However, this method is only for the connections. It does not consider the design of the building as a whole system and does not include performance- or resilience-based design objectives. Tsampras et al. (2016; 2022) and Lee et al. (2024) applied the force-based design method to RC wall building, self-centering rocking braced-frame building, and RC wall core building numerical models with force-limiting connections. They observed a reduction in the higher-mode shear-dominant acceleration and force responses as well as a reduction in their dispersion due to ground motion-to-ground motion variability. They also demonstrated the remarkable ability of these building systems of changing R_{DC} used to design the force-limiting connections to intentionally modify the global seismic response and the distribution of the ductility demand on the building numerical models. Mayorga and Tsampras (2024) observed a reduction in the dispersion of the distribution of the mean peak shear-dominant acceleration and force responses that may be developed due to the uncertainty in the material properties of the longitudinal reinforcing steel bars of RC wall-building numerical models with force-limiting connections. The ability to reduce this variability enhances the confidence in the numerical seismic response prediction of these building systems and, as a result, enhances the confidence in the development of future methods to design buildings with force-limiting connections considering resilience-based design objectives.

The use of force-limiting connections is a practical design solution for buildings with accelerated post-earthquake functional recovery, which contributes to national earthquake resilience. Nevertheless, there is currently no simple and practical design method available that provides the means for the holistic design of buildings with force-limiting connections to achieve targeted performance and post-earthquake functional recovery levels.

NIST/NSF Disaster Resilience Research Program Funded Project

In 2024, the authors initiated a research project that focuses on the design of buildings with a rocking inelastic base mechanism equipped with force-limiting connections strategically placed in the seismic load path. The project was funded by the NIST/NSF Disaster Resilience Research Program (Award No. 70NANB24H013). This building system is preferred over conventional designs due to their potential to achieve faster and more reliable post-earthquake functional recovery, owing to their limited seismic responses and reduced sensitivity to variability in strong earthquake ground motions. Additionally, it is expected that including force-limiting connections provides a solution for the trade-off between the seismic responses associated with structural damage and those associated with damage to non-structural components present in the design of conventional buildings aiming for faster post-earthquake functional recovery.

Figure 4 shows a schematic representation of the vision and goal of the project. The vision of the project is to advance the fundamental knowledge that will democratize the simple, efficient, rational, and practical design of high-performance buildings that even after earthquakes preserve the functions that are critical for the well-being of our society and reduce the immediate and long-term financial losses. The project aims to develop and validate a design method that will integrate performance and resilience criteria. It is expected that the proposed design method will popularize high-performance earthquake-resilient building systems designed to meet specific performance and resilience criteria.

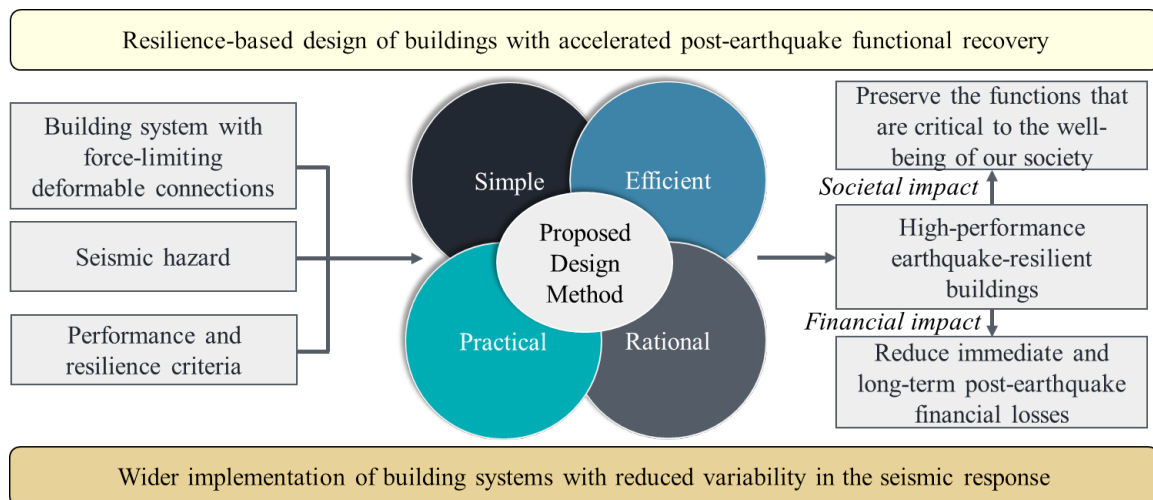


Figure 4 Vision and goal of the proposed project.

The scope of the project is to develop a procedure that will essentially integrate performance and resilience objectives. To do that, the seismic response modification factors for the force-limiting connections and seismic force-resisting system components along with the equivalent lateral forces that will be used for the proposed design method are going to be determined. The overstrength factors needed to apply the capacity design principles to protect the structural elements not designed for excessive inelastic deformation are going to be determined. Additionally, the expected inelastic demands in the force-limiting connections and seismic force-resisting system components are going to be estimated. Finally, the structural performance limits are going to be linked to functional recovery limits for a given level of hazard considering a probabilistic approach.

Conclusions

An introduction to the NIST/NSF-funded research project that focuses on the development of a resilience-based design method for buildings with accelerated post-earthquake functional recovery was provided. This research project is motivated by the critical need to reevaluate traditional seismic design approaches due to the fatalities, financial losses, non-functional communities, and significant environmental impacts due to debris after strong earthquakes. The development of a design method for buildings with accelerated post-earthquake functional recovery can contribute to the national earthquake resilience by reducing downtime and economic losses following seismic events.

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CONVERSION PROJECT FROM EXISTING SUPERMARKET TO TOWN HALL

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Tokyo/Japan

Abstract

The project involves the conversion and revitalization of the Tarui Town Hall in Gifu Prefecture. The old town hall, built in 1966, was deteriorating and required a new building to serve as a disaster prevention center in preparation for the anticipated Tokai earthquake. The project explored various options, including reconstruction and relocation, but ultimately decided to repurpose a closed commercial facility due to its sound condition, strategic location along a main road, and proximity to a cultural center, enhancing daily activity and disaster response coordination. This conversion is a rare challenge in Japan, driven by Tarui Town's strong commitment to sustainability and decarbonization, aiming to set a precedent for future government office projects.

To improve seismic performance, the project included four structural methods:

1. Reducing building volume and live loads.
2. Creating voids called "Environmental Wells."
3. Installing additional outframe structures.
4. Constructing an annex for heavy load use.

Despite challenges, the project aimed to reduce CO₂ emissions by 62% compared to rebuilding, demonstrating that high-quality, low-cost, and sustainable government offices can be achieved through conversions.

Introduction

Carbon neutrality is a common global challenge, and numerous efforts are being made in the field of architecture as well. Recently, there has been active development in timber structures, with large-scale wooden buildings being constructed worldwide. However, it is evident that, beyond simply transitioning to timber construction, extending the lifespan of buildings plays a crucial role in reducing CO₂ emissions. In general, the lifespan of buildings in Japan is shorter compared to those in Western countries, with residential buildings lasting approximately 20–30 years and other buildings around 60 years. Factors contributing to the lifespan of a building include not only physical durability due to material degradation but also functional and social lifespan considerations. Functional lifespan refers to the point at which a building no longer meets current needs due to equipment deterioration, while social lifespan pertains to the loss of utility of a building as the surrounding environment changes.

In recent years, there has been a growing trend of adaptive reuse or conversion, whereby buildings that have reached their functional or social lifespan are repurposed into new uses without demolition or replacement, utilizing the existing structural frame. Important approaches to extending building longevity include¹⁾:

- Prevention of material degradation
- Enhancement of spatial adaptability
- Increased resilience of the building structure to external disturbances
- Aesthetic improvements

This paper introduces examples where buildings, having reached their social lifespan, have been repurposed through a change in use.

Project Overview

This project involves the revitalization of a government office building through conversion in Tarui Town, Gifu Prefecture. As the former town hall, built in 1966, continued to deteriorate, there was an urgent need to establish a new government building that could serve as a disaster prevention hub capable of withstanding the anticipated Nankai Trough Earthquake. As part of the new town hall development project, various possibilities, including reconstruction, renovation, and relocation, were explored. Among these, a plan emerged to repurpose a closed supermarket through conversion. The decision was driven by the existing building's location along a main road and its proximity to a cultural center, allowing for potential daily activity and strong connections in case of disaster.

Japan's seismic standards were significantly revised in 1981, establishing the current seismic code. Although the existing supermarket, built in 1986, met the current seismic standards, it was required to increase its seismic safety by 1.5 times to ensure its function as a disaster prevention center. This was achieved by installing an outer frame of cast-in-place concrete columns and beams spaced about three meters apart around the perimeter. This outer frame functions as an environmental feature to mitigate sunlight and as a walkway to improve accessibility with adjacent buildings. By rhythmically continuing along the facade, it revitalizes the former commercial facility's appearance and creates a dignified exterior suitable for a government building.



Figure 1. Building Exterior (Left: Before Conversion, Right: After Conversion)

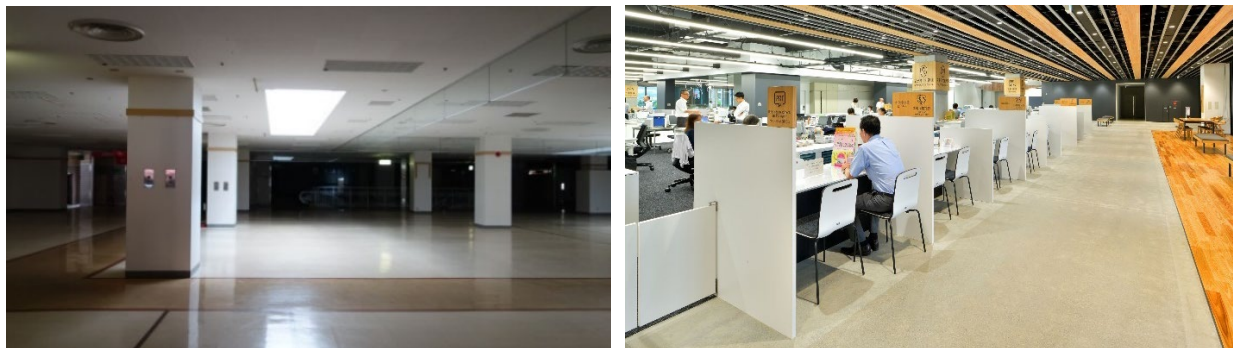


Figure 2. Building Interior (Left: Before Conversion, Right: After Conversion)

To improve the enclosed interior environment of the former commercial facility, four atrium spaces called "Environmental Wells" with skylights were introduced to promote natural lighting and ventilation. Beneath these warm, welcoming spaces featuring locally sourced wood in the interior, areas were designated for interaction and relaxation.

On the south side along the national highway, a new two-story steel-frame hall building, structurally independent from the existing building, was added. The first floor houses the "Tarui Hall," a multi-purpose space intended for events and various uses. Given the proximity of a river to the site, essential

equipment needed for the disaster response hub was consolidated on the second floor to prepare for potential flooding.

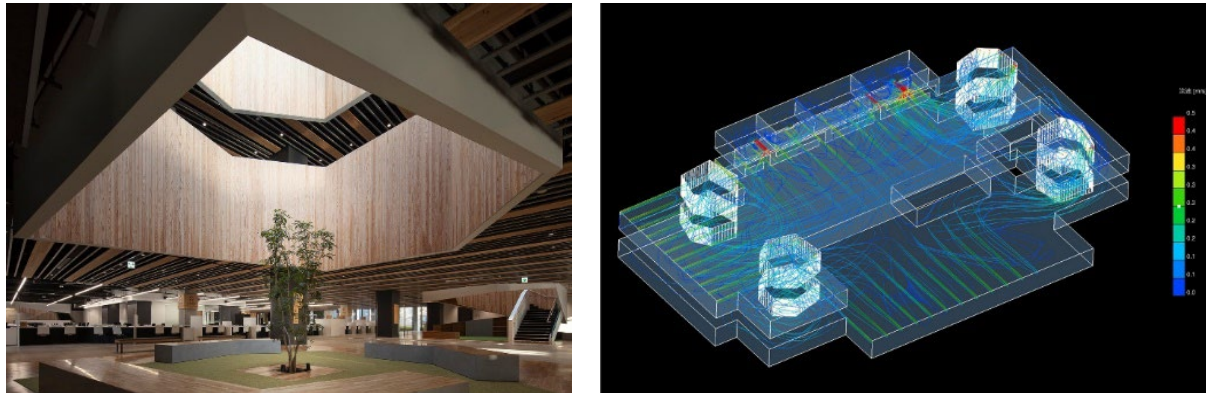


Figure 3. Airflow simulation of "Environmental Wells".

Existing Building Survey

The existing commercial facility targeted in this project is a two-story reinforced concrete supermarket built in 1988, with a total floor area of 7,150 square meters. Having been in place for 30 years, the building had reached a stage of functional and social obsolescence, despite reinforced concrete previously being considered to have a lifespan of 60 years.

For this conversion, a preliminary survey was conducted to assess the building's deterioration before design work began, which included measurements of member cross-sections and material testing. Core samples of concrete were extracted from the existing building to confirm the concrete strength and the depth of carbonation.

The concrete strength test results showed an average strength of 290.7 kgf/cm², exceeding the design strength of 210 kgf/cm². The carbonation depth ranged from 0.0 mm to 26.4 mm on average.

Table 1. Concrete compressive strength test results

Floor	Test piece No.	Apparent density	Corrected compressive strength	
		g/cm^3	N/mm^2	kgf/cm^2
1st Floor	1-W-1	2.25	22.1	225
	1-W-2	2.26	32.5	331
	1-C-1	2.20	17.6	179
	Average	-	24.1	245.0
	Standard deviation	-	7.6	77.9
2 nd Floor	2-W-1	2.23	29.7	303
	2-W-2	2.26	34.3	350
	2-C-1	2.28	34.9	356
	Average	-	33.0	336.3
	Standard deviation	-	2.8	29.0
Average		-	28.5	290.7
Standard deviation		-	7.1	72.6

Table 2. Average value of carbonation depth

<i>Floor</i>	<i>Test piece No.</i>	<i>Room</i>	<i>Finishing condition</i>	<i>Average value of carbonation depth (mm)</i>
1st Floor	1-W-1	Sales floor	Mortar(25mm)+Wallpaper	2.7
	1-W-2	Sales floor	Exposed concrete	15.2
	1-C-1	Sales floor	Mortar(25mm)+Wallpaper	1.9
2 nd Floor	2-W-1	Sales floor	Mortar(20mm)+Wallpaper	17.2
	2-W-2	Sales floor	Exposed concrete	26.4
	2-C-1	Sales floor	Mortar(25mm)+Wallpaper	0.0

The progression of carbonation²⁾ can be estimated by Equation (1):

$$C = A \times \sqrt{t} \quad (1)$$

where C is the average carbonation depth of the concrete (mm), A is the carbonation rate coefficient ($\frac{\text{mm}}{\sqrt{\text{year}}}$), and t is the age of the material(year).

The carbonation rate coefficient was calculated from the measured carbonation depth and material age of the building. Using this value, the age at which the carbonation depth would reach the rebar cover thickness of 40 mm was estimated, indicating a building lifespan of approximately 68 years due to carbonation. It is expected that the building will remain serviceable for another 40 years.

Thus, despite being over 30 years old, the building shows no deterioration affecting structural strength, confirming its suitability for the new town hall plan.

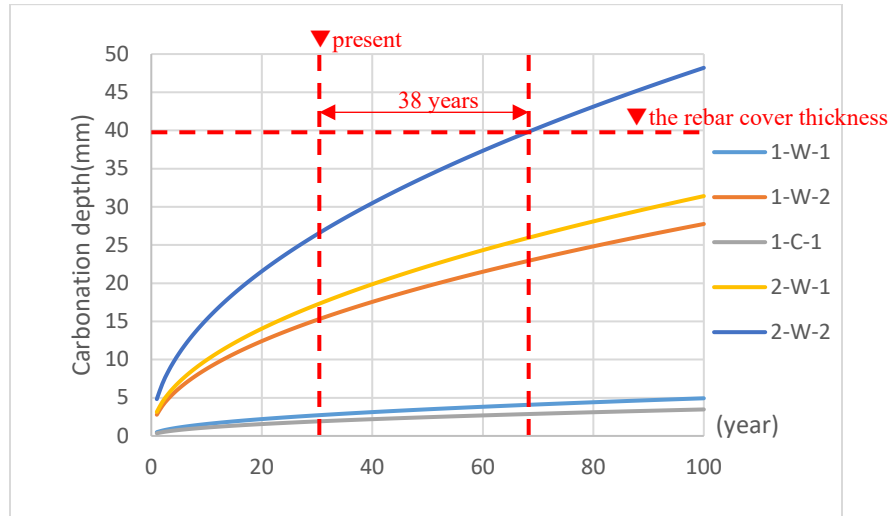


Figure 4. Carbonation Progress Prediction and Estimated Service Life.

Enhancement of Seismic Performance as a Disaster Prevention Center

The existing building was designed using Seismic Design Route 2-3 for reinforced concrete, which is a calculation route intended for structures planned with a focus on ductility rather than strength. As a result, the existing columns and beams were sufficiently reinforced against shear. However, due to the building's original function as a commercial facility, the walls, primarily located along the outer perimeter, had

numerous openings. This layout contributed to potential brittle failure in the columns and beams, resulting in a frame that lacked adequate deformation capacity.

For the current design, aimed at establishing the new town hall as a disaster response hub, it was necessary to enhance the seismic performance by increasing the building's lateral load-carrying capacity to 1.5 times the required lateral load-carrying capacity, as specified in the standards for government buildings.

To improve seismic performance, the project included four structural methods:

1. Reducing building volume and live loads.
2. Creating voids called "Environmental Wells."
3. Installing additional outframe structures.
4. Constructing an annex for heavy load use.

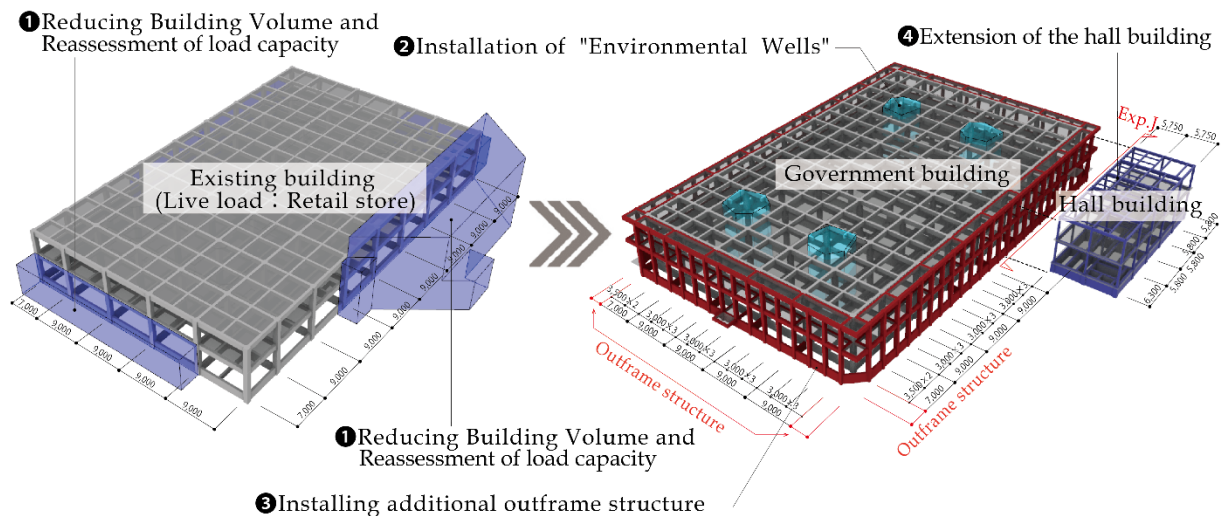


Figure 5. Structural Methods for Seismic Performance Enhancement

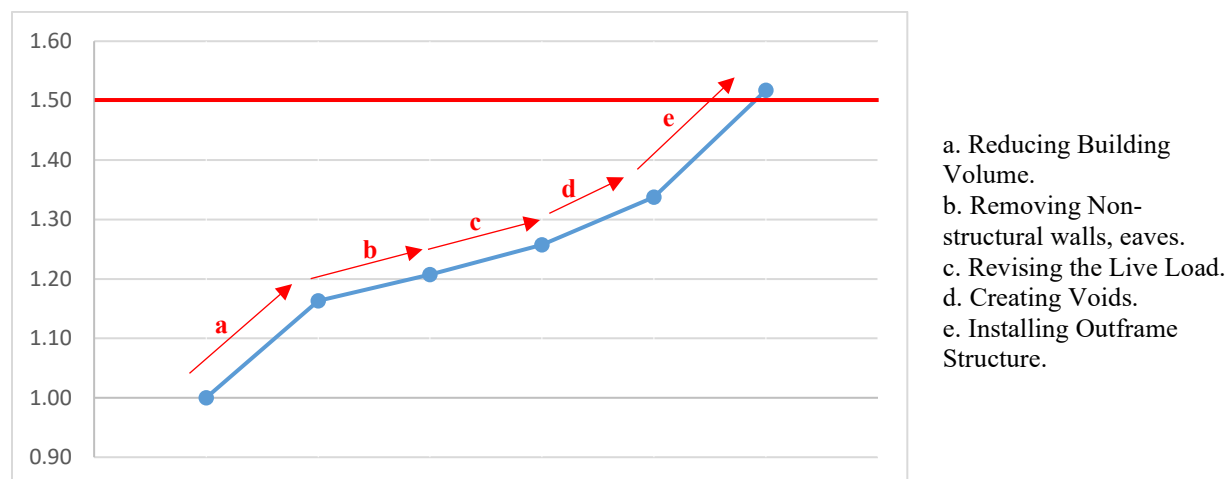


Figure 6. Percentage Increase in Seismic Performance through Structural Methods.

Reducing Building Volume and Live Loads. To enhance seismic performance, the first step was to reduce the building's weight. This was achieved by revising the load requirements from a commercial facility (seismic load of 1300 N/m²) to an office use (seismic load of 800 N/m²). In addition, non-structural walls, eaves, and the atrium area in the entrance, which did not significantly contribute to

structural strength, were removed, resulting in a 25% reduction in building weight. Furthermore, walls such as spandrel walls and parapets, which contributed to brittle failure of columns and beams, were also removed, bringing the structure to a skeletal state. This reduction in weight, combined with improvements in the frame's deformation capacity, effectively enhanced the building's lateral load-carrying capacity. In reducing the existing building to its skeletal state, care was taken to avoid imparting shocks to the existing structural frame. The disk sawing method was adopted for demolition, enabling careful block-by-block dismantling.



Figure 7. Demolition Process Using Disk Sawing Method.

Creating Voids Called "Environmental Wells". By introducing four atrium spaces called "Environmental Wells," the floor area was reduced, achieving an approximate 8% decrease in building weight. The shape of each environmental well is unique. During the removal of the floor slab and secondary beams, structural safety was ensured by carefully addressing potential rebar anchorage deficiencies in adjacent members and changes in boundary conditions.

Installing Additional Outframe Structures. To ensure openness and flexibility in the interior space, no structural reinforcements were added internally; instead, an outer frame was installed along the perimeter to improve seismic resistance. While the existing building had columns arranged in a 9.0m x 9.0m grid, the outer frame columns were added along the building's perimeter at 3.0m intervals. This outer frame was designed to absorb approximately 20% of the seismic forces acting on the building, providing the necessary rigidity and strength to enhance the overall structural capacity.

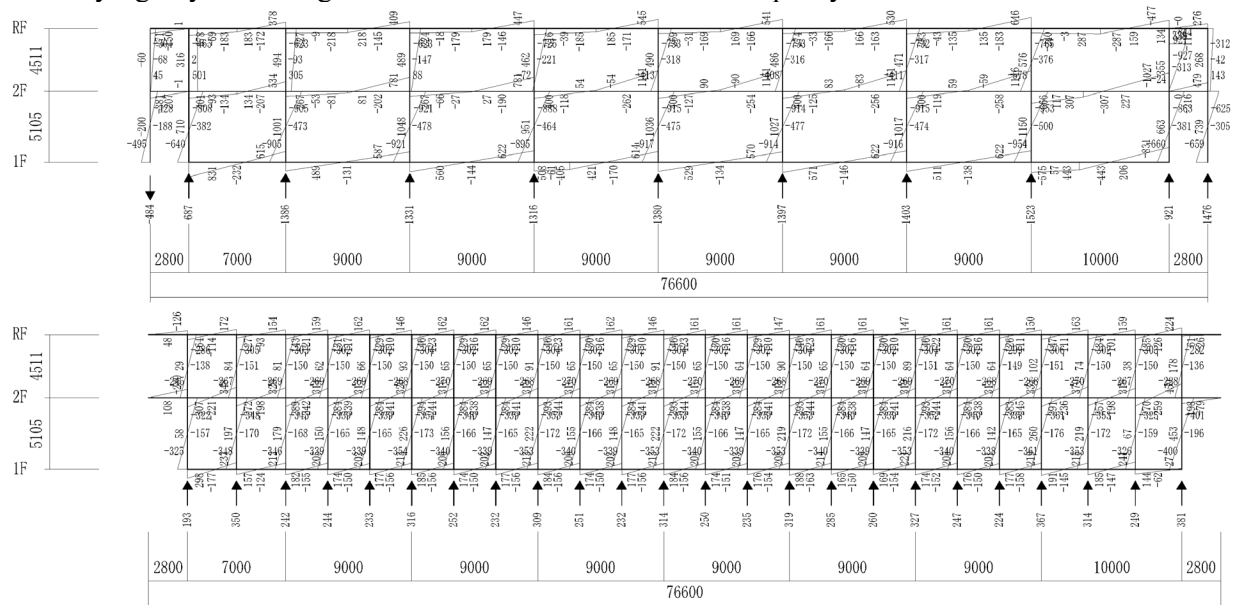


Figure 8. Stress Diagram (Top: Existing Y2 Frame, Bottom: New Y1 Outer Frame)

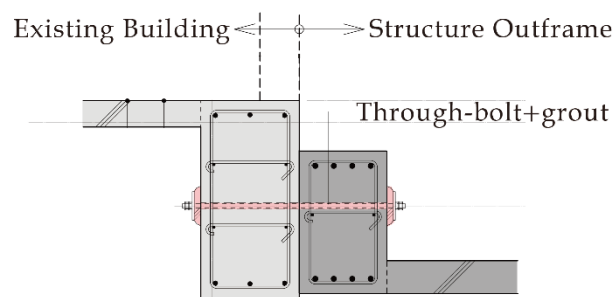
Table 3. Lateral Load-carrying Capacity for Each Frame(under X-direction Loading)

<i>Frame</i>		<i>Lateral Load-carrying Capacity</i>	
		<i>kN</i>	<i>%</i>
Outframe	Y8	4041	11%
Existing Frame	Y7	4359	78%
	Y6	4716	
	Y5	4783	
	Y4	4721	
	Y3	4761	
	Y2	4597	
Outframe	Y1	4044	11%
SUM		36022	100%

In Japanese standards, post-installed anchors are permitted only for seismic reinforcement purposes. Since the addition of the outer frame was classified as an extension under regulatory standards, it was not possible to use post-installed anchors to connect the outer frame to the existing structure, necessitating a review of the connection method. In this plan, the existing structure was drilled to accommodate through-bolts, integrating it structurally with the outer frame. The connection points between the existing structure and the outer frame were designed to transmit seismic forces. The transfer of lateral forces was achieved through the shear strength of the through-bolts and the bearing of the concrete, with the number of through-bolts calculated based on the transmitted lateral force.

Table 4. Calculation of the number of through-bolts

<i>Loading Direction</i>	<i>Floor</i>	<i>transmitted lateral force.</i>	<i>Through Bolt diameter</i>	<i>The number of through-bolts</i>
X	2 nd Floor	1675	M22	64
	1 st Floor	2886	M27	64
Y	2 nd Floor	1675	M27	40
	1 st Floor	2886	M33	40

*Figure 9. Connection detail of the outer frame to the existing structure.*

Constructing an Annex for Heavy Load Use. To reduce the load on the existing building, essential equipment required for the disaster response hub was installed in a structurally separate, newly constructed hall building. By consolidating heavy equipment in the new hall structure, the load on the town hall building was minimized, allowing for reinforcement measures to be kept to a minimum.

Conclusion

In a conversion project, careful reduction of the existing structure is crucial. Before developing the reinforcement plan, a thorough analysis of the existing building was conducted to determine where reductions could be made, maximizing the use of the existing frame.

During the construction phase, many discrepancies were found between the design-stage visual inspections and measurements and the actual locations of partitions and beams. Many structural details became apparent only after removing ceiling materials, necessitating a meticulous process of demolition, measurement, verification, and construction. This iterative workflow ensured precision and adaptability throughout the project.



Figure 10. Rebar Location Scanning and Drilling Process During Construction.

Finally, by repurposing the existing building through conversion, CO₂ emissions were reduced by 62% compared to reconstructing a similar type of building.

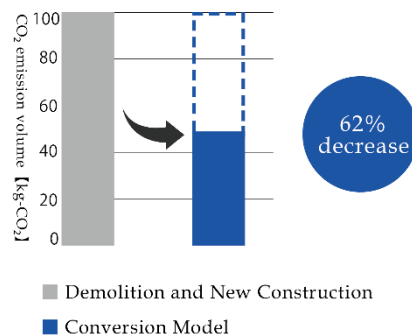


Figure 11. A sample figure. Comparison of CO₂ emissions

This project exemplifies a method for extending building lifespan through conversion. Conversion-based development of government buildings remains rare nationwide and represents a significant challenge. However, in the context of advancing decarbonization and a sustainable society, this approach carries substantial social significance. Driven by Tarui Town's strong commitment, this project demonstrates a model for government building development through conversion, setting an example for society.

References

- 1) Architecture Institute of Japan, 2024, "Building Structure for Global Environment"
- 2) Architecture Institute of Japan, 2016, "Recommendations for Durability Design and Construction Practice of Reinforced Concrete Buildings"

STRUCTURAL DESIGN OF HIGH-RISE BUILDING AIMING FOR HIGHER BUSINESS CONTINUITY PERFORMANCE

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Abstract

This paper presents structural design of a 165-meter high-rise office building that aims to have the higher business continuity performance after a major earthquake. In order to meet this objective, this building has strict design criteria, including the response acceleration of each floor. For this reason, various technologies have been introduced into the building. mainly core walls of spine structure that are combined with outrigger beams, and seismic isolation systems. The core wall has a H-shaped planar shape, and its structural characteristics were confirmed through experiments. The analytical model of core wall used in the design was a fiber model, and it was confirmed that it corresponded well with the experimental results. The outriggers are composed of RC-S composite beams and belt trusses installed on the 7th floor. And it was confirmed that these were effective in suppressing deformation of the upper structure and reducing the surface pressure of the seismic isolation bearings during earthquakes. Isolation system has two unique types of oil dampers, and it showed reducing response acceleration of each floor and addressing large earthquakes that exceed extremely rare earthquakes. In addition, because it is also important to evaluate the stability of the seismic isolation system against strong wind, wind tunnel tests were carried out to evaluate the wind forces. During this test, a study was also conducted on the building shape to reduce wind forces, and the results were reflected to the design.

Introduction

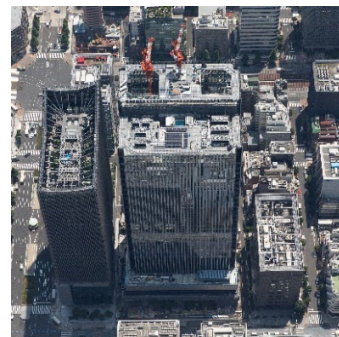
TODA BUILDING is a high-rise building located near Tokyo Station with 28 floors above ground and three floors underground, approximately 165 m in height, and a total floor area of approximately 95 000 m² (Figure 1) and opened in November 2024. It locates in Tokyo, close to the Sagami Trough. The probability of a magnitude 7 class earthquake occurring along the Sagami Trough within the next 30 years is estimated to be 70% by the Headquarters for Earthquake Research Promotion Japan. This building has been approved as “special district for urban regeneration”, and one of the major themes is "strengthening disaster prevention capabilities" by Tokyo Metropolitan Government. And it was aimed for high business continuity performance, preventing even internal damage after a major earthquake. It is one of the largest seismic isolation buildings in Japan. This paper reports one of the structural solutions for high-rise buildings with excellent building continuity performance.



(a) Exterior of building



(b) Construction status photo(Nov. 2023)



(c) Construction status photo(Jul. 2024)

Figure 1. Overview of TODA BUILDING

Structural Design

Figure 2 shows structural system of TODA BUILDING. The characteristic points of this structure are described below.

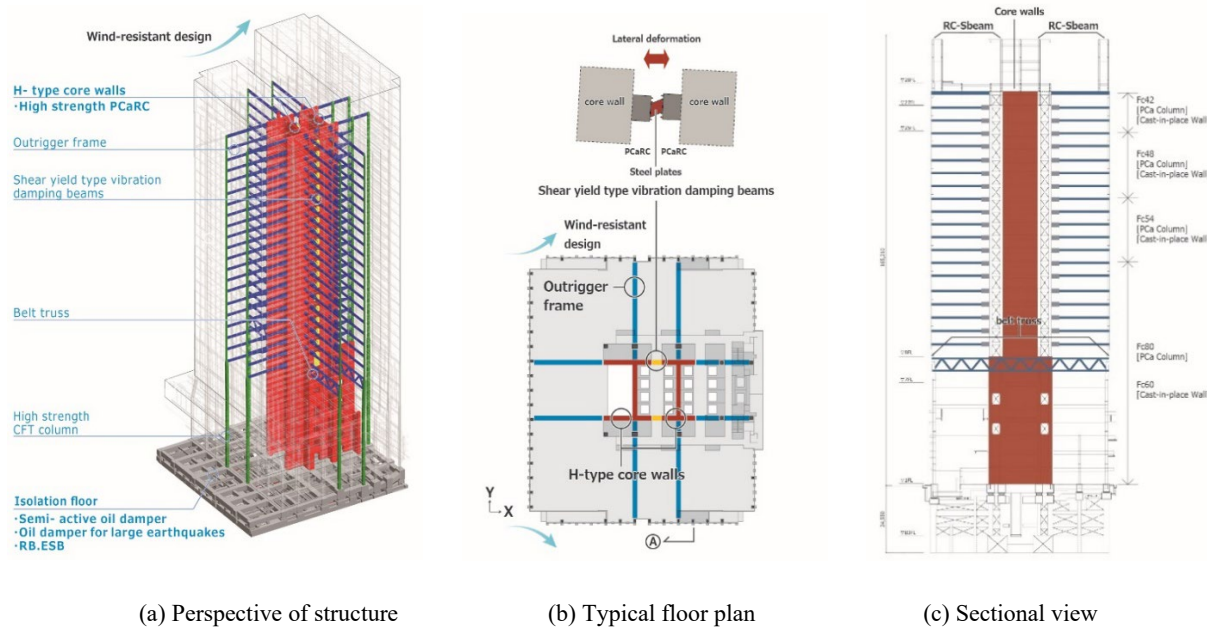


Figure 2. Structural System of TODA BUILDING

Core Walls. This building has two H-type core walls of spine structure at the center of floor plan, which use elevator shaft walls, and they are combined with outrigger frame. These are consisted of high-strength precast reinforced concrete (PCaRC) and cast-in-place concrete. Their specified design strength of concrete is from 42N/mm^2 through 80N/mm^2 , and yield strength of main reinforced bar is from 490N/mm^2 through 685N/mm^2 . Core walls are located from 1st floor through 27th floor. Figure 3a shows detailed drawing of bar arrangement at the joint position of PCaRC and cast-in-place wall. For the lower floors from the 1st to the 7th floors, two H-type core walls are integrated to form a larger core wall, enabling it to resist large overturning moment. Its structural performance for design was verified by structural experimental study that was carried out by 1/6 scale H-type core wall specimen^[1] (Figure 3b). The relationship between load and deformation, the ultimate state of the specimen, and the condition of the joints between the precast and cast-in-place sections were confirmed.

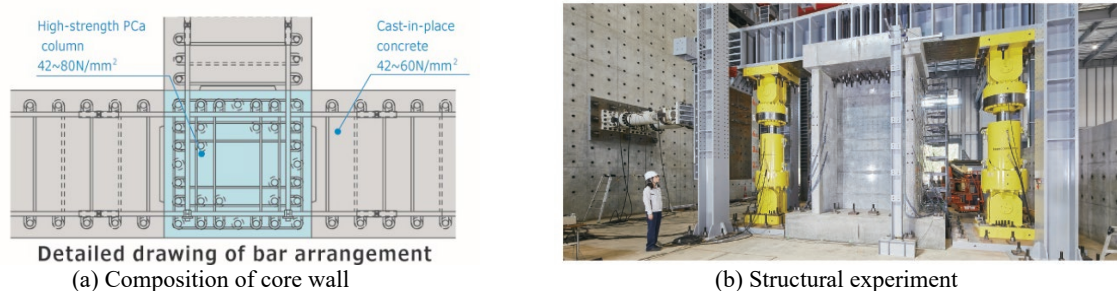


Figure 3. Core Wall of This Project

Figure 4a shows the overview of Fiber model of H-type core wall. The elements of fiber are consisted of confined concrete of PCaRC, non-confined concrete of PCaRC, non-confined concrete of cast-in-place wall and reinforcement bar. Constitutive rule of reinforcement bar is bi-linear model with yield point as break point. The New RC formula is used as the constitutive rule for confined concrete, while the Shah formula is used as the constitutive rule for non-confined concrete. Figure 4b shows the result of relationship between bending moment and rotation angle. The experimental results and analytical results demonstrate good agreement.

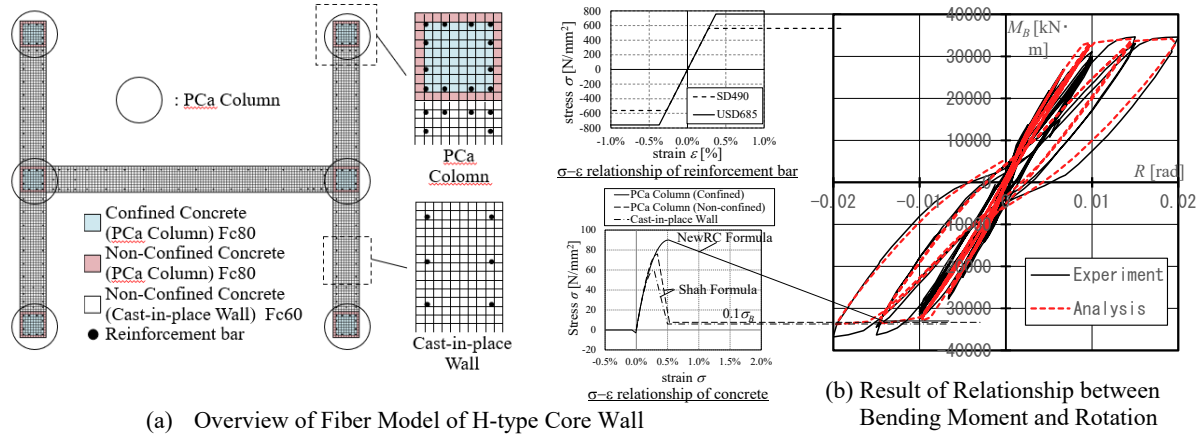


Figure 4. Analysis of Core Wall

Outrigger Frame. The outrigger frame is combined with core walls to take advantage of it for lateral load resisting and reducing the surface pressure of the seismic isolation bearings during earthquakes. The outrigger beams of typical floors are RC-S composite beams (Figure 5a) [2], which arrange steel and RC members in series due to smoothly transmit force from core walls to external steel frames. Since larger bending forces occur in the direction of strong axis (Y direction shown in Figure 2b) of the H-type Core walls, truss-type outrigger beams called “Belt truss” (Figure 5b) at the seventh floor. Buckling restrained braces are incorporated into these trusses for restrict input force when subjected to forces larger than it is considered. The seventh floor is designated as the machine room floor, so there were few planning restrictions regarding the placement of the huge trusses. The outrigger beams are connected to high-strength CFTs (Concrete Filled Tubes). These CFTs consist of steel tubes with a tensile strength of 780 N/mm² and concrete with a maximum specified design strength of 150 N/mm². The outriggers effectively suppressed the horizontal deformation of the building and the surface pressure on the seismic isolation bearings. In numerical analysis of seismic design, compared to a model without the outriggers, the horizontal deformation of the building was reduced by approximately 25 %, the maximum compressive surface pressure of the seismic isolation device was reduced by 7 %, and the maximum tensile surface pressure was reduced by 18 %.

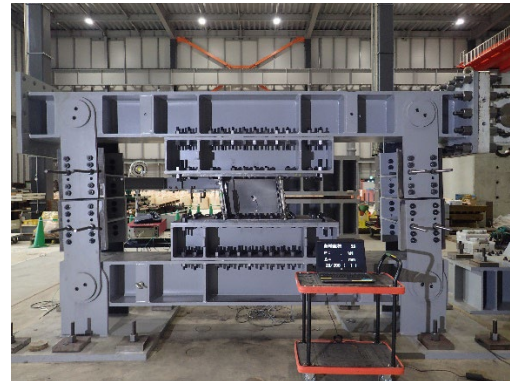


Figure 5. OutriggerFrame of This Project

Damping Beams. Between the H-shaped core walls, vibration control beams, called "shear yield type vibration damping beams" are installed. They consist of steel at midsection and RC at beam ends (Figure 6a). They are designed to reduce damage of beams and core walls by using shear deformation caused by vertical deformation differences and absorbing seismic energy. Steel plates of the damping beams utilize low-yield-point steel, with a yielding stress of 225 N/mm^2 and maximum thickness of 25 mm. The durability of steel plates subjected to high cyclic loading is verified by a full-scale experimental study (Figure 6b) [3]. Dynamic response analysis of this building, described below, confirmed that damping beams were able to absorb approximately 10 % of the input seismic energy during large earthquakes.



(a) Damping beams between core walls



(b) Cyclic loading test of steel plates

Figure 6. Damping Beams of This Project

Isolation systems. The isolation floor is located below the first floor, and it has unique isolators and oil dampers to get achieve better performance across a wide range of earthquakes, from small to large. Figure 7 shows placement of seismic isolation devices on the isolation floor. Two types of bearings, rubber bearing (RB) and elastic sliding bearing (ESB) are installed on the seismic isolation floor. Maximum size of the RB of diameter is 1800 mm making it the largest size in Japan, manufactured by Bridgestone Corp. Two types of ESB are used: high friction type with a friction coefficient of 0.093 and a low friction type with a friction coefficient of 0.014. These isolation systems can achieve a longer natural period for the isolation floor and provide sufficient damping. In addition, two types of newly developed unique dampers, semi-active oil damper and oil damper for large earthquakes are installed.

Figure 8a shows the characteristics of the semi-active oil damper. It features two types of damping coefficients, higher-damping (CH) and lower-damping (CL) and changes its damping coefficient depending on the direction of increase and decrease in the response displacement of isolation floor. It is programmed to switch to CL in the direction of increasing deformation and acceleration, and to CH in the direction of increasing velocity, for effective acceleration reduction and energy absorption. Its maximum damping force is 1000 kN and the relief force is 800 kN. The damping coefficient of CH is $3750 \text{ kN} \cdot \text{s/m}$ and of CL is $1250 \text{ kN} \cdot \text{s/m}$. The maximum stroke is 650 mm. The damping coefficient is switched by referring to the measurements of displacement meters placed on the seismic isolation floor, which send electrical signals to semi-active oil damper [4].

Figure 8b shows a conceptual diagram of the oil damper for large earthquakes. It features a zone that is ineffective at absorbing seismic forces to avoid increasing response acceleration during small and middle-level earthquakes and becomes effective only during large earthquakes. The deformation at which damping begins (trigger deformation) is 150 mm. It features self-restoration to its original position using springs stored within the device and includes rubbers to absorb impact force when it reaches the trigger deformation [5]. Its maximum damping force is 949 kN and the relief force is 800 kN. The damping coefficient is $2500 \text{ kN} \cdot \text{s/m}$, and the maximum stroke is 700 mm. The elastic stiffness of the rubbers is 763 kN/mm .

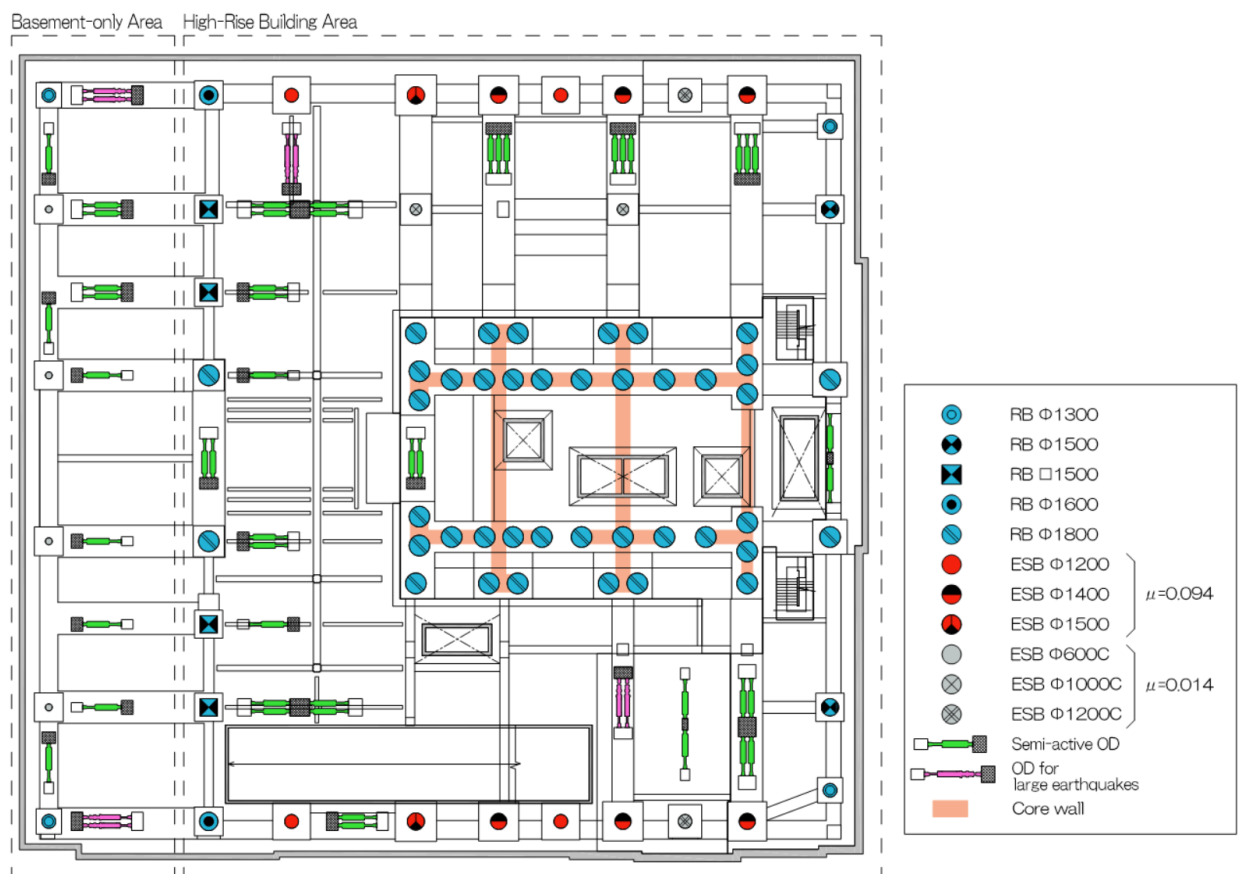


Figure 7. Placement of Seismic Isolation Devices

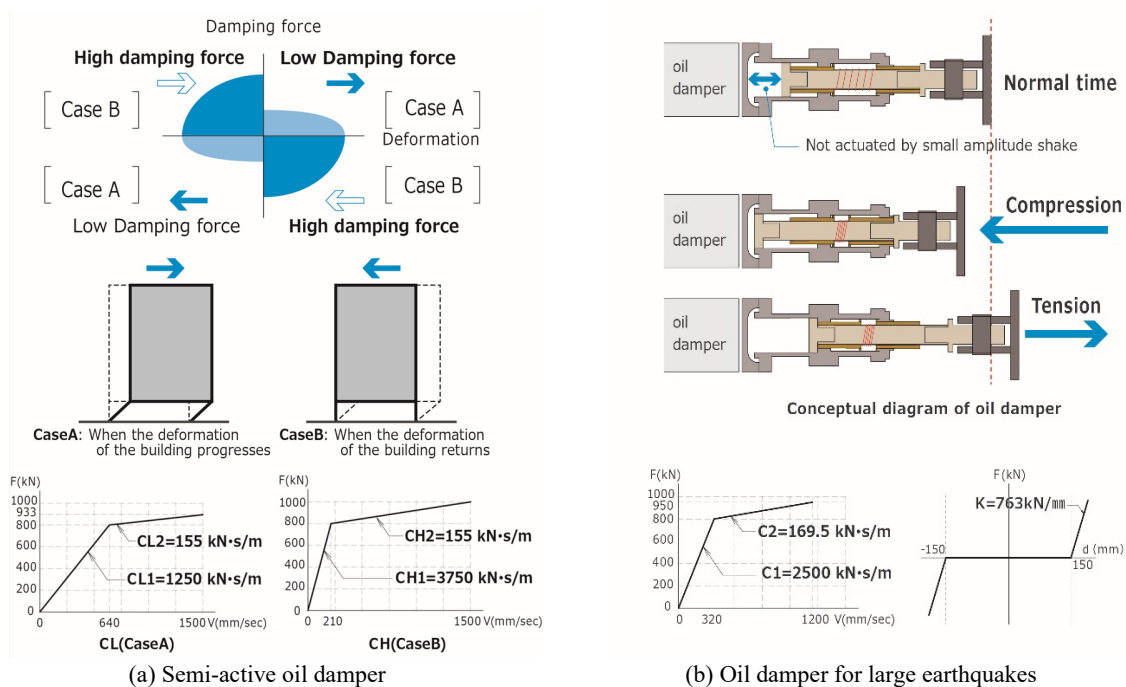


Figure 8. Characteristics of Installed Oil Dampers

Structural Performance

The criteria for this building aim to maintain complete functionality during Level 1 earthquake (with a return period of 50 years) and ensure the main functions during Level 2 earthquake (with a return period of 475 years). Table 1 shows the specifications of earthquakes for design. Figure 9 shows pseudo velocity response spectrum of these Level 2 earthquakes for design. This building's performance during earthquakes is verified through studies using a 3D full-frame numerical analysis model that includes the upper structure, isolation floor, and basement structure. SNAP (KOZO SYSTEM, Inc.) is used for the structural analysis. The core walls are modelled as a Fiber model in this full-frame model.

Table 1. Specifications of Earthquakes for Design

	Level 1			Level 2		
	Max.Acc. (mm/s ²)	Max.Vel. (mm/s)	Time (sec.)	Max.Acc. (mm/s ²)	Max.Vel. (mm/s)	Time (sec.)
El Centro NS (1940)	2554	250	53.8	5108	500	53.8
Taft EW (1952)	2483	250	54.4	4965	500	54.4
Hachinohe NS (1968)	1745	250	234.0	3490	500	234.0
CODE-KO ^{*1}	786	109	60.0	3930	547	60.0
CODE-HA ^{*1}	698	113	120.0	3490	563	120.0
CODE-RA ^{*1}	779	96	120.0	3897	479	120.0
KANTO ^{*2}	-----	-----	-----	3840	556	163.8
NK-KA1 ^{*3}	-----	-----	-----	455	213	655.4

*1: Stipulated by Building Standards Act of Japan

*2: Simulated seismic wave of this site

*3: Long-period ground motion

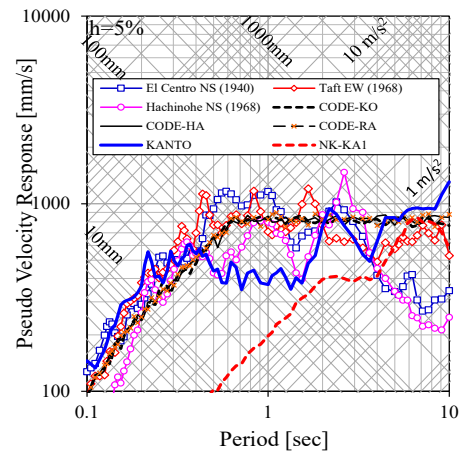


Figure 9. Pseudo Velocity Response Spectrum of Level 2 earthquakes

Design Criteria for Seismic Proof Design. Table 2 shows Criteria of structural design. These were set based on the highest-grade setting for seismically isolated buildings indicated by the JSCA (Japan Structural Consultants Association) ^[6].

Table 2. Criteria for Seismic Design

Contents	Seismic Inputs	
	Level 1	Level 2
Definition	Return period of 50 years	Return period of 475 years
Acceleration Response	-	Less than 2.0 m/s ² (at stories above isolation floor)
Maximum story drift angle	1/400	1/200
Members condition	Less than short-term allowable stress	
Safety factor for accumulative damage* of damping beams	More than 10	More than 2
Horizontal deformation of isolators	Less than 372 mm (150% against total rubber thickness)	Less than 620 mm (250% against total rubber thickness)
Surface pressure of isolators	Less than short-term allowable surface pressure (RB:30 N/mm ² , ESB:40 N/mm ²)	
Surface tensile strain of isolators	Non-allowable	RB: Within 5 % ESB: Non-allowable
Velocity of oil damper	Less than 1.5 m/s	

* "accumulative damage" is estimated as the Df value based on the Manson-Coffin's law and Miner's law ^[7].

Results of Numerical Analysis for Seismic Proof Design. The core walls bore approximately 80% of the horizontal forces in this building’s structure system. The natural period of the building including seismic isolation floor was approximately 5.6 sec when the isolation devices experienced horizontal deformation during level 2 earthquakes. In addition, the natural period of upper structure above the isolation floor was approximately 2.2 sec (1.3 % of building height in meters). It is considered that the core walls provide this building with sufficient rigidity compared to other typical steel-framed structures in Japan, and high acceleration reduction effect can be achieved by combining this with seismic isolation systems. Figure 10 shows results of dynamic analysis. The maximum story drift angle was approximately 1/350, which met the criterion with a fair margin. The maximum response acceleration was approximately 1.65 m/s², and it also met the criterion. Considerations were also made for a large earthquake 1.5 times larger than level 2 earthquakes. In this case, the maximum response acceleration was approximately 1.8 m/s². It has been reported that furniture can be prevented from falling over by keeping the floor response acceleration below 2.0 m/s², according to a reference [8]. The structural system of this building has a better performance for floor response acceleration, which prevents furniture from falling over on all the stories.

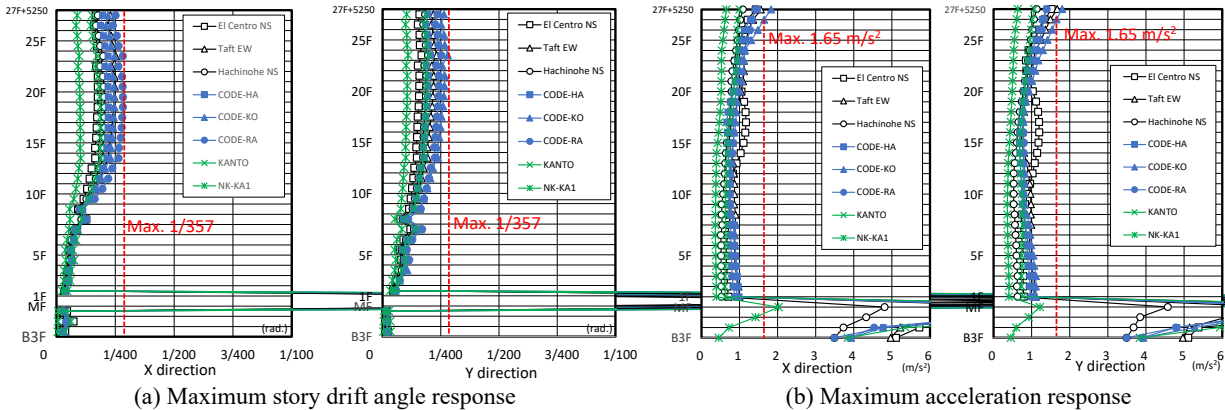


Figure 10. Results of Dynamic Analysis

Wind proof design for seismic isolation system. Ensuring stability under wind-resistant conditions is also an important issue for seismic isolated buildings. For this building, the criterion was set to ensure that the ESB would not slide even in a once in 500-year strong wind (Level 2 wind). Wind proof design for seismic isolation system was conducted through wind response analysis. The wind force used for the analysis was obtained through wind tunnel tests (Figure 11). Wind tunnel tests were carried out to compare different building shapes. The final shape of this building reduces wind force 25% less than normal rectangular shape (Figure 12).

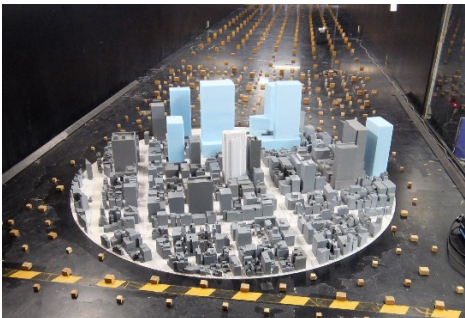


Figure 11. Model of Wind Tunnel Test

CASE	Final Shape	With corner cutting Without PH cutting	Without corner cutting With PH cutting	Rectangular shape
Model Photo				
Wind force reduction rate	0.75	0.85	0.90	1.00

Figure 12. Differences in Wind Forces due to Building Shape

Conclusions

This paper presented a structural system for a 165-meter high-rise building designed to achieve high business continuity performance by preventing internal damage even after a major earthquake. To achieve this goal, various technologies were applied, including H-shaped core walls, damping beams, and a seismic isolation system using two new types of oil dampers. Numerical analysis of the H-shaped core wall was shown to accurately evaluate experimental results. The combination of the core wall of spine structure that are combined with outrigger beams, and seismic isolation systems significantly reduces the lateral displacement and acceleration on each floor, meeting criteria with a fair margin. The proposed structural system was demonstrated to suppress the maximum response acceleration during a level 2 earthquake to 1.65 m/s^2 and to minimize damage, including the toppling of indoor furniture. Outrigger beams were demonstrated to be effective in suppressing the surface pressure on seismic isolation devices and serve as an important element in high-rise seismically isolated buildings. It was shown that the building's shape was designed to reduce external wind forces to prevent the seismic isolation layer from sliding out during a level 2 strong wind. The structural system presented here is considered one of the most effective methods for designing a high-rise building that aims to require no restorations after large earthquakes and to provide maximum safety for users against predicted massive earthquakes. However, further research is likely to be required for designing high-rise buildings over 200 meters.

Acknowledgements

The authors would like to express our sincere gratitude to Dr. Nobuyuki Izumi, former professor at Graduate school of Chiba University, for his valuable advice in the design of this building.

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NEW GUIDELINES FOR POST-EARTHQUAKE REPAIR BASED ON PERFORMANCE-CRITICAL DAMAGE

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Abstract

This paper introduces the contents of and basis for the forthcoming FEMA P-2335, *Guidelines for Post-Earthquake Repair and Retrofit of Buildings Based on Assessment of Performance-Critical Damage*. These Guidelines, which have been developed by a committee of practicing engineers and researchers, are intended to inform the assessment of buildings to identify the need for post-earthquake structural repair or retrofit. In doing so, the document aims to reduce uncertainty about repair and retrofit decision making in the post-earthquake environment. The Guidelines cover the identification of damage, the evaluation of observed damage in terms of its effects on the building's seismic performance, the determination of the necessity of repair and/or retrofit, and the identification of repair methods. The Guidelines adopt a new concept of performance-critical damage, which identifies damage to components and structures that, if occurring, causes a loss of strength or deformation capacity, jeopardizing seismic safety performance in future events. Performance-critical damage must be repaired. In this paper, the contents of the Guidelines are introduced, showing how experimental and analytical research and case studies were used to develop the recommendations therein.

Introduction

After an earthquake, owners and engineers face a fraught decision-making environment, characterized by substantial uncertainty about what buildings need to be repaired, retrofitted or demolished. This uncertainty can produce situations when buildings are not repaired but should be, endangering community safety. Conversely, this uncertainty can also lead to long delays and excessive demolition, which does not well serve goals of community resilience and recovery.

FEMA P-2335, *Guidelines for Post-Earthquake Repair and Retrofit of Buildings Based on Assessment of Performance-Critical Damage*, is intended to reduce this uncertainty by providing guidelines that can be used after an earthquake to identify buildings for which repair or retrofit is critical. It also summarizes information about repair strategies that can be used to achieve the necessary performance objectives for repair. In doing so, the Guidelines address a need to update existing earthquake repair guidance (e.g., FEMA 306, 307 and 308) to ensure that this guidance is consistent with modern building practices, engineering workflow, and performance expectations. At this time, the Guidelines provide provisions for reinforced concrete buildings, but the methodology is general and could be extended to other materials in the future.

Performance-critical damage. The central goal of the Guidelines is to identify damage that leads to a reduction in component and/or building strength and/or deformation capacity. This damage is referred to as “performance-critical damage”. Performance-critical damage corresponds to the point of initiation of

component strength loss (peak) on the force-deformation backbone of a component or building (Figure 1). If the structure is deformed beyond this point, strength and deformation capacity are degraded. Performance-critical damage is so-defined because, if it occurs, the future seismic performance of the building is impaired relative to the pre-earthquake condition, implying elevated collapse risk and amplified drift demands in future events. This damage therefore indicates the need for performance-critical repairs.

The project team defined the onset of component strength loss as performance-critical damage based on extensive background work (see also Opabola et. al 2024). Figure 2 shows a summary of evidence from shake table tests, compiled from work by a number of researchers (Shah 2021). In each case, the experimental specimens were tested multiple times using the same input motion (denoted “Run 1” and “Run 2”). The plot shows the data relative to a 1:1 line, indicating that, in general, under the same excitation, the drift demand in Run 2 was approximately the same as that in Run 1. As a result, Figure 2 demonstrates that under a broad number of conditions, damage does not lead to impaired performance in terms of drift demands. Indeed, the damage needs to be significant to lead to impaired performance. The results also indicated that an exception to this finding occurs when the amplitude of the second run is lower than that of the first, in which case stiffness reduction due to damage can be significant. However, these drift demands are not critical for collapse and life safety performance.

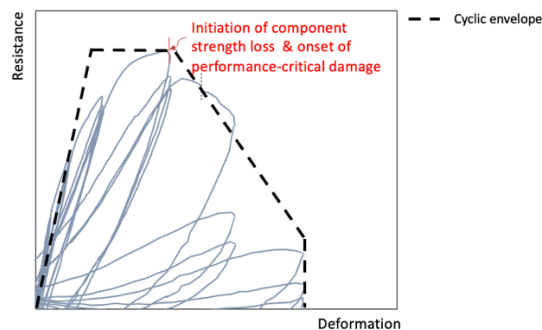


Figure 1. Illustration of component strength loss and onset of performance-critical damage. (Reproduced from FEMA P-2335)

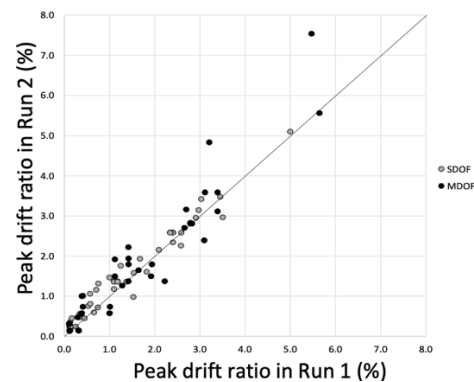


Figure 2. Compilation of shake table test data showing that prior damage does not increase drift demand, provided that there was no degradation in Run 1. (Reproduced from FEMA P-2335)

The project team further supported the concept and definition of performance-critical damage with analytical studies (e.g., Murray et al. 2022; Opabola et al. 2024; Safiey et al. 2022), illustrated in Figure 3. Figure 3(A) provides results for example single-degree-of-freedom systems with different plastic rotation capacities, θ_p , where the plastic rotation capacity defines the onset of component strength loss. These results show that performance (as measured by drift demand on the y-axis compared between the undamaged and damaged building) is not worsened until damaging motion peak drifts are large, and that those systems with lower deformation capacity have an earlier onset of performance-critical damage, due to the smaller plastic rotation capacity. Figure 3(B) provides results of analysis for a reinforced concrete special moment frame system. Performance is impaired (as indicated by y-axis values exceeding 1) when the damaging motion peak story drift is greater than about 2.5% drift. The onset of system strength loss for this building was around 3% story drift.

Overview of Guidelines. The intended use of FEMA P-2335, *Guidelines for Post-Earthquake Repair and Retrofit of Buildings Based on Assessment of Performance-Critical Damage*, is to identify the building

repair outcome, as outlined in Figure 4. This building repair outcome indicates whether the building requires performance-critical repairs or seismic retrofit in addition to performance-critical repairs.

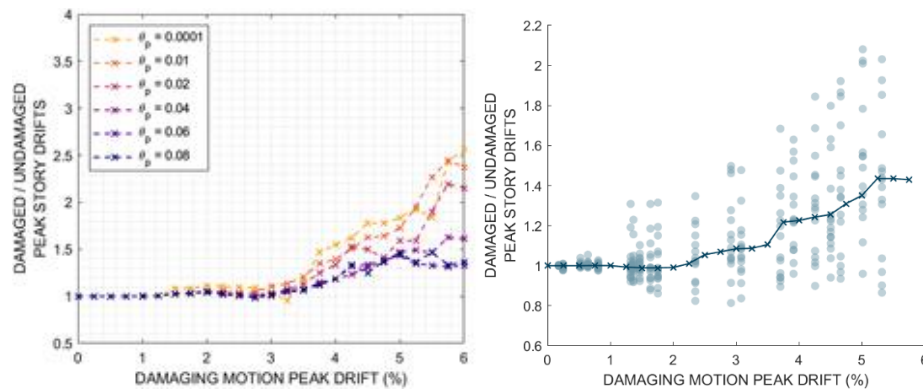


Figure 3. Analytical studies run by project team for (A) single degree of freedom and (B) RC special moment frame systems. These results show that unless the models are deformed past the point of performance-critical damage, earthquake performance is not worsened relative to its pre-damage condition. (See also Murray et al. 2022)

The Guidelines are organized with five chapters, appendices, and extensive electronic resources. Chapters 1-4 outline the principles of post-earthquake repair decision making which apply regardless of building material and system. Chapter 5 provides extensive qualitative and quantitative information defining criteria needed to apply these principles to reinforced concrete buildings. It is envisioned that subsequent editions will provide criteria like that shown in Chapter 5 for other materials and systems, including reinforced masonry and steel structures.

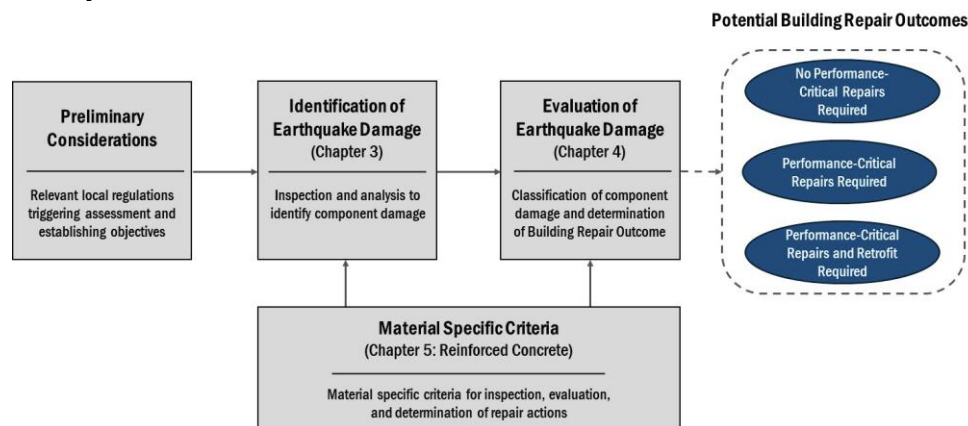


Figure 4. Overview of FEMA P-2335 Guidelines (Reproduced from FEMA P-2335).

The Guidelines do not establish policy related to post-earthquake evaluation, repair, or retrofit of buildings. Rather, they provide technical criteria that can be used to consistently implement policies in the context of both voluntary and triggered mitigation. Critical for U.S. application, the Guidelines were written to be consistent with concepts in the International Existing Building Code (ICC, 2024), which is the governing reference standard in many U.S. jurisdictions.

The intended users FEMA P-2335 are engineers with experience in seismic design and assessment and building officials with authority over such projects. The Guidelines address damage only to structural components, and do not provide direct guidance for assessment, repair, or retrofit of secondary structures,

nonstructural components, or contents. Earthquake effects beyond ground shaking, such as surface fault rupture, foundation settlement, liquefaction, lateral spreading, and landslide, are not directly considered.

Identification of Earthquake Damage

Earthquake damage to buildings is identified through inspection and analysis. The goal of this step is to develop a good understanding of the performance of the building in the earthquake by identifying all locations of potential performance-critical damage, and documenting this damage with sufficient detail such that this damage can be evaluated subsequently. As shown in Figure 5, this procedure emphasizes visual inspection. Three different levels of inspection are described, which involve varying levels of disruption. For buildings where detailed inspection of the structural components is impeded by architectural enclosures or other obstructions, a structural analysis of the building under the damaging earthquake shaking may be performed to help identify damage.

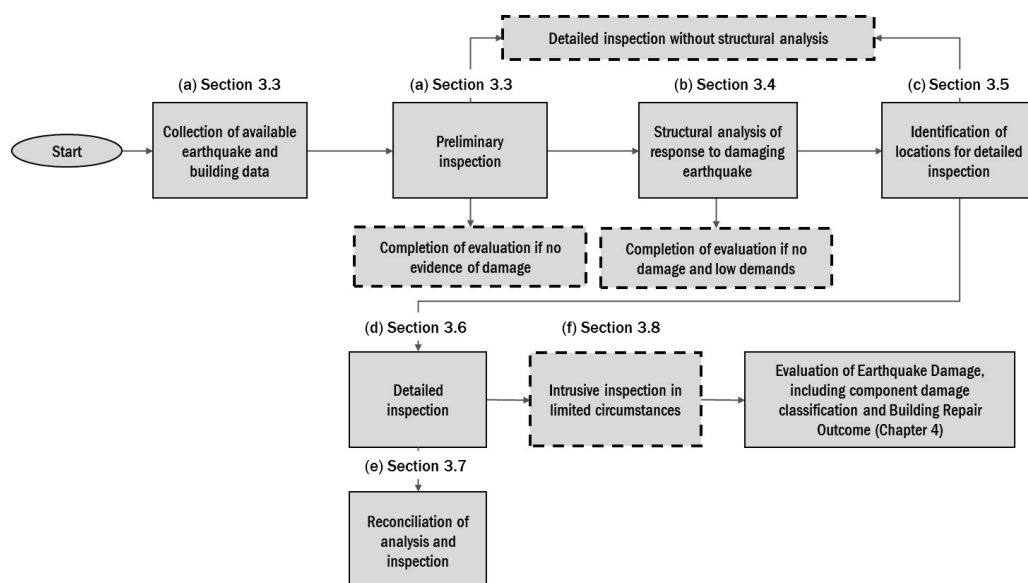


Figure 5. FEMA P-2335 procedures for identifying earthquake damage through inspection and analysis. Section numbers refer to the relevant chapter of FEMA P-2335. (Reproduced from FEMA P-2335)

The first inspection is a preliminary inspection. This inspection may involve non-destructive removal of nonstructural finishes such as ceiling tiles. At this stage, the engineer should also collect information about the intensity of damaging earthquake shaking at the site, and existing building drawings if they are available. The role of the visual inspection is therefore to identify structural earthquake damage, identify irregularities, identify pre-existing conditions, and collect/confirm existing earthquake data.

To determine the intensity of damaging earthquake shaking at the site, in the US, the USGS ShakeMap (<https://earthquake.usgs.gov/data/shakemap/>) can be used. Although ShakeMap previously represented ground motion intensity with PGA and spectral acceleration 0.3s and 1s periods, for future damaging earthquakes the intensity will be defined by 22 spectral accelerations at a variety of periods. Ground shaking at a particular site, as well as information about the nearest available time histories for analysis (if desired), are available from the ShakeMap Sampling Tool.

Detailed inspection follows preliminary inspection. Detailed inspection involves inspection that of the surface of structural components and, therefore, may include the destructive removal of non-structural finishes (e.g., fireproofing, ceiling tiles). Detailed inspection should generally begin in locations where

damage is most likely to have occurred based on the engineers' understanding of the design, construction and vulnerabilities of the building. FEMA P-2335 provides recommendations whereby if initially selected locations do not show damage, the inspection frequency can be reduced. Conditions always requiring inspection are also defined, depending on the material and structural system. For reinforced concrete systems, conditions always requiring inspection include heavily-loaded columns and slab-column connections, and certain shear and flexure-critical walls.

The engineer may choose to develop a structural analysis model to inform the detailed inspection. This analysis model is subjected to estimates of the damaging earthquake shaking. The Guidelines provide procedures describing the use of structural analysis results to identify inspection locations for detailed inspection. To do so, demands are estimated on structural components and compared to the inspection indicators. These inspection indicators are defined in the Guidelines such that demands less than these indicators are highly unlikely to correspond to performance-critical damage (see also Opabola et al. 2023). Figure 6 shows results of a case study frame structure damaged in the Christchurch, New Zealand, earthquake, showing the identification of inspection indicators. The analysis results in this figure are described by a demand-capacity-ratio or DCR. All structural components have $DCR > \text{Inspection indicator}$ indicating that this analysis identified a need to inspect all structural components. The figure also shows that because of the large amount of damage sustained, this level of inspection was appropriate.

The final type of inspection used sparingly in the methodology is intrusive inspection. Intrusive inspection is needed in locations where little damage is observed but other indicators strongly suggest damage is likely. Intrusive inspection may also involve nondestructive testing.

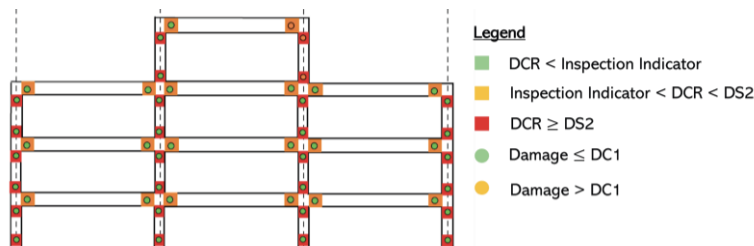


Figure 6. Example frame structure, showing relation between structural analysis results (demand-capacity-ratio or DCR) and inspection indicator. In this example, all frame components require inspection (i.e., all are colored orange or red). (Reproduced from FEMA P-2335)

Evaluation of Earthquake Damage

The evaluation process of FEMA P-2335, outlined in Figure 7, is used to determine whether the earthquake-damaged building needs repair to restore strength and deformation capacity (i.e., performance-critical repair). A building may also be found to need both repair and retrofit. This process involves, first, classification of damage to earthquake-damaged components identified through inspection and, second, determination of the building repair outcome.

Damage classification. Once earthquake damage is identified, the engineer classifies the damage of each component into a damage classification (DC). These damage classifications are defined in Table 1 and the associated image. Performance-critical damage is classified as DC2. If a component is in this damage state that component, and therefore the building structure, has lost strength and deformation capacity and its performance is likely to be significantly impaired in a future earthquake. Therefore, repair of the identified performance-critical damage is required.

The visual inspection process is used to classify damage for each component in the building. This process is aided by descriptions of damage associated with each damage classification for each type of component and visual damage state databases. The databases are electronic resources (Rodriguez Sanchez et al. 2024, <https://www.designsafe-ci.org/data/browser/public/designsafe.storage.published/PRJ-4772>) that compile experimental tests on components. As shown in Figure 8, for each test, the database provides information on the geometry and key characteristics of the specimen, reproduces the force-deformation responses, and then provides images and descriptions associated with each damage classification. These visual tools aid the appropriate identification of performance-critical damage.

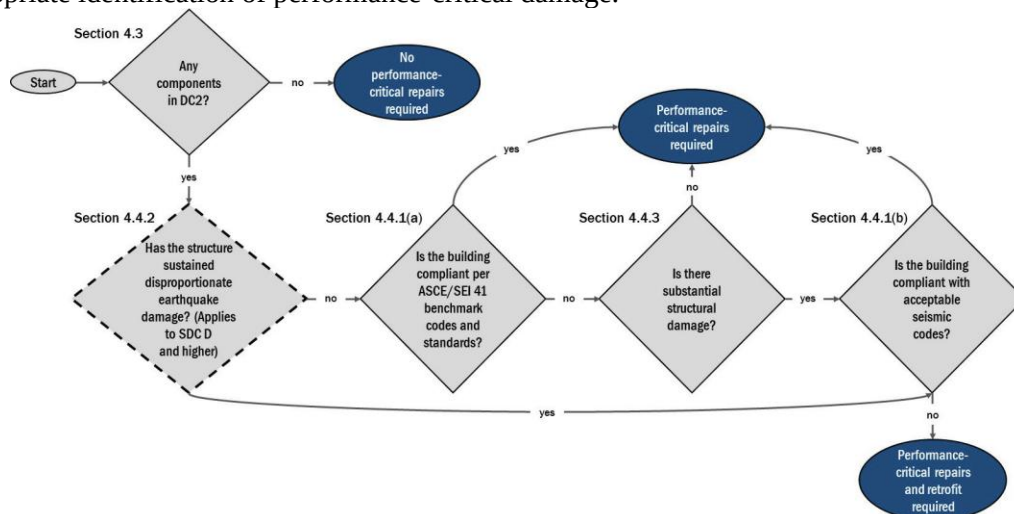
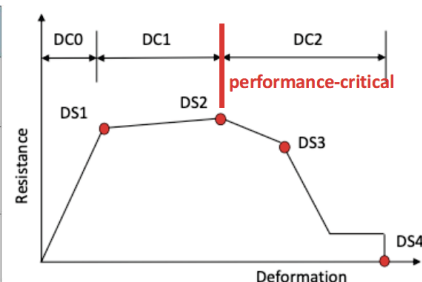


Figure 7. FEMA P-2335 procedures for evaluating earthquake damage. The outcome of this process is damage classifications for all structural components and determination of the building repair outcome. Section numbers refer to the relevant chapter of FEMA P-2335. (Reproduced from FEMA P-2335)

Table 1. Definition of component damage classes in FEMA P-2335.

Damage Class	Relation to Component Damage States	Damage Description
DC0	Prior to DS1: Essentially elastic response	No performance-critical damage; unlikely to require cosmetic, durability, or serviceability repairs
DC1	Between DS1 and DS2: Inelastic response without component strength loss	No performance-critical damage unless indicated by other material-specific checks (e.g., fatigue damage), but cosmetic, durability, or serviceability repair may be warranted
DC2	Past DS2: Inelastic response with component strength loss	Performance-critical damage



To use the visual damage databases, the engineer identifies the type of component (e.g., a shear-critical wall). Then, referring to the data/images from their inspections and the text describing performance-critical damage for each component type, the engineer develops initial estimates of the damage classifications. Subsequently, following guidance in the visual damage databases about the critical parameters, the most representative laboratory tests are identified in the databases. The images provided in the databases are used to confirm or revise the damage classification.

Figure 9 provides an example of damage classification using the visual damage state databases for a building damaged in the Christchurch earthquake. This building had a number of damaged components, and the inspection documented the condition of those components. These photos are then compared to images from test specimens in the database, which help indicate that this specimen is between damage

state (DS) 1 and DS 2, or damage classification 1. DS2 is the lower bound threshold for DC2, hence it can be concluded that these beams did not experience performance-critical damage.

If the visual evaluation is inconclusive, FEMA P-2335 also provides quantitative limits that can be used to identify if performance-critical damage has occurred. These limits (developed as described in FEMA P-2335 and Opabola et al. 2023) are to be used with a structural analysis model; if the demand in the analysis model exceeds the limits, then performance-critical damage has occurred. For deformation-controlled components, these values are typically 0.75 times ASCE/SEI 41 (ASCE/SEI 2023) values for modeling parameters a or d . The 0.75 converts ASCE 41 limits to the appropriate limit for performance-critical damage. For force-controlled components, these limits are typically based on expected strength. The Guidelines also provide procedures to assess if the reinforcement has experienced low-cycle fatigue.

GENERAL INF.			GEOMETRY						Vertical Reinforcement		Horizontal Web Reinforcement		Force-Deformation Response	Drift at which damage photos were taken	DC1					
Spec #	Author(s)	Specimen Name	Axial Load Ratio (%)	Shear Span Ratio	Shape	Aspect Ratio H_w/L_w	f_{cu}/f_{yw} (%)	Measured f_{cu} (MPa)	S/d_v	$f_{wy,prov}/f_{wy,act}$	ρ_w (%)	Ratio $(A_{wy,prov}/A_{wy,act}) \times 100$			P_{ov} (%)	$f_{y,ov}$ (MPa)	P_{sh} (%)	$f_{y,ov}$ (MPa)	DS1-A (Yielding)	DS1-B (Initiation of Cover Spalling)
1	Almeida et al., 2014	TW2-Web BE	3.21	1.17	4	0.74	61	50.7	21.7	0.00	0.50	0.00	0.50	460	0.36	460				

Figure 8. Screenshot showing information in visual damage state databases. There are 14 such databases for reinforced concrete available on Design Safe (Rodriguez-Sanchez et al. 2024).

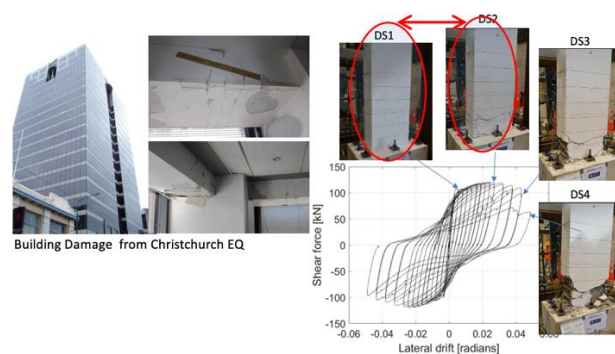


Figure 9. Case study showing use of visual damage database for reinforced concrete components in a building damaged in Christchurch.

Building repair outcome. Component damage classifications provide the basis for the determination of the building repair outcome, which indicates whether performance-critical repair and/or retrofit are required (Figure 7).

If performance-critical repairs are required, retrofit may also be needed. The IEBC includes criteria that use observed earthquake damage “to identify especially vulnerable buildings at critical points in their useful lives and to require evaluation and possibly upgrade” (2018 IEBC, commentary to Section 405.2.2). In the IEBC, this determination is made through evaluation of compliance with acceptable codes and standards, substantial structural damage and disproportionate earthquake damage. The FEMA P-2335 Guidelines can be used to inform judgment as to whether substantial structural damage or disproportionate earthquake damage has occurred. Figure 7 provides the suggested workflow for these evaluations.

According to the IEBC, if a structure complies with acceptable codes and standards, regardless of the level of damage incurred, it may be repaired to its pre-damaged condition and retrofit (i.e., seismic upgrade) is not required. There are a variety of procedures for demonstrating compliance. In FEMA P-2335, the workflow suggests first checking compliance with reference to the benchmark building codes and standards defined in the most recent version of ASCE/SEI 41 Table 3-2. Buildings not satisfying the benchmark codes and standards may also be shown to be compliant by demonstrating that the building meets strength and stiffness criteria with reduced seismic forces, taken as 75% of the prescribed seismic forces for new buildings, or by following ASCE/SEI 41 Tier 3 procedures.

Substantial structural damage occurs if either the lateral-resisting system or gravity-resisting system have degraded beyond the limits specified in the IEBC. For the lateral capacity, substantial structural damage has occurred if the damage has reduced the capacity by $\geq 33\%$. FEMA P-2335 damage classifications can be used to determine this reduced capacity by setting the capacity of components in DC2 to zero, accounting for the presence of performance-critical damage in the calculation of reduced lateral capacity. Work done by the project team confirmed that this approach is appropriate for calculation of lateral capacity with the IEBC’s 33% limit. Substantial damage observed to gravity-essential components indicates that the capacity of vertical components carrying gravity load has been substantially reduced from its pre-damage condition. If gravity-essential components in DC2 support more than 30% of the load at the roof or any individual floor, the building is deemed to have sustained substantial structural damage. Gravity-essential components are explicitly identified in the Guidelines. For reinforced concrete systems, these include columns classified as non-conforming and flexure-shear or shear-controlled, walls classified as shear-controlled or non-conforming flexure-controlled, certain beam column joints, slab column connections without continuity reinforcement, and transfer beams. Although the IEBC has a gravity capacity check, in FEMA P-2335, the details of the gravity load-carrying capacity condition are modified such that they are appropriate for effects of seismic damage on loss of gravity-load-bearing capacity.

Disproportionate earthquake damage is earthquake damage that is excessive given the level of shaking that occurred, indicating a potentially significant problem in the building. According to the Commentary of the 2018 IEBC, disproportionate earthquake damage “exists where a building has significant damage in even a very small earthquake. This damage is an indicator of severe damage, possibly collapse, in a larger event.” Disproportionate earthquake damage is identified if the ground shaking is relatively low compared to the mapped acceleration parameter and the lateral-force-resisting capacity of any story in any horizontal direction has been reduced by more than 10% from its pre-damaged condition. The Guidelines also recommend that the evaluating engineer form a view as to whether the damage observed is consistent with the expected modes of inelastic deformation of the structural system in question. For example, significant shear damage or failures in beams or columns or joints would be inconsistent with the desired modes of inelastic deformation for a modern moment resisting frame.

Of course, even if not required, an engineer or an owner may choose to conduct additional repairs or retrofit. If performance-critical repair is not required, repair may still be warranted for other purposes (e.g., cosmetic, durability). If retrofit is not required, an engineer may still recommend retrofit to address

pre-existing earthquake vulnerabilities and improve performance in future earthquakes. Where retrofit is needed, the engineer is referred to existing documents on retrofit, primarily ASCE/SEI 41.

Repair of Performance-Critical Damage

If performance-critical damage is found and repair is therefore warranted, FEMA P-2335 also provides guidelines on repair techniques. For reinforced concrete components, the Guidelines document whether the repair techniques for concrete and reinforcement act as a repair to restore appearance or durability, a performance-critical repair, or both. FEMA P-2335 also categorizes repair types by damage type, and provides descriptions of the major repair techniques, and their advantages and limitations.

Conclusions and Ongoing Work

The goal of this paper is to introduce new post-earthquake repair guidelines soon to be published by FEMA. Work is ongoing to expand the resources to include reinforced masonry and steel structures.

Acknowledgments

This is an ongoing project under FEMA NEHRP Architect-Engineer Contract. Final results, conclusions and recommendations are still pending. The author(s) are solely responsible for the accuracy of statements or interpretations contained in this publication. No warranty is offered with regard to the results, findings and recommendations contained herein, either by the Federal Emergency Management Agency, the Applied Technology Council, its directors, members or employees. These organizations and individuals do not assume any legal liability or responsibility for the accuracy, completeness, or usefulness of any of the information, product or processes included in this publication.

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PERFORMANCE OF REINFORCED CONCRETE BUILDINGS DAMAGED DURING THE 2024 NOTO PENINSULA EARTHQUAKE IN JAPAN

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Abstract

The January 1st (M_j=7.6), 2024, Noto Peninsula earthquake in Japan led to one of the strongest ground shakings ever measured. During this earthquake, some reinforced concrete buildings were damaged. This paper describes immediate observations of damage to RC buildings in the 2024 Noto Peninsula earthquake. One RC building toppled over onto a neighboring building due to some cause. Furthermore, some RC buildings were observed to have tilted or settled, though the structural members of the superstructure in the building were not heavily damaged. The global tilt angle of some buildings was more than 2%. Although not all are clear, pile damage in the substructure of several buildings has been identified. Therefore, the functional use of some buildings was not allowed after the earthquake. Some structural elements of the superstructure were damaged, but these buildings were generally constructed before 1981 when the Japanese Building code was updated. This damage investigation was also conducted in large ground displacement based on data obtained by aerial laser survey. As a result, the damage to some buildings was confirmed to be associated with ground deformation, and it was relatively consistent with the result of the aerial laser survey.

Introduction

The January 1st (M_j=7.6), 2024, Noto Peninsula earthquake in Japan led to one of the strongest ground shakings ever measured. This earthquake and the accompanying tsunami caused damage, such as the collapse of many buildings, mainly in Ishikawa Prefecture. In response to this earthquake, the National Institute for Land and Infrastructure Management (hereafter, NILIM) of the Ministry of Land, Infrastructure, Transport and Tourism and the Building Research Institute (hereafter, BRI) of the National Research and Development Agency began a field survey to grasp the damage situation, mainly to buildings, and to utilize it for necessary measures. The field survey investigated the damage to wooden buildings, reinforced concrete buildings, foundations and ground, steel frame buildings, non-structural members, and the damage caused by the tsunami and fire. This paper describes immediate observations of damage to RC buildings in the 2024 Noto Peninsula earthquake.

General Performance of Reinforced Concrete Buildings

In 1981, a new design law for the upper structure in Japan required the minimum performance level at which the building should continue operating during medium earthquakes and not collapse during large earthquakes. This paper classifies the buildings surveyed into two categories: those designed before 1981 and after 1981. Table 1 lists some of the investigated buildings. The building damage is shown in another chapter, but damage to the reinforced concrete structure was relatively minor, while damage degree to the pile foundations was severe.

Strong Motion Record. Strong ground motion earthquake records at the time of the main shock are available from K-NET and KiK-net of the National Research Institute for Earth Science and Disaster Prevention and from seismic intensity stations of the Japan Meteorological Agency (NIED and JMA).

Figure 1 shows the epicenter and observation points on the Noto Peninsula. Figure 1 also indicates acceleration response spectra in Wajima City and Nanao City.

Table 1 List of some of the investigated buildings

Location	Building Number	Number of Stories	Structural types	Construction year	Seismic retrofit	Damage degree of pile foundation	Damage degree of super structure
Wajima	A-1	2	RC	1968	2013	Moderate~Severe	
Wajima	B-1	3	RC	1970	2012	Severe	Moderate
Wajima	B-5	3	RC	1970	2013	Severe	Slight
Wajima	C-1	3	RC	1973	2013	Severe	Minor
Wajima	D-1	4	RC	1985	–	Moderate~Severe	Minor
Wajima	G	3	RC (Wall structure)	1999	–	Minor~Severe	
Anamizu	J-2	2	RC	1994	–	Moderate~Severe	
Wajima	M	2	RC+S (roof)	2001	–		
Kanazawa	N	3	RC	1983	–		
Wajima	AA	7	RC	1972	–		
Wajima	AB	7	RC	1977	–	Severe	
Wajima	AC-1	5	RC	1975	–	Severe	
Wajima	AC-2	5	RC	1976	–	Moderate~Severe	
Wajima	AC-3	5	RC	1977	–	Moderate~Severe	
Wajima	AD	5	RC	1978	–	Severe	
Anamizu	AG	2	RC	1979	–	Severe	
Nanao	AI-1	6	RC	1972	–	Severe	
Nanao	AI-2	10	SRC	1982	–	Minor~Severe	
Nanao	AJ	7	RC	1961	–	Severe	
Anamizu	AM	2	RC	1974	–		
Wajima	BA	2	S	1992	–		

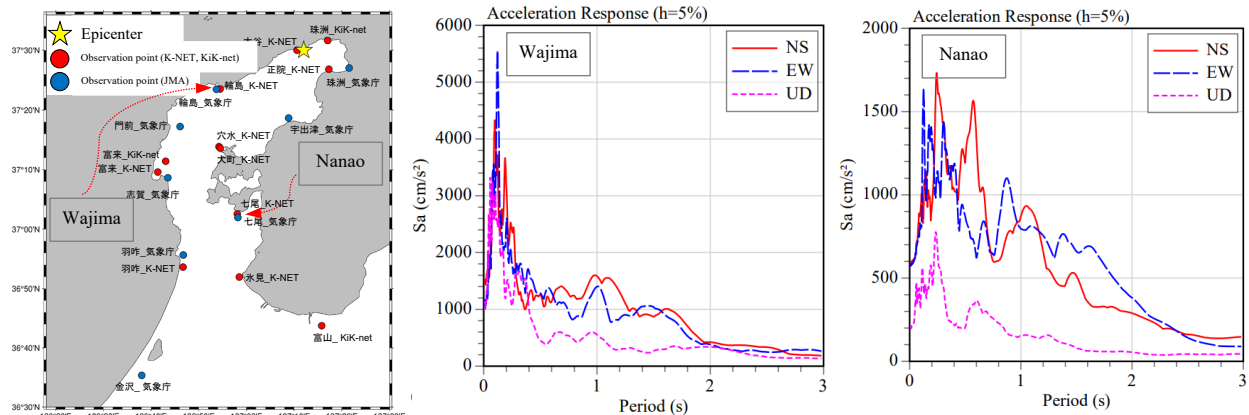


Figure 1. The January 1st ($M_j=7.6$), 2024, Noto Peninsula earthquake in Japan led to one of the strongest grounds (NILIM and BRI, 2024)

Buildings constructed or retrofitted after 1981. Building A-1 is a two-story RC building constructed in 1968. It was seismic retrofitted in 2013, and an addition, Building A-2, a two-story steel structure, was added in 2020. The foundation is a pile foundation with precast reinforced concrete hollow piles (hereafter, RC piles) with a diameter of ϕ 300 mm used for Building A-1. Figure 2 shows the building A-1 and A-2. In the superstructure of Building A-1, damage to expansion joints (hereafter, Exp. J.) and deformation of partitions were observed, but no notable damage to structural members was observed. However, in Building A-1, The global tilt angle of Building A-1 was about 12/1000. A pile foundation investigation was conducted at Building A-1, confirming that the pile heads were damaged and longitudinal reinforcement was exposed.



Figure 2. The building A-1 and A-2



Figure 3. Damage condition of piles

Figure 4 shows Building C, which is an educational facility. The school has five buildings, Buildings C-1 through C-5, separated by Exp. J. Building C-1 is a three-story RC structure, Buildings C-2 through C-4 are two-story RC structures, and Building C-5 is a one-story steel structure. Buildings C-1 through C-4 were constructed in 1973, with Building C-5 added in 1980, and seismic retrofitting of Buildings C-1, C-2, and C-5 was performed in 2013. The seismic retrofit of RC buildings C-1 and C-2 included the installation of framed steel bracing, as shown in Figure 5, installing additional seismic walls, and increasing the cross-section of RC columns. Pile foundations in the existing section are precast prestressed concrete hollow piles (hereafter, PC piles) with 300-400 mm pile diameters in Buildings C-1 to C-5. Building C-1 was found to slope 18/1000 to the northwest, Building C-2 12/1000 to the west, and Building C-3 13/1000 to the west. As shown in Figure 6, on the north side of Building C-1, a pronounced undulation of the first floor was observed. Figure 7 shows the shear cracking of RC walls. Cracks were observed in some columns and walls, but it was not significant necessary.



Figure 4. Building C-1



Figure 5. Seismic retrofit of building C-1



Figure 6. The undulation of the first floor



Figure 7. Shear cracking of wall

Building G is a three-story wall RC structure constructed in 1999, with a horizontal plan shape of seven dwelling units per floor, as shown in Figure 8. The eastern slope is subject to a large-scale landslide. Significant ground deformation is also occurring on the west side in the westerly direction. The pile head was observed to be displaced from the footing at the southwest corner. The displacement of the footing from the pile head joint surface was approximately 600 mm in the west direction and 600 mm in the north direction. According to the design documents, the piles of Building G are precast prestressed high-strength concrete hollow piles (hereafter, PHC piles) with a diameter of $\phi 400$ (Figure 10 and Figure 11). As for damage to the superstructure, cracks (crack width of about 0.3 mm) were observed in the beams of the stairwells, as shown in Figure 9.



Figure 8. Building G



Figure 9. Cracks on RC beam



Figure 10. The gap between piles and the upper structure



Figure 11. Exposed cross section of pile head

Buildings constructed before 1981. Building AA is a seven-story RC structure constructed in 1972, as shown in Figure 12. A small drone was used to confirm the damage to the overturned building. On the overturned upper surface (west side of the building), significant damage was observed near the boundary between the second and third floors. Figure 14 shows the condition of the pile cap and foundation beam.



Figure 12. The building AA (South side)



Figure 13. The building AA (West side)

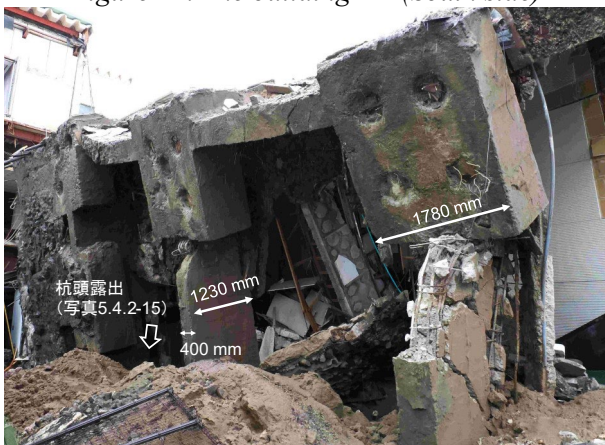


Figure 14. Damage condition of pile cap and foundation beam

Building AB is a seven-story RC (partially SRC) structure constructed in 1977, as shown in Figure 15. Figure 16 shows the swelling of the 1st floor. On the first floor, a slope of 76/1000 was observed in the northeast direction. The settlement amount was 1390 mm on the east of the 1st-floor columns. Cracks of about 0.3 mm were observed in the columns on the first floor, and cracks of about 1.0 to 1.5 mm were observed in some of the sleeve and hipped walls, but the damage was small compared to the degree of settlement and inclination.

The AI-1 building is a 6-story RC structure, and the AI-2 building is a 10-story SRC structure. Figure 17 shows Building AI and Figure 18 shows the typical damage of non-structural walls. AI-1 was completed in 1972, and AI-2 in 1982. Building AI-1, located on the south side, slopes 1.4% to the west. Building AI-2, located on the north side, was sloped 0.4% to the north. According to the design documents, the piles used in Building AI-1 were RC piles with a cross-shaped cross-section, while the piles used in Building AI-2 were PC piles with diameters of 450 mm and 500 mm.

Figure 19 shows Building AM, a two-story RC building constructed in 1974. Figure 20 shows the RC columns' damage condition under the beams on the first floor. This RC column had spalling of the concrete.



Figure 15. The building AB (southeast side)



Figure 16. Swelling of the first floor



Figure 17. The building AI

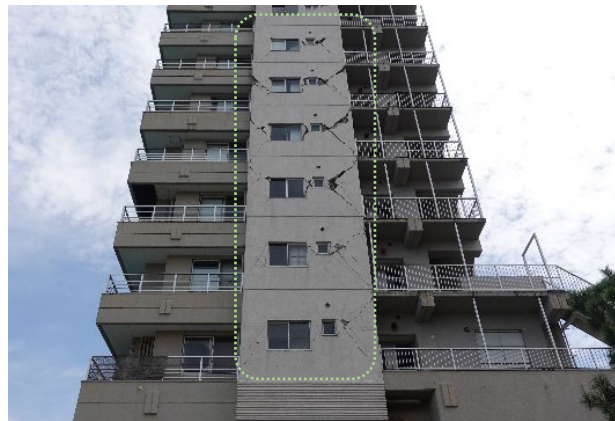


Figure 18. Shear cracking of non-structural walls



Figure 19. The building AM



Figure 20. Damage to RC column

Below are some results of the ground displacement evaluation by aerial laser measurement, as shown in Figure 21. A particularly large ground displacement was found in the northeastern part of the subject area, so the building BA was investigated.

Building BA is a two-story steel structure built in 1992, according to the facility manager. Part of the ground had collapsed, and part of the 1st floor was severely deformed. Ground deformation and exposed

pile heads were observed in the surrounding area (Figure 22). Cracks were observed at the pile cap. In addition, a diagonal crack was observed in the center of the pile cap.

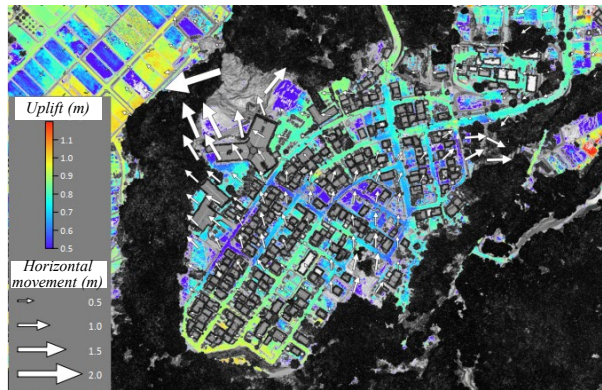


Figure 21. Results of areas with large ground deformation based on aerial laser measurements



Figure 22. Ground deformation

Recent Studies on Piles and Pile Caps in NILIM and BRI

This section discusses recent piles and pile caps studies conducted by NILIM and BRI. The superstructures of buildings are typically designed for two types of earthquakes: small to medium and large. However, the design of foundation structures, such as piles, usually follows a working stress design and does not often consider ultimate conditions. Ongoing research on pile and pile head joints is being carried out to ensure the safety and continued usability of buildings with concrete pile foundations during a large earthquake. This research aims to gather essential primary data needed for the performance design of pile members at NILIM and BRI.

Figure 23 shows the loading setup of the precast reinforced concrete pile (hereafter, PRC pile). This is a loading system in which four 5MN capacity jacks are placed on the specimen, and shear force is applied by the two 4MN hydraulic jacks in the center. The experimental results of the Q-R relationship and failure mode are also shown in Figure 25 (Mukai et al., 2019). Brittle failure was found in the PRC pile with an axial load ratio of 0.18.

Figure 24 shows the outline of the development of ductile precast concrete piles. Some buildings were tilted in the 2011 Tohoku earthquake and the 2016 Kumamoto earthquake due to precast pile damage. Therefore, it was difficult for these buildings to be used continuously after the earthquake. To solve this problem with precast piles, the research project “Development of seismic resistance evaluation technology for continuous use of existing buildings after an earthquake (2019-2021)” was conducted in BRI (Mukai et al., 2023). This project involved more structural tests for several issues. Furthermore, case studies with ductile concrete pile foundation systems were conducted.

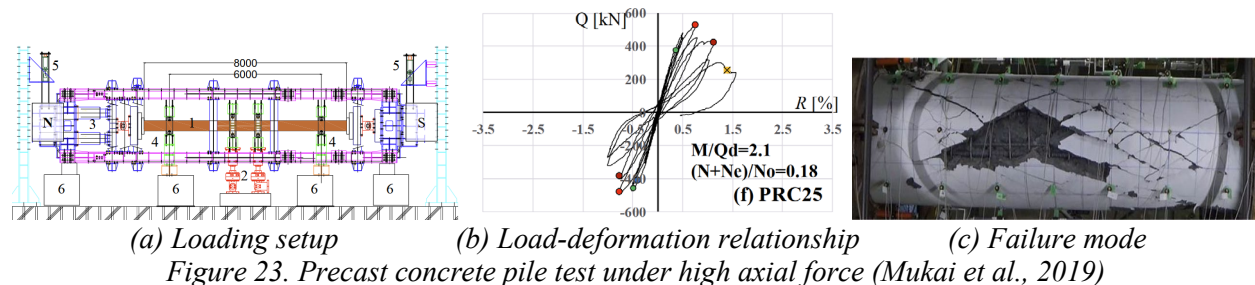


Figure 23. Precast concrete pile test under high axial force (Mukai et al., 2019)

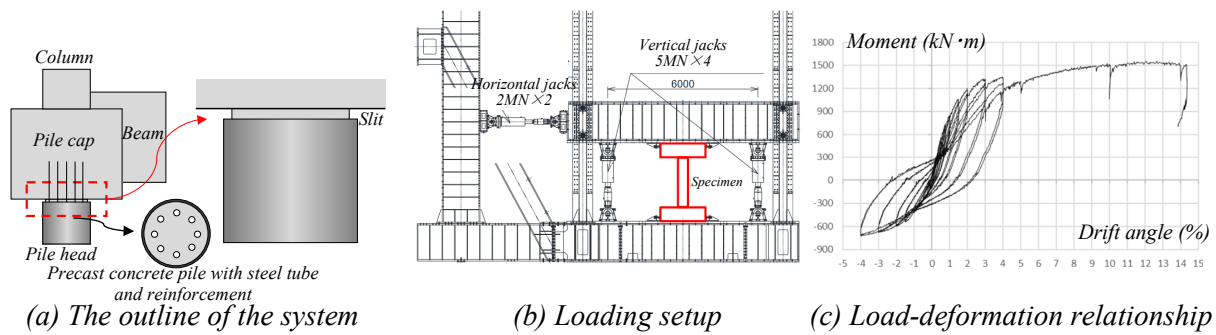


Figure 24. The outline of the development of ductile precast concrete piles (Mukai et al., 2023)

Conclusions

This paper describes the outline and characteristics of damage to reinforced concrete buildings caused by the 2024 Noto Peninsula Earthquake. Below is a summary of the damage's characteristics.

- In buildings constructed or seismically retrofitted after 1981, no significant damage to the superstructure's structural members was observed. However, tilt-up damage equivalent to a major failure of the pile foundations was noted, presumably due to foundation damage. The damage to the superstructure is minimal compared to the damage suffered by the foundation.
- For buildings constructed before 1981, damage was observed in columns, beams, and vertical walls, yet there was no substantial damage overall. Additionally, overturning and tilt damage indicative of catastrophic failure, likely caused by foundation issues, was noted. Building AA is RC building without an earthquake-resistant design for piles; this is the first instance of such a case in Japan.
- A focused survey of areas with significant ground displacement was conducted using an aerial laser measurement system. An evaluation of the relationship between the results of the building collapse assessment from aerial laser measurements and actual building damage revealed a certain degree of consistency.

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A DATA-DRIVEN APPROACH FOR POST-EARTHQUAKE ASSESSMENT OF SEISMIC DAMAGE IN WOODFRAME BUILDINGS

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Abstract

Data-driven seismic damage assessment has significantly advanced the evaluation of potential structural damage caused by earthquakes, particularly for steel and reinforced concrete structures. However, the assessment procedure for woodframe (WF) buildings, which pose a significant disaster preparedness challenge due to their prevalence and vulnerability, are yet to be fully developed. This paper aims to address this gap by combining computing technology, remote sensing, and data science to develop a post-earthquake damage assessment tool specifically tailored for WF buildings. The primary objective is to establish a robust SHM framework that utilizes acceleration data from instrumented WF buildings for real-time damage assessment. Structural health monitoring (SHM) techniques are adopted to utilize structural vibration response data. Four key stages of the process are: health definition, data acquisition, feature engineering, and machine-learned damage assessment. At the beginning, limit states of WF buildings are defined and damage indices developed for accurate assessment. We report findings of the health definition phase in this paper. Dataset obtained from a series of shake table tests, along with acceleration data from instrumented woodframe buildings, are utilized. The outcomes are expected to enhance understanding of WF building performance during earthquakes, contributing to more effective disaster preparedness and response strategies for these structures.

Introduction

Post-earthquake damage evaluation is essential for earthquake resilience, with experts employing various methods like drift analysis (Dai et al. 2017), image processing (Abdi and Jabari 2021), and visual inspection. Recently, data-driven damage assessment techniques, particularly machine learning (ML), have become popular for real-time evaluations, making structural health monitoring (SHM) a dynamic field that adapts to advancements in remote sensing. SHM's goal remains to replace manual inspections with automated systems that can guide maintenance decisions. Comprehensive methodologies in SHM have been developed to detect, localize, and quantify structural damage in civil structures. SHM fundamentally involves analyzing changes in a structure's dynamic properties due to health deterioration and is generally categorized into two approaches: signal-based and model-based.

Model-based SHM uses finite element models (FEM) to simulate responses under input excitations, comparing these to actual responses (Moaveni et al. 2013). Although effective, it demands substantial resources and prior structural knowledge. In contrast, signal-based SHM is more efficient and adaptable, as it only requires a response signal and an advanced mathematical model, eliminating the need for prior system knowledge (Ay and Wang 2014). This technique identifies damage by analyzing exclusive features in the response signal through statistical models like the Auto-Regressive (AR) model, ARMAX (Ay and Wang 2014), Discrete Fourier Transform (DFT), Wavelet Transforms (WTs) (Hou et al. 2013), and Signal Simulation-based Feature Selection (SSFS) (Gharehbaghi et al. 2021). Following signal processing, ML is often applied for effective feature selection and damage detection (Muin and Mosalam 2021; Huang, Li, and Fu 2019).

Analyzing signal characteristics is also crucial for assessing building damage. Signals from accelerometers and potentiometers can reveal insights for energy dissipation-based damage evaluation (Choi and Stubbs 2004). However, signals from reinforced concrete (RC) buildings and steel-frame buildings differ significantly, and characteristic studies are needed for each. Although effective studies exist for RC (Kappos 1997) and steel-frame buildings (Kalateh-Ahani and Amiri 2021), woodframe buildings are underrepresented. This study addresses this gap by analyzing signals from an instrumented two-story woodframe building to determine a damage-sensitive feature, termed Damage Index (DI), that integrates nonlinear structural characteristics.

As a part of the NEESWood project, a two-story woodframe structure equipped with instrumentation was constructed in five different phases and subjected to testing on two tri-axial shake tables, each simulating distinct levels of ground motion intensity. The project database (van de Lindt et al. 2007) offers useful building response data during the test which is utilized in this study for feature engineering using displacement and energy based damage assessment model (Park and Ang 1985). The effectiveness of this model for global level damage detection has been demonstrated to exhibit a strong association with observed damage in the soft story woodframe building, as evidenced by previous study (MD. Ashikin, Sifat Muin, and Asif Iqbal 2023). This study on two story woodframe building having no soft-story will generate more reliability in the DI based damage assessment and lead to constructing genuine feature set making ML algorithm more accurate later on.

Methodology

Structural Health Monitoring (SHM) is a field of science that works with defining and determining the performance ability of the structure. There are many techniques of SHM and machine learning (ML) algorithm is one of the most successful modern day techniques (Gomez-Cabrera and Escamilla-Ambrosio 2022). However, developing the framework for SHM using ML involves few steps, damage conceptualization is the first of them which refers to defining and classifying the nature as well as the extent of damage the structure encounters. The next steps include data acquisition, feature extraction and finally model development. This study investigates particularly on a 2 story instrumented full scale woodframe building which has been tested on a shake table at different ground motion intensities and proposes a systematic guidance on data acquisition and discusses the potential of Damage Index (DI) as a damage indicating feature to pave the way of developing the SHM framework.

Health Definition. In the context of structural health monitoring (SHM), defining the health of the structure and correlating the definition with established four performance levels) through statistical analysis is crucial. Depending on the nature of structure being analyzed, health definition might vary in terms of factors that are taken into account for determining the nature and extent of damage, but it must be capable of consistently correlating with the defined performance levels. Table 1 describes the definition of damage and health state of the woodframe building used for structural health definition of this particular study.

This table outlines a damage scale ranging from 0 to 4, describing various levels of structural and non-structural damage of the test building. A score of 0 signifies no visible damage in any components, while 1 indicates minor damage in non-structural elements. Level 2 suggests significant damage in non-structural parts with occasional minor signs of damage in structural elements too which becomes more pronounced in level 3. Finally, a score of 4 indicates material failure and severe structural damage. This scale showing a clear progression from no damage to severe structural failure, has been adopted while defining the structural health of the NEESWood building after tests.

Table 1. Structural Health Definition of Wood Building

<i>Damage State</i>	<i>Damage Description</i>
Numbers	No damage should be visible in either structural or non-structural members.
0	Hair-line cracks and minor signs of damage are expected at the non-structural members.
1	Significant damage in the non-structural members, sometimes minor sign of damage in structural components may be evident.
2	Cracking and splitting of the structural members will show up.
3	Failure of the material will happen, and severe structural damage will be evident.
4	No damage should be visible in either structural or non-structural members.

Data Acquisition. The data acquisition is the most important step of the SHM framework, and the performance of this framework greatly depends on the appropriateness in the data acquisition method. However, data required for developing any sort of seismic health assessment framework can be classified into three groups. One is the input motion data which generally refers to the ground motion or shake table data while the second one is the building response data and finally the building information data. Ground motion or the shake table data is the earthquake time series data in the form of acceleration waveform whereas the building information data includes building weight, building plan and elevation etc. and building response data includes acceleration and displacement data recorded at different location of the building. In practice, acceleration data are gathered by accelerometer response data only after some signal processing have been done to omit any noise of both low and high frequency in nature. And displacement is either found by double integrating accelerometer data (MD. Ashikin, Sifat Muin, and Asif Iqbal 2023) or directly from string potentiometer.

In order to evaluate component level DI, which is then converted into global DI, two variables are required from the building response dataset namely, maximum displacement D_{max} and the amount of energy $\int dE$ that is dissipated by the component wall in hand. Accelerometer tributary mass from the building information data set and the acceleration response data from the component wall is utilized to build the time history of base shear force for that particular wall which later in couple with displacement time series recorded by string potentiometer or double integrated accelerometer data, makes the hysteresis plot for the component wall.

Damage Index. Damage Index (DI) is a concept that incorporates both elastic and inelastic behaviour of the building to evaluate structural and non-structural damage. It has been established as an effective feature of damage detection for concrete and steel frame buildings (Kalateh-Ahani and Amiri 2021) and the same is found for soft-story woodframe building (MD. Ashikin, Sifat Muin, and Asif Iqbal 2023).

$$DI = \frac{D_{max}}{D_{ult}} + \frac{\beta \int dE}{D_{ult} F_{yield}} \quad (1)$$

D_{max} , D_{ult} , D_{yield} and F_{yield} are maximum, ultimate, yield displacement and yield force respectively. β is a coefficient which depends on the type of the structure and $\int dE$ is the amount of energy dissipation. D_{ult} , D_{yield} and F_{yield} are obtained either from material test data or from numerical static pushover curve (Pan, Ventura, and Tannert 2020; Liang, Wen, and Foliente 2011) and finally $\int dE$ is the area inside the global hysteresis plot.

Equation (1) demonstrates the capability to accurately detect damage at the component level. However, certain adjustments are necessary in order to transform it into a global damage feature (Pan, Ventura, and

Tannert 2020). In order to achieve this objective, the Damage Index (DI) is initially computed for each individual wall. The DI contribution of each wall is then aggregated by multiplying it with the corresponding weighting factor, resulting in a global-level DI.

$$\lambda_i = \frac{E_i}{E_{total}} \quad (2)$$

Here, E_i is the amount of energy dissipated by a particular wall and E_{total} is the total dissipated energy by the system which is the summation of the energy dissipated by each contributing wall of the building. Finally, DI_{global} is calculated as per equation (3).

$$DI_{global} = \sum_{i=1}^n \lambda_i DI_i \quad (3)$$

Shake Table Tests

Building Structure. The NEESWood benchmark testing examines a two-story townhouse in California, designed as a production house in the 1980s or 1990s. The test building simulates 1800 sft of living space of a townhouse and an attached two-car garage. In terms of structure of the building, the exterior walls are constructed with OSB sheathed shear walls, gypsum wallboard on top of it and finally stucco to cover them up. The building skeleton is constructed with 2x4 dimensional lumber for framing, except for the first level of the garage. Sheathing panels are 7/16 in. thick oriented strand board installed vertically, and connectors are pneumatically driven 8d common nails. As far as the architecture goes, Figure 01 depicts how the building architecture looks like and the direction along with the position of the string potentiometers to measure the displacement response of the test.

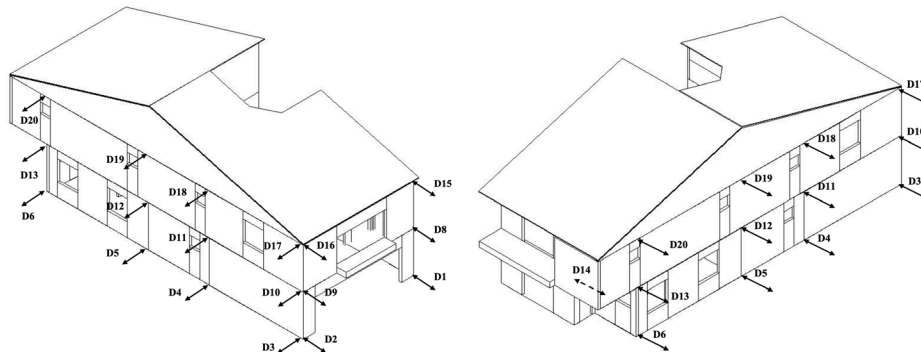


Figure 1. NEESWood test building with string potentiometer arrangement (Christovasilis, Filiatrault, and Wanitkorkul, n.d.).

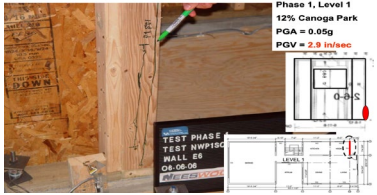
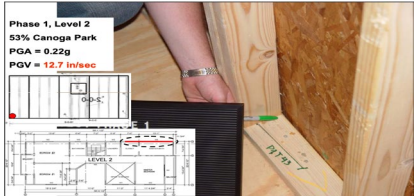
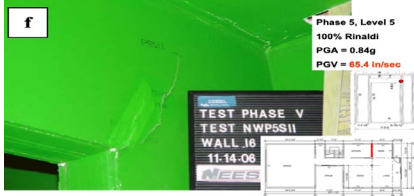
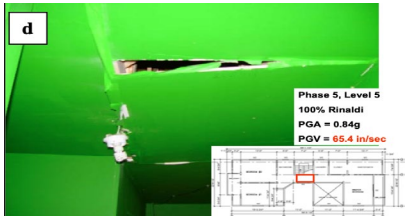
Tests at NEES-UB. The full-scale test building was built and experimented at the University of Buffalo (UB-NEES site) which is based on one of the four index buildings in the CUREE-Caltech Woodframe Project and represents a one of the three units of the two-story townhouse, approximately 1800 sft of living space, and an attached two-car garage. The test structure, which is 18 ft tall and weighs 80 kips, is covered with stucco, OSB sheathed shear walls, and gypsum wallboard on the exterior. The program aimed to understand the construction elements that influence the seismic response of woodframe buildings and should be considered in performance-based seismic design. Therefore, the test structure was built in 5 gradual phases and each phase was tested on shake table with proper instrumentation.

The initial phase of the testing procedure focuses on the fundamental components of wooden structures. During the second phase of the experiment, passive fluid dampers are incorporated into certain wood

shear walls to investigate their impact on the structural behaviour. Phase 3 involves attaching 1/2-inch thick gypsum wallboard to the load-bearing walls with specific screws and later on for Phase 4, the gypsum wallboard is extended to encompass all walls and ceilings. In the final phase, Phase 5, a layer of stucco measuring 7/8-inch in thickness is applied to all exterior walls. This stucco is reinforced with steel wire mesh, which is spaced at a 6-inch on-centre spacing.

The tests aimed to assess the performance of a test building under various levels of seismic shaking intensity. Two sets of tri-axial historical ground motions were used, one from Canoga Park (CP) and the other from Rinaldi (RN), recorded during the 1994 Northridge Earthquake in California. The Canoga Park ground motions were scaled by adopting amplitude scaling factor of 0.12, 0.53, 0.86 and 1.20 respectively to produce four different levels which were used to determine the test structure's response under more frequent ground motions. Up to five levels of seismic tests were performed, with the input ground motions for the first four levels being scaled versions of the Canoga Park record and the input motions for the fifth level being the Rinaldi record, both recorded during the 1994 Northridge Earthquake.

Table 2. Damage Class Based on The Visual Damage Observation

<i>Damage State</i>	<i>Inter-story Drift (%)</i>	<i>Damage Description</i>	<i>Typical Damage Photo</i>
01	0.1-0.5	- Minor splitting of sill plates to cracking of vertical studs - Hairline GWB cracking - Minor cracking of stucco	
02	0.5-1.0	- Sill plate splitting and crack propagates - Partial nail pull-out - GWB cracks and diagonal crack propagates - Garage wall stucco cracking and spalling	
03	1.0-2.0	- Permanent movement of panels - Major cracking and splitting of sill and top plates - Crushing of GWB at the corners	
04	>2.0	- Separation of GWB from ceiling - Significant crack propagation in garage walls - Buckling of GWB at door openings	

Damage Observation and Evaluation. The investigation encompassed visual assessments of damage was carried out subsequent to each seismic test, giving vital insights into the structural and non-structural

elements of the test building. The photographs presented in Table 2 depicts significant forms of damage discovered throughout each stage of testing. The observed damage in three primary components, namely wood shear walls, gypsum wallboard, and outside stucco, is concisely reported in Table 2.

Phases 1 and 2 showed minimal non-structural impairments, ranging from hairline cracks in the stucco and instances of spalling to larger cracks in the gypsum wall board. Phase 3 and 4 shows significant damage in the gypsum wallboard including propagated cracks and crush at the wall corners. Phase 5 shows signs of some structural damages including cracks in sill plates particularly in the garage wall. Accelerometer data along with their corresponding tributary masses are documented in the project report (Christovasilis, Filiatrault, and Wanitkorkul, n.d.) and utilised for developing global hysteresis plot. The specimen test result was utilised to determine the yield and ultimate displacement and so is yield force. Finally, the coefficient β is taken as to be 0.30 (MD. Ashikin, Sifat Muin, and Asif Iqbal 2023).

Equation (1), which helps to evaluate Damage Index (DI) of the test building has two contributing parts. The first one, $\frac{D_{max}}{D_{ult}}$ is the ratio of the maximum displacement and the ultimate displacement of the member. And the second one, $\frac{\beta \int dE}{D_{ult} F_{yield}}$ plays the most pivotal role in the DI calculation, as it accounts for the building's response in terms of energy dissipation during the seismic event. An investigation has been conducted to assess the individual contributions of these two components of DI to the overall DI value of the shear walls at various levels of shaking, corresponding to varying levels of potential damage. Figure 2 which shows the analysis result for each shear wall at different level of shaking and it is quite evident that when the overall DI is low and there is no expected structural damage, the DI value is mainly contributed by the drift ratio. However, as the DI (Damage Index) continues to rise and structural damage is believed to have taken place, the energy contribution to the DI value also continues to increase.

Results and Discussion

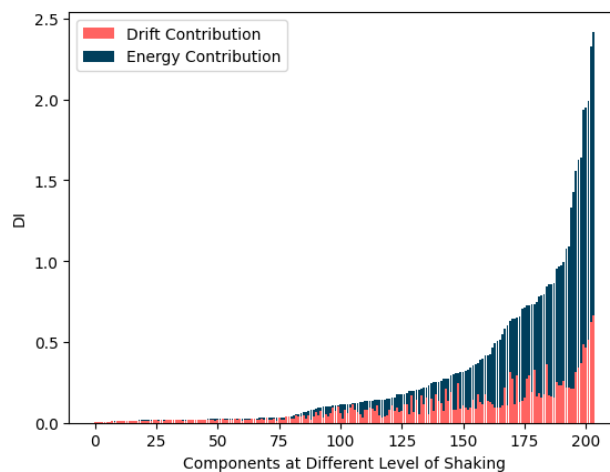


Figure 2. Damage Index for Components at Different Levels of Shaking.

Limit State of Damage Index. A further look into the results has identified 4 different regions that later correlates with the observed damage states defined in the NEESWood project report (Christovasilis, Filiatrault, and Wanitkorkul, 2009). After obtaining the component level Damage Index (DI) result and classifying it, the attention is directed towards transforming it into a global level DI that represents the entire damage of the system, rather than simply a particular component of the building. Subsequently, the

global damage index (DI) is compared to the four observed damage states (Table 2) and it is determined that they are in precise alignment with the four regional classifications of DI discussed earlier in this section which is tabulated in the table 3. Based and correlational observation of DI and damage, limit state criteria proposed are as in Table 4.

Table 3. Global Damage Index (DI) and Corresponding Observed Damage State

<i>Damage State</i>	<i>DI Values</i>
1	0.03, 0.03, 0.02, 0.03, 0.02, 0.01, 0.13
2	0.24, 0.30, 0.32
3	0.60, 0.81, 0.64, 0.81, 0.78
4	1.75, 1.47

Table 4. Proposed Limit State Criteria

<i>Damage State</i>	<i>Damage Description</i>	<i>DI Range</i>
1	Minimal visible damage	0.0 - 0.15
2	Observable structural impairments	0.15 - 0.40
3	Visible damage, may require repair	0.40 – 1.00
4	Severe damage, requires rehabilitation	> 1.0

Conclusion

This study conducted on a two-story full-scale woodframe building insights into buildings behaviour subjected to different level of seismic loading and investigation into Damage Index (DI) has been proven as a tool to understand and categorize the structural damage of the building. Thorough examination and analysis of component level DI, story-wise DI, global DI, correlation between DI and the associated components of it, have unveiled an intricate correlation with the observed damage states. Establishment of four distinct damage region based on DI values along with their correlation with predefined damage states, have paved the way of proposing 4 limit states for global DI values. Further look into story-wise DI has uncovered the uneven damage distribution between the first and the second story, specifically true for higher level damage, which demonstrates the significance of localized damage assessment within multi-story buildings.

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RISK AND RESILIENCE ASSESSMENT IN PRACTICE: A UNIVERSAL TAXONOMY AND LESSONS LEARNED

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Abstract

Resilience of buildings and infrastructure under disasters from natural hazards is crucial for business and operational continuity for stakeholders such as large corporations, public agencies, and national governments. Identifying, understanding, and communicating risks is fundamental to designing mitigation measures and retrofits to manage risk and achieve resilience. Over the past decades, Arup has been involved in risk and resilience assessments at various scales, guiding decision-makers on key property and operational decisions to achieve resilience.

There is currently no universal regulatory framework that differentiates between levels or types of risk and resilience assessment for natural hazards and climate change. A universal taxonomy for natural hazard and climate risk and resilience assessments has been developed, providing a framework and guidance for conducting risk and resilience assessments at various scales with different use cases. This paper summarizes the published universal taxonomy, the general concepts, and their applications. Examples are provided to illustrate how risk and resilience assessments were conducted in practice, across different areas of the taxonomy spectrum. Lessons learned from these assessments, along with identified challenges, are discussed.

Introduction

There is an increasing need for resilience and adaptation in the built environment. Different stakeholders, from public and private property owners to government agencies and financiers, are leading the way to understand the risks to their assets and their desire for resilience in their buildings, infrastructures, and ultimately their businesses and communities. The structural engineering community plays a key role in this, with performance-based and resilience-based design and assessment having been extensively studied and promoted over the past decades. Resilience in the built environment requires more than just design work, which typically focuses on the asset itself and a particular natural hazard. Horizontally, resilience typically requires the consideration of multiple hazards holistically, including the effects of climate change. Vertically, it involves looking beyond structural performance to evaluate the implications of damage, utility disruption, and how business operations will be impacted by such events. This requires mitigation and resilience planning beyond design work, typically involving risk and resilience assessments, which are key engineering steps.

In the past decade, Arup's Risk and Resilience group has worked extensively with numerous clients, including large corporations, public agencies, and international governments, to help them evaluate and improve their asset and operational resilience across the building and infrastructure sectors. Risk and resilience assessments considering different hazards, assets with various levels of complexity, and technical approaches were used. It was often found that practitioners, users and general stakeholders were not speaking the same language, causing misalignment and unmet expectations between the consumers and providers of risk and resilience information and strategies. Ultimately, this inhibits the speed and quality of enacting resilience and adaptation measures at the necessary scale. There is currently no universal regulatory framework that differentiates between levels or types of risk assessment for natural

hazards and climate change, and no criteria governing what should be considered adequate or appropriate for each situation requiring a risk assessment.

The risk class taxonomy (Arup, 2024) was developed and published to fill this gap, providing a framework for establishing the minimum levels of risk assessment needed to make resilience-related decisions or take actions with sufficient confidence to increase resilience. This paper summarizes the established universal risk taxonomy, the general concepts, and its applications. Examples are provided to illustrate how risk and resilience assessments were conducted in practice, following different approaches proposed by the universal risk taxonomy. Lessons learned from these assessments, along with identified challenges, are discussed.

Universal Taxonomy of risk and resilience assessment

The risk class taxonomy outlines various levels of risk assessment to increase confidence in making progressively more costly or complex decisions as users embark on or advance in their journey to resilience. It identifies a number of common applications, including decisions or actions that may be supported by each Risk Class, resulting in plans or projects that align with a given resilience class. Simplified risk class definitions and use cases are summarized in Figure 1 below. Lower Risk Classes are generally intended for awareness, reporting, risk screening, or low-stakes resilience decisions, while higher Risk Classes are intended to unlock higher-stakes resilience and adaptation decisions, which may be costly, invasive, or disruptive. The Risk Class Taxonomy explicitly outlines the requirements for each component of risk assessment, including hazard data, exposure data, and vulnerability data requirements. It includes sub-taxonomies for seismic, flood, wind, and heat hazards, as well as a taxonomy for incorporating climate change effects in risk assessments.

	Basic Hazard Screening Class 0	Basic Risk Screening Class 1	Enhanced Risk Assessment Class 2	Advanced Risk Modeling Class 3
Application	Pre-screening and awareness	Preliminary risk assessment for screening and low-stakes decisions	Risk assessment for material decisions, preliminary resilience actions, and budgets	Advanced risk assessment to inform high-stakes decisions, resilience-based design, implementable resilience actions, and costs
Assessment approach and resolution	Hazard and exposure assessment	Archetype-specific risk assessment	Building-specific risk assessment	Building-specific component-level probabilistic risk modeling
Accuracy and confidence level	Lowest	Low	Medium	High
Level of effort	Lowest	Low to Medium	Medium to High	Highest
Typical risk outputs	Qualitative hazard ratings or scores	Qualitative risk ratings	Quantitative risk metrics (best estimate)	Quantitative risk metrics (fully probabilistic)

Figure 1. Simplified risk assessment taxonomy summary (Arup 2024).

Another useful component of the published taxonomy is the user personas, which are semi-fictional representations of the intended users of the framework. They represent specific roles and responsibilities within an organization that are most likely to find the taxonomies helpful in addressing their specific needs around risk and resilience topics.

The Resilience Class Taxonomy has also been defined to help stakeholders understand the level of resilience solutions development appropriate for their specific needs, with progressively higher resilience classes representing more confidence in the feasibility, complexity, and cost to implement the solutions in existing buildings or new construction. The Resilience Class Taxonomy aligns with the Risk Class Taxonomy, as resilience and adaptation solutions are largely informed by the identification of the factors

driving risk. It focuses on physical interventions and operational measures aimed at managing and reducing risk, increasing adaptive capacity, and/or creating redundancy at the asset level.

Overall, the developed universal taxonomy provides tools for practitioners and stakeholders with consistent terminology and definitions to understand the requirements and approaches for conducting risk and resilience assessments. This ultimately enhances the speed and quality of enacting resilience and adaptation measures at the necessary scale

Example assessments of different risk classes

Class 1 risk assessment. Lower class risk assessments are mostly intended for awareness, reporting, risk screening, or lower-stakes resilience decisions. These assessments are typically conducted at the portfolio level, where limited data is available. The vulnerability of the assets is usually represented at the asset level through an asset fragility curve, similar to the approach used in Hazus (FEMA 2020). The archetype approach is typically used, where predefined archetypes and the associated building-level vulnerability data represent the entire portfolio. Figure 2 shows an example of a building-level vulnerability curve, which relates financial loss and business downtime to peak ground acceleration for earthquakes. This will be combined with the hazard curve, and through integration, the annual risk value can be calculated. The outcome of the assessment will be a risk rating (qualitative) or a range of risk metrics (quantitative) for each asset and hazard, as shown in Figure 2, representing risk ratings for all hazards. This will help stakeholders understand which hazards and assets pairs pose the highest risk warranting further investigation. A highly digitalized workflow is needed for this type of assessment.

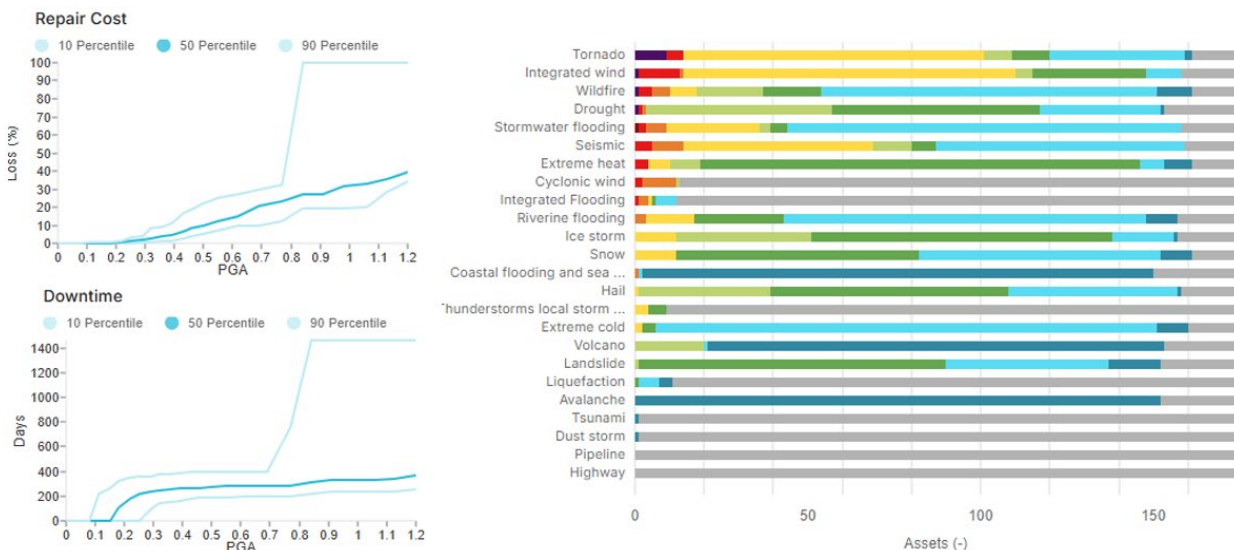


Figure 2. Illustration on Class 1 assessment. Left: Building archetype relation. Right: Example outcome of the assessment.

Class 3 risk assessment. Class 3 risk assessments will be building-specific component-based probabilistic assessment. The purpose is to identify main risk drivers and to inform high-stakes decisions, resilience-based design, implementable resilience actions and cost. The example below in Figure 3 shows a case study conducted for a university campus, examining the seismic risk and resilience of the campus. Detailed structural analysis models were conducted to understand the collapse risk of the buildings and probabilistic seismic risk assessment were conducted to understand the functional downtime of each building as well as the entire campus. The results of the risk assessment are used to help develop the

seismic resilience road map of the campus and building retrofit prioritization based on the building risk results.

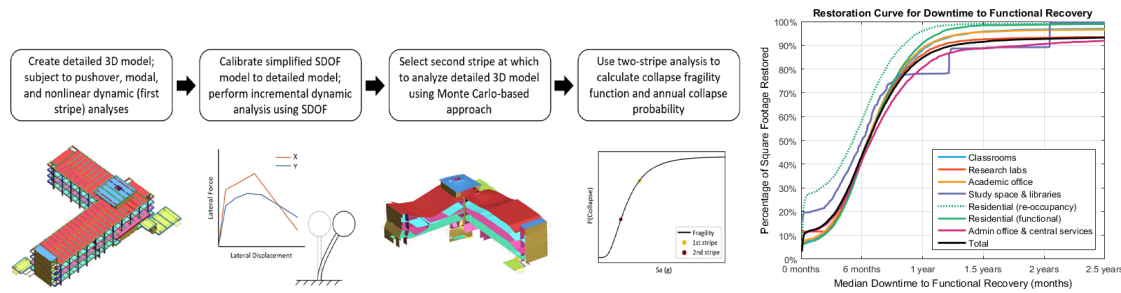


Figure 3. Class 3 seismic risk assessment. Left: Detailed structural analysis to assess building collapse risk Right: Seismic risk analysis to assess the building and campus recovery time.

Lessons learned and discussions

Technical challenge on risk and resilience assessment

- The key step for risk and resilience analysis is engineering risk evaluation. Many studies are based on the probabilistic seismic risk modeling approach outlined in FEMA P58, and earthquake risk modeling has been extensively studied. However, there is a lack of guidelines and frameworks to address other types of hazards despite the early Hazus program (FEMA 2022a, FEMA 2022b).
- Asset vulnerability (whether expressed as general vulnerability or fragility) is one of the key data points for risk assessment and mitigation design. Although extensive data has been published, such as FEMA P58 (FEMA 2018) and Hazus, gaps remain. These include fragility considerations for other types of hazards and asset types including infrastructure, fragility adjustment methods for different existing or retrofit conditions of assets or components, and mismatched fragilities with different qualities and development methods.
- Risk and resilience assessment frameworks published by researchers typically require a large number of inputs, which in practice are often not available even when trying to compile detailed requests for information. While more complicated models are valuable for better capturing different aspects of the assessment, they might not be useful for quick and high-level practical applications given the large uncertainties and the unavailability of information.

Stakeholder engagement

- Resilience design and mitigation actions ultimately need to be initiated by stakeholders, typically building or asset owners or managers, with whom design engineers might not have direct interaction if contracted by architects. Seeking more avenues to directly engage end users will help promote the resilience design concept in practice.
- Resilience-based design should be a stakeholder engagement process. The performance targets of new designs, retrofits, and mitigation actions for existing assets need to be co-developed with stakeholders, considering budget, timeline, regulation, and policy requirements holistically.
- In risk and resilience work, there are more opportunities to work directly with stakeholders. This can potentially lead to more resilience-based design work, incorporating risk mitigation as part of the risk and resilience consulting offers.

Other aspects of Resilience

- Climate resilience is a growing interest for both private and public sectors. This is due to the design life of their assets and the potential for an increase in frequency or severity of natural hazards affected by a changing climate over time, such as floods, heat events, hurricanes, and wildfires, which have significant implications due to climate change.
- Multi-hazard consideration is crucial for resilience. While earthquakes have been studied more due to their cascading effects and building code developments, other hazards could potentially have a greater impact. Considering resilience from a multi-hazard perspective is important and may require not just structural engineers but also other disciplines (such as mechanical engineers, climate scientists, geophysical and hydrologic scientists, economists, among others) in asset planning and management and building design.
- Achieving structural resilience through enhanced structural design is an important step but might not be sufficient in certain cases when operational resilience is important. Sometimes resilience can be compromised even without significant physical damage. For example, during extreme heat events or wildfire smoke events, staff may not be able to work, significantly compromising operational resilience.
- The resilience of an asset is always influenced by infrastructure resilience. It is quite often that asset functional downtime is controlled by the loss of site access or utilities during a natural hazard event. While public utility assets are hard to consider as part of the asset design or evaluation, on-site utility connection points and components are important to consider in design and resilience planning.

Summary

Risk and resilience assessments have seen increasing demand over the past decade as stakeholders strive to understand the risks to their assets and practical ways to achieve better resilience through mitigations. This typically begins with a risk assessment to identify key risk drivers and vulnerabilities, followed by designing mitigations to enhance resilience. Depending on the applications and the needs of stakeholders, the scale and method of risk and resilience assessments can vary.

A universal taxonomy for risk and resilience assessments has been developed to establish a framework for determining what levels of risk assessment should be undertaken to make resilience-related decisions or take actions with sufficient confidence to increase resilience. It provides a reference for both stakeholders and practitioners to better understand the level of risk and resilience assessments that should be conducted for various resilience applications.

While structural resilience-based design is an important step towards resilience, engineers should think beyond the design work and collaborate more closely with stakeholders. This holistic approach ensures that stakeholders are engaged in the resilience journey and that their needs are fully addressed.

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IMPROVING THE SEISMIC RESILIENCE OF NEW ZEALAND HOSPITAL BUILDINGS

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Abstract

The establishment of Health New Zealand | Te Whatu Ora in 2022 has enabled more direct national oversight of public hospital buildings across New Zealand. This has also led to the need for a more comprehensive understanding of the seismic status of hospital buildings, and what is needed to address their vulnerabilities.

Following a 2022 independent report which highlighted key areas of the vulnerability of hospitals to earthquakes, a national seismic work programme was established by Health NZ. This included the establishment of a Health Engineering Advisory Group to develop technical guidance for the design of new hospital buildings, with a particular focus on the post-earthquake functionality of non-structural elements, infrastructure systems and specialist medical contents. Part of this work has involved clarifying Importance Level categorizations, and the functional recovery requirements applicable to the various medical functions delivered in hospital buildings.

Another key workstream has focused on improving the post-earthquake response of hospitals by establishing Priority Response Agreements between engineering practices and the hospitals. These agreements document the understanding that nominated locally-based engineers familiar with the buildings will give the hospitals their priority as they respond following a major earthquake.

This paper, authored by members of the Health Engineering Advisory Group, outlines the Health NZ seismic work programme and progress made over the past two years to improve the seismic resilience of New Zealand's hospitals. The key features of the draft technical guidance for the design of hospital buildings, which builds upon the Australasian Health Facilities Guidelines, are outlined.

Introduction

A number of public hospital buildings across New Zealand were constructed in eras that preceded the advent of modern seismic codes. Some more modern hospital buildings have also been found to contain design shortcomings that were highlighted by the Canterbury and Kaikoura earthquakes and more recent advances in engineering knowledge. Seismic assessments have been undertaken for the majority of hospital buildings over the past decade, but not all. Some key hospital buildings have been found to have low seismic ratings, with some receiving earthquake prone building notices from their local territorial authority.

In July 2022 a Crown entity, Health New Zealand - Te Whatu Ora (Health NZ) was created to take over the planning and commissioning of services and the functions of the previous 20 District Health Boards to remove duplication and provide more effective national planning.

The status of seismic information across New Zealand's hospitals was summarized in a report by Kestrel Group in 2022 (Kestrel, 2022). Key issues addressed in the report included the interpretation of Importance Levels, approaches to evaluating non-structural elements and the components of seismic information that should be included in business cases, and the need for technical guidance for new and

existing hospital buildings. The report emphasized the vulnerability of non-structural elements (such as ceilings, partition walls, building services, pipe runs and heavy specialist medical equipment), and the criticality of their continued operation to enable hospital buildings to deliver acute services following an earthquake. It highlighted the lack of clear design criteria for non-structural elements in new hospital buildings, and that the adequacy of the seismic restraint of non-structural elements in existing hospital buildings had rarely been assessed.

This report contained a number of recommendations to better understand and address the vulnerability of hospital buildings, and these formed the basis of a seismic work programme initiated by Health NZ.

This paper summarises the Health NZ seismic work programme and the recently completed technical guidance for the seismic design of new hospital buildings. The establishment of Priority Response Agreements to enable the rapid response of local engineers to hospitals following major earthquakes is also outlined.

The Health New Zealand Seismic Work Programme

A seismic work programme was established by Health NZ in 2022 to support and implement the recommendations in the Kestrel report, with the following priorities:

1. Updating the seismic information held about all public hospital buildings, including obtaining information about those buildings not yet assessed, and developing an approach for the prioritisation of mitigation work
2. Preparing a Seismic Policy and Seismic Risk Management Strategy to guide a uniform approach across all hospital buildings
3. Preparing technical guidelines that establish seismic and structural performance objectives for new and existing hospital buildings and associated design criteria
4. Putting in place more specific procedures and arrangements for post-earthquake response, including implementing priority agreements with engineers

A national seismic assessment panel was established at the end of 2022 to undertake further assessments, both of hospital buildings that haven't been assessed and to update earlier less reliable assessments where considered necessary. Building typologies have been established in order to focus on where assessments are required. A specialist engineering panel (the Health Engineering Advisory Group, comprising the authors of this paper) was established from the wider panel to prepare technical guidelines for designing new and assessing existing hospital buildings.

The following sections in this paper outline the output from the two key elements of this work programme as at the end of June 2024.

Health New Zealand Seismic Design Guidance

Overview. The *Technical Guidelines for the Seismic and Structural Design of Hospital Buildings* (Health NZ, 2024) is one of several design guidance notes that Health New Zealand has developed to support safe, fit-for-purpose health facilities. This document is in draft form at the time of preparing this paper, and is subject to formal approval from Health NZ prior to its release in 2025.

The Seismic Design Guidance clarifies where Health New Zealand requires designers to go beyond the requirements of the Building Code (and where they only require the Code to be met). One of the key new

areas of focus of the Guidance is to establish a seismic performance framework for new and strengthened hospital buildings, covering both *life safety* and *building functionality*.

The main driver for the Guidance has been to define technical criteria around ‘functional’, as this has never been defined in design standards or by Health New Zealand or its predecessors. Acute services buildings are required by the Building Code and AS/NZS1170.0 to be functional following a 500 year earthquake, but different interpretations of what ‘functional’ means have had cost impacts on both primary structure and non-structural elements (e.g. partitions and services restraints). Key new material provides clarification and specific criteria for designing to achieve appropriate levels of post-earthquake functionality. This involves an emphasis on non-structural elements and their impact on the design of the primary structure and building services.

While the primary focus of the Guidance is on the design of new acute services buildings, coverage is also given to the full range of building types, including extensions to existing buildings.

The Guidance provides information and requirements that relate to design process aspects. This includes for example the need for more comprehensive geotechnical information at masterplanning stage, and the requirement for a *Non-Structural Element Seismic Strategy* to be established at the concept design stage.

Classifying Hospital Building Functions and Importance Levels. The Guidance divides hospital facilities into four building categories and assigns the various clinical functions or services to each category. Each category is then associated with a particular importance level and its own functional continuity requirement.

The four categories are:

Acute Services: Acute Services and essential support services (ie. *special post-disaster functions* in accordance with AS/NZS1170.0). This includes Emergency Departments, Operating Theatres and Intensive Care Units. These are designated at Importance Level 4 (IL4), with enhanced functional continuity requirements.

Other Inpatient Facilities: Including general wards. These facilities are designated at IL3, however they have an enhanced functional continuity requirement, beyond that prescribed in the New Zealand Standards for IL3 facilities.

Other Medical Services: Including inpatient facilities, elective surgery, and non-critical medical services. These are designated at IL2. They also have a modest functional continuity requirement beyond code requirements.

Other Medical Services: Generally non-medical facilities such as offices, kitchens, and laundries. These are designated at IL2.

Seismic Performance Requirements. The seismic performance requirements in the Guidance are defined in a series of cascading levels as shown in Figure 1, starting at the broad outcome objective relating to life safety, functional continuity, and asset protection. These proceed through to more detailed requirements, progressing from general descriptors of performance goals intended to be understood by non-technical readers, to a description of likely post-earthquake physical states, which serve as pivot to the detailed engineering criteria for design engineers.

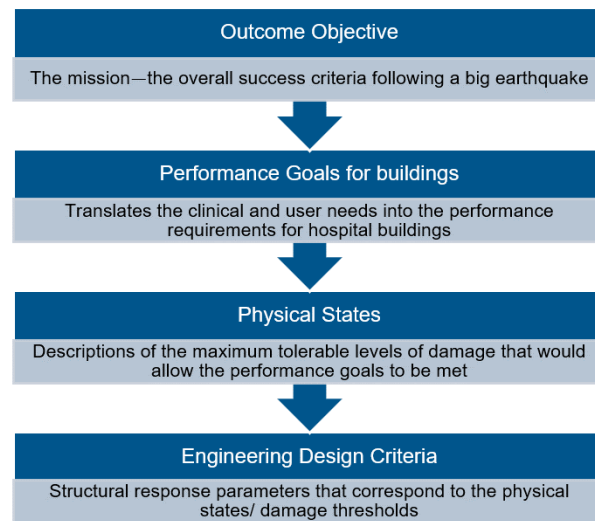


Figure 1: The cascading relationship between the overall Outcome Objective, Performance Goals, Physical States and Engineering Criteria

A key aspect of the Guidance is the descriptions of tolerable reduction in functionality, as well as anticipated timeframes for functional recovery. These are shown graphically in Figure 2 below.

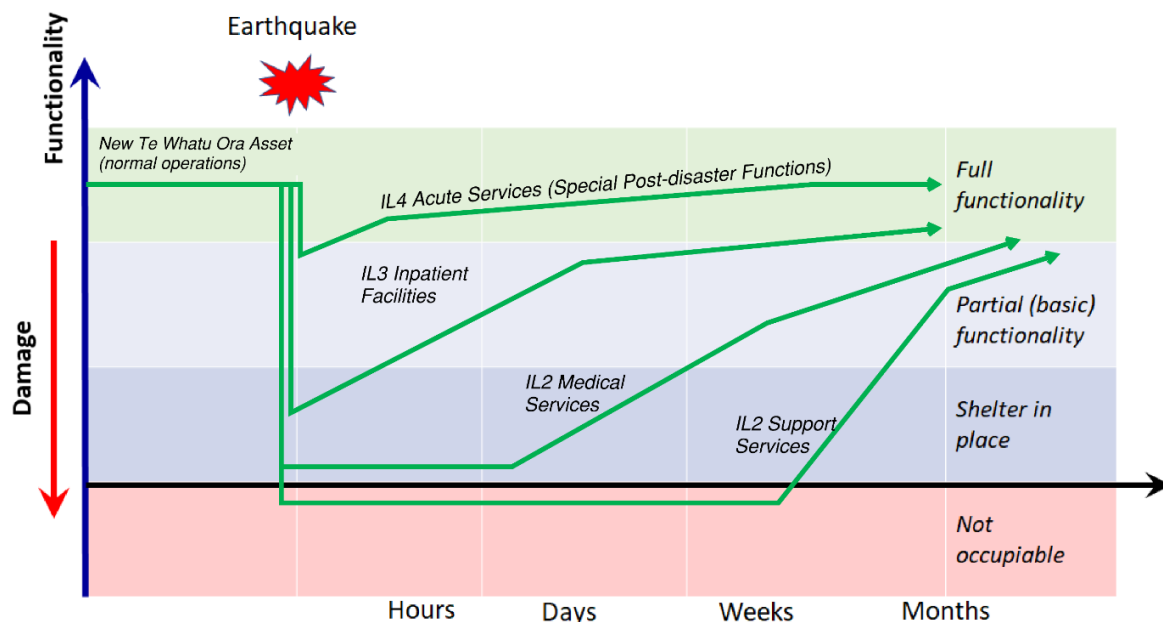


Figure 2: Tolerable reductions in functionality for Health NZ assets, and timeframes to full recovery, indicated in graphical form for different asset types (health service categories).

Design return periods are defined for each building category and limit state. A Damage Control Limit State (DCLS) is introduced to reflect the asset protection objective, alongside the SLS2 objective reflecting Continued Functionality. The return periods for DCLS and SLS2 have been aligned to streamline the engineering checks required.

Table 1: Design limit states and return periods, defining the criteria for meeting Seismic Performance Goals.

Health Service Category	Importance Level	Annual Probability of Exceedance for Seismic Hazard			
		Damage avoidance (also implying unimpeded continued use)	Asset protection objective	Continued Functionality objective	Life Safety objective
		SLS1	DCLS	SLS2	ULS
Acute Services	IL4	1 / 25	1 / 500		1 / 2500
Inpatient Wards	IL3	1 / 25	1 / 250		1 / 1000
Other Services - Medical	IL2	1 / 25	1 / 250		1 / 500
Other Services - Support			1 / 250	n/a	

Structural and Geotechnical Requirements. A wide range of detailed engineering requirements are covered within the Guidance. These address both process-related matters, such as roles and responsibilities and documentation requirements, as well as technical requirements.

Many of the process-related matters are intended to ensure that current good practice is undertaken early in the design process, where it can be most effective. These include:

- Incorporating engineering information into the master-planning phase. This includes early geotechnical input, consideration of the engineering implications of altering or refurbishing buildings, and considering campus-wide earthquake resilience.
- Commentary regarding site-wide resilience, including approaches for considering building adjacencies, and interdependencies between various types of infrastructure.
- Specific documentation requirements relating to Design Features Reports, Movement Acceleration and Loading Reports, Non-Structural Element Strategies, and Geotechnical Reporting.
- Requirements to consider and present a range of structural options.
- Expectations for structural engineers to report on embodied carbon content in the structure.

Important technical requirements include:

- Definitions of design return periods, including elevated requirements for SLS2/DCLS limit states.
- Specific robustness requirements, including reference to *Earthquake Design for Uncertainty* (NZSEE, 2022) and *Interim Design Guidance of Conventional Structural Systems* (SESOC, 2022).
- For durability purposes, a Specified Intended Design Life of 100 years is required, with particular emphasis on areas which cannot easily be accessed for maintenance. A 50 year design life is applied for deriving seismic design loadings.

- Clarifications regarding structural performance in fire, emphasizing the requirement for post-earthquake risk management (rather than compliance) for fire safety systems, and acknowledging that the existence of managed emergency procedures in hospitals can be used as part of the risk mitigation.
- Geotechnical clarifications including encouragement for settlement tolerant design, appropriate consideration of repairability and risk-based foundation selection.
- Damage control recommendations, which are ultimately expected to align with the SESOC *Low Damage Seismic Design Guidelines* (in preparation; previewed in Sullivan et al, 2024).

Non-structural Elements. A significant portion of the Guidance is focus on non-structural elements, as these are crucial to the ongoing operational continuity of a hospital, and because seismic design information for non-structural elements has until recently been somewhat limited. The opportunity has therefore been taken to capture industry best practice.

The Guidance defines a number of process-related elements for the design of non-structural elements that focus on roles and responsibilities, including:

- Requiring the implementation of a *Non-Structural Element Seismic Design Strategy*, beginning in concept design. This is to be led by the seismic designer, with active input required from other design disciplines.
- Providing a process to facilitate load coordination between structural engineers and designers of non-structural elements, with associated expectations for structural system robustness.

The technical requirements for non-structural elements are set out on an element-by-element basis. Topics covered include:

- Appropriate ductility limits to avoid undue damage.
- Strategies for managing movement and displacement incompatibility.
- Advice around concrete anchor selection, including appropriate use of C1/C2 categorization.
- Recommended drift limits for partitions and façades to avoid excessive damage.
- Requirements for validation and testing of ceiling systems.
- Enhanced design requirements for elements that could cause significant consequential disruption such as liquid containing tank or systems, hazardous systems, and vertical transportation.

Recent advances in establishing more consistent approaches to the design of non-structural elements in New Zealand are summarized in an accompanying workshop paper (Stanway et al, 2024).

Priority Arrangements with Engineers for Post-earthquake Response

Background. It is vital that decisions on which hospital buildings can continue to be used following damaging earthquakes are made quickly. A key component of the decision-making process is timely input from experienced engineers that are familiar with the buildings.

The devastating earthquake and subsequent aftershocks that affected a significant part of Turkey and Syria on 6 February 2023 provided further examples of the consequences of major urban hospitals not having direct technical support from local engineers capable of rapid response (EERI, 2023). Recommendations in the EERI report included the need to have on-call experienced structural engineers, preferably those familiar with the facility, available by phone/video link for consultation on post-earthquake safety assessments immediately after the event.

A decision to either continue to deliver medical services in a damaged building or evacuate to an alternative facility is a significant one that needs to take into account a number of clinical and functional considerations, with compromises inevitably being required. Well-focused and early input from structural engineers that are familiar with the buildings is a key aspect of hospital emergency planning. It is therefore essential that specific arrangements are in place with engineering consultants to respond to any earthquake event as required. The specific response expectations and mechanisms need to be clearly mapped out, including outline inspection plans and the nature of initial reporting. The response arrangements for the engineers should be integrated within hospital emergency plans, with associated annual 'readiness' activities to ensure that the arrangements are up to date.

Outline Structure of Hospital Priority Response Agreements. A Priority Response Agreement (PRA) between engineering practices and operators of critical facilities documents the understanding that nominated locally-based engineers familiar with the buildings will give operators their priority as they respond following a major earthquake. The agreements nominate three engineers who are local to the hospital and specific representatives from the hospital Emergency Planning and Property/ Facilities Management departments. While three is a somewhat arbitrary number of engineers, it provides reasonable assurance that at least one engineer familiar with the site and buildings will be able to attend within a reasonable period of time, along with the wider backing of the practice. The national set of engineering practices that are party to the centrally-organised set of agreements also provides what is effectively a national support network.

The PRAs are a best endeavours agreement, having due regard to the range of personal situations that the nominated personnel may face after a major earthquake. When able to respond, the nominated locally-based engineers will give the hospital their priority with the full backing of their engineering practice. The engineering consultancies were selected based on their proximity to and familiarity with the hospitals. Some of the larger hospitals have teams from two engineering consultancies named in their PRA.

Rapid Assessment Plans. *Rapid Assessment Plans* are developed for each hospital, and set out the details of:

1. How post-earthquake rapid assessments will be undertaken for key hospital buildings; and
2. How the findings for each building will be conveyed to hospital incident management

Rapid Assessment Plans are structured in two sections:

- **Part A** provides information to help with the preparation and maintenance of the Rapid Assessment Plan. It is intended to assist prior to an earthquake.
- **Part B** is the Rapid Assessment Plan itself. It is intended for use on the ground during and immediately after an earthquake.

Information relating to Response Activation, Response Procedure and Occupancy and Evacuation Decisions is standard across all hospitals.

Hospital-specific information is provided under the following sub-headings - *Safety Information; Contact Details; Onsite Communication; Meeting Point; Priority Building and Default Inspection Priority; Building Information and Available Seismic Instrumentation.*

With respect to activation, the Hospital Rapid Assessment Plans are to be immediately activated in response to any earthquake large enough to cause concern to either the hospital team or the nominated engineers. If in doubt, nominated engineers should begin making their way to the hospital meeting point and call the Duty Service Person on the way. In more moderate earthquakes that result in only nominal damage, there is still high value in the reassurance provided by responding engineers. The ability of modest events to serve as a 'live drill' for communication protocols is also considered beneficial.

In practice, the creation of the Rapid Assessment Plans also serves as a catalyst for the consolidation of critical information. Most notably, this involves the collation of existing documentation in a readily accessible form, and agreement within the hospital as to which individuals will serve as the primary interface to the engineering team.

It is emphasized that in an emergency situation, it is the Hospital Incident Controller's role (in consultation with other hospital stakeholders, and considering ongoing technical advice) to make decisions about evacuation or continued occupancy of a building. Input from engineers is inevitably influential, but engineers are not the decision-makers in this situation.

Summary

The Health New Zealand seismic work programme for hospital buildings established in 2022 covers new building design, the assessment of existing buildings and the enhancement of post-earthquake response arrangements. It has been overseen by a group of New Zealand's leading earthquake engineers with extensive experience with new and existing hospital buildings.

A key output from this programme has been the Seismic Design Guidance. This document establishes a seismic performance framework for new and strengthened hospital buildings, covering both life safety and building functionality. One of the key areas of focus of the Guidance is to provide more specific criteria to meet the serviceability limit state (SLS2) requirements in AS/NZS1170.0 for hospital buildings that house acute services to ensure functionality following a 500 year earthquake, as this has not been fully defined in design standards or by Health New Zealand or its predecessors.

Timely input from experienced engineers that are familiar with the buildings is a key component of the post-earthquake decision-making process for critical facilities operators. This is particularly the case for hospital buildings, where decisions to continue to deliver medical services in a damaged building or evacuate to an alternative facility are significant ones. Major earthquakes around the world continue to highlight the life safety risks associated with transferring patients in such situations.

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STRUCTURAL DESIGN AND CONSTRUCTION OF A BASEBALL STADIUM WITH A RETRACTABLE ROOF AND A SEISMIC ISOLATION ROOF

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Abstract

In a cold and snowy region of Japan, 35,000-capacity baseball stadium with a retractable roof and natural grass field has been constructed. The roof, covering the field and the seats, needs to have approximately 160m span and 70m internal height. The 167m-span retractable roof were designed to grow natural grass and shield the harsh environment. The retractable roof was designed in a gable shape like traditional roof shapes in this region. Its structure made effective use of this shape with small height upside-down V-shape trusses whose thrust forces were supported by substructures. This system allows us to reduce the building height and to maintain the 70m-high interior volume simultaneously. The retractable roof moves horizontally in 132m on reinforced concrete track-girders by 24 traveling bogies. The bogies were tilted to support not only vertical loads but also thrust forces. Three-hinge trusses make it possible to run smoothly on the track-girders and accommodate thermal movements.

The other 159m-span roof over the seats adopted seismic isolation to enhance earthquake safety of hanged facilities and accommodate thermal expansion and contraction. In the construction, the sliding construction method was used for both two long-span roofs to aim to reduce the construction period. This construction method seemed very efficient, but we needed to solve how we constructed the large cantilevers in this consequent process. We developed a new construction procedure that focuses on vertical and horizontal deformations of each construction step and confirmed that the actual deformations were almost the same as the structural analysis.

Introduction

ES CON FIELD HOKKAIDO is a professional baseball stadium with a seating capacity of 35,000, located in a cold and snowy area on the north side of Japan. To reduce the burden on players during play, the client requested a natural grass field. To grow natural grass, sunlight and wind are necessary, but for the sake of player and spectator environment, it is preferable to avoid harsh cold and snow by indoor environment. To solve both issues, a retractable roof and a glass wall are designed. During winter, the retractable roof is closed to maintain indoor environment, and as the warmer season approaches, the roof is opened to let in sunlight and wind. The retractable roof is designed to simplify the system and reduced malfunctions, with a horizontal one-way opening and closing format that prioritizes maintenance



Figure 1. Exterior view in winter

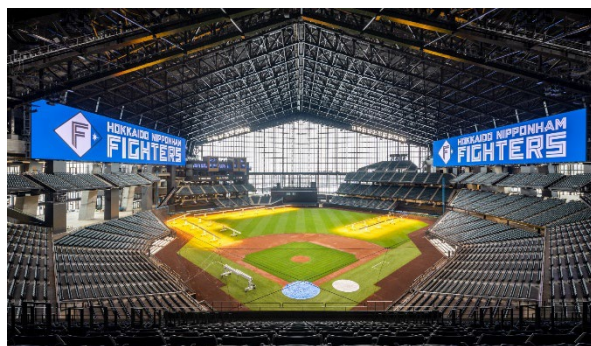


Figure 2. Interior view of southeast side



Figure 3. Panoramic view

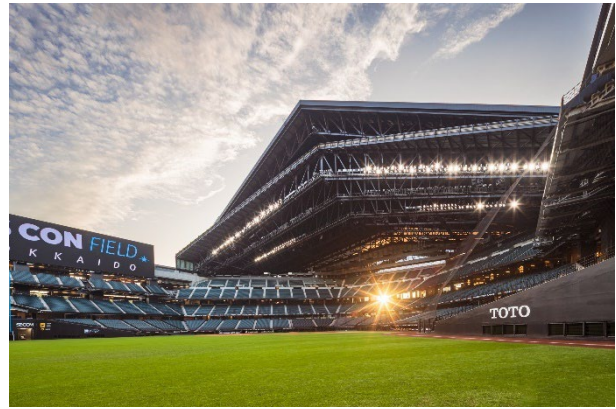


Figure 4. Interior view of northwest side



Figure 5. Panoramic view

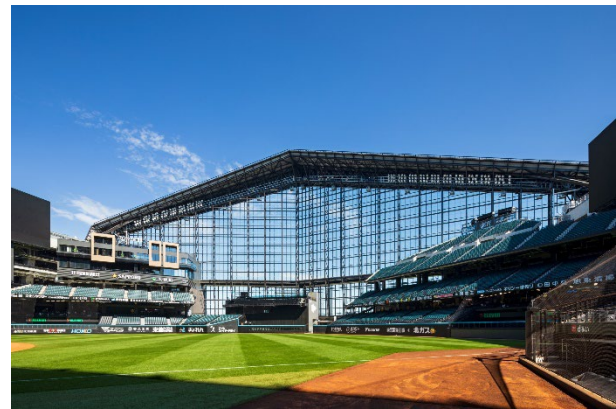


Figure 6. Interior view of glass wall

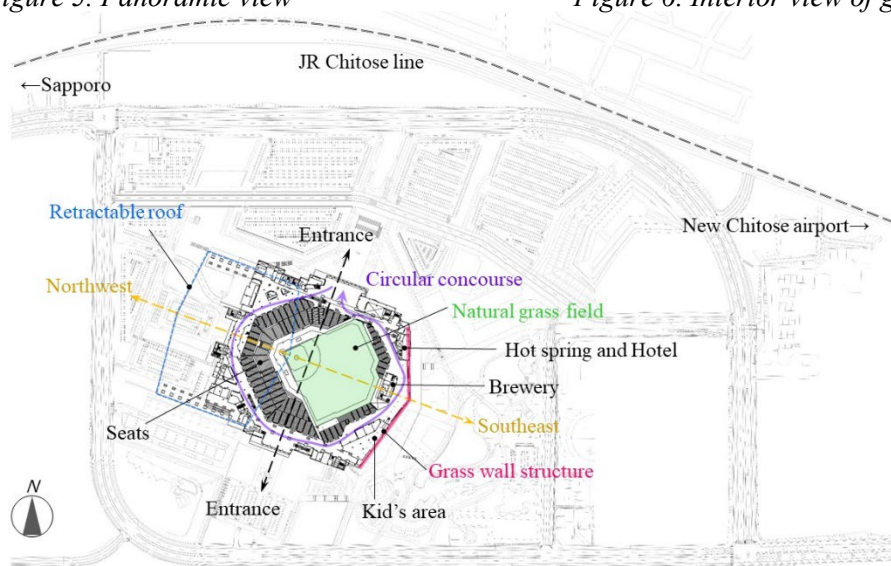


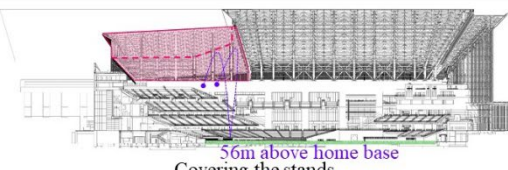
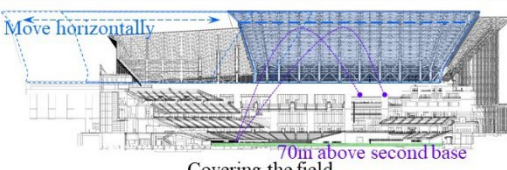
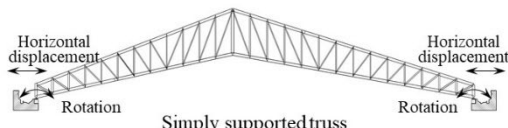
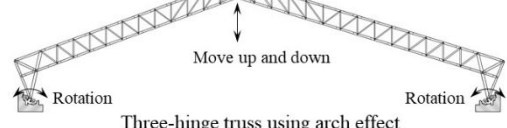
Figure 7. Site plan

performance. The retractable roof and the glass wall not only bring sunlight to the natural turf but also create a sense of unity with the surrounding area and contribute to the entertainment aspect by providing diverse viewing environments. The roof is designed in the traditional gable roof style that has been used in this region. On the left side facing the field, there are a hot spring and a hotel. On the right side is a kids' area where people can gather even when there are no games. The center area between these two

areas is a batter's eye which houses a beer brewery. The area surrounding the stadium, named F Village, has also seen the development of residential complexes, nursing homes, hotels, and other projects involving public and private partnerships to advance city planning and community-building efforts beyond just the stadium.

Overview of structure plan

This stadium has two large-scale roofs: the fixed roof covering the stands behind the backstop and the retractable roof covering the field. Both roofs use one-way steel truss beams to achieve a large span and reduce weight, but the way the force is handled at the supports differs depending on the given conditions of each roof. The retractable roof can be moved horizontally by motorized bogies at the bottom of the truss frames. The thrust force of the retractable roof is handled by the track-girders through the bogie. The track-girders supporting the traveling path of the bogies is made of reinforced concrete with high rigidity to ensure more stable running of the roof. The structural safety of the track-girders is improved by tilting the outer columns for bending stress reduction due to truss effect and introducing prestresses to control

	Fixed roof	Retractable roof
Location	 56m above home base Covering the stands	 70m above second base Covering the field
Structure	- Relieve thermal stress - Natural periods do not change due to snow accumulation - Absorbs rotational deformation of legs  Simply supported truss	- Relieve thermal stress - Accommodate track-girder displacements during running - Accommodate track-girder displacements during earthquake  Three-hinge truss using arch effect
T r u s s	10m~16m × 7trusses	6.5m × 12trusses
S p a n	159m	166m
Support	Spherical sliding bearings : seismic isolations that release thrust	Span direction : Support thrust with track-girders Travelling direction : Seismic isolation by motorized bogies

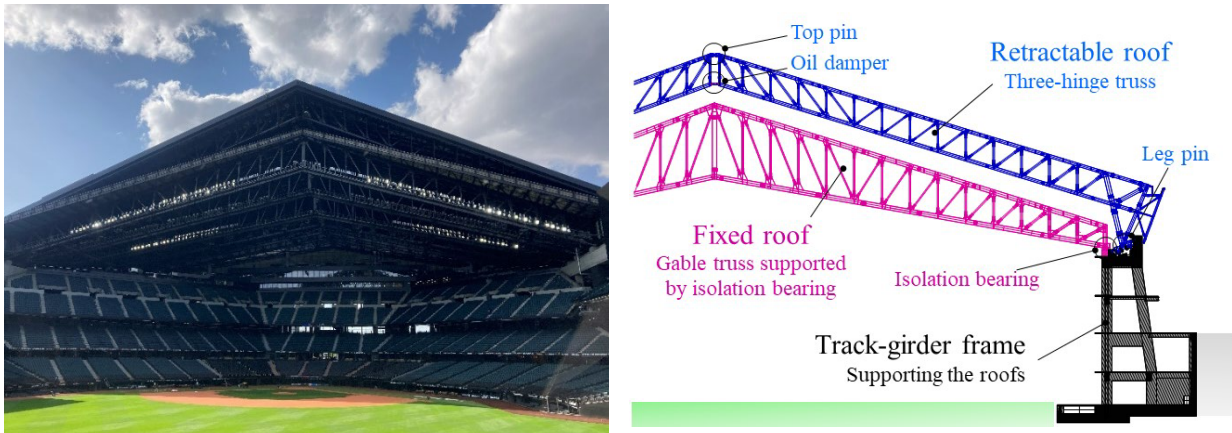


Figure 8. Fixed roof structure and Retractable roof structure

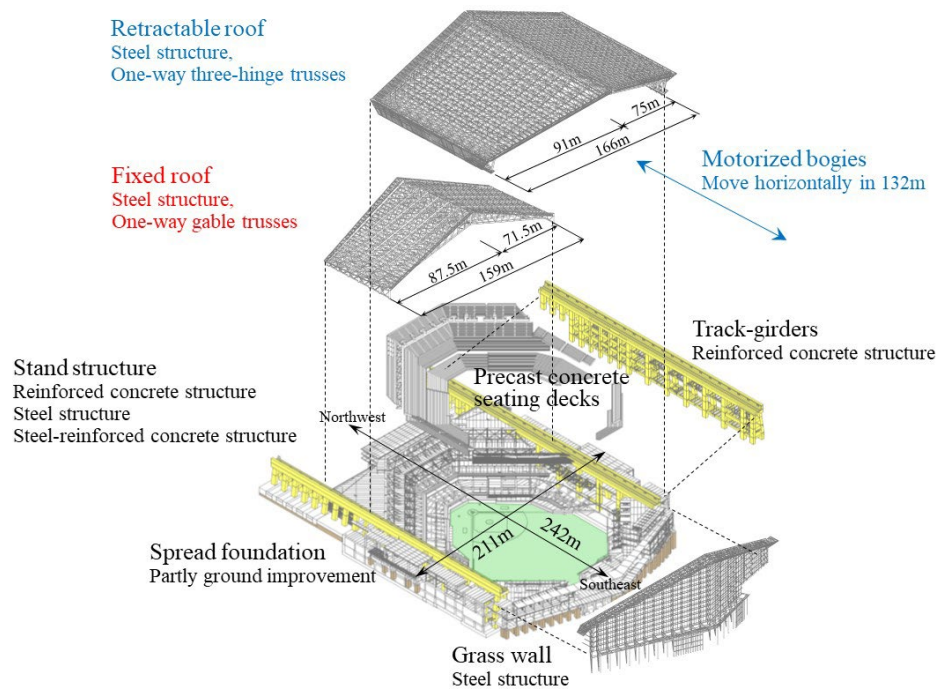


Figure 9. Overview of structural plan

cracks. The foundation is a spread foundation with heavy mat slabs to resist the thrust force of the retractable roof. The fixed roof is also supported by girders. The fixed roof is a seismic isolation roof, which releases thrust forces. The glass wall structure located on the southeast side of the stadium was designed with thin steel frames to allow for sunlight to promote the growth of natural grass.

The retractable roof

The retractable roof has a span of 166m and a horizontal projection area of approximately 25,000m². The weight is about 10,000t including finishing materials, but this double to over 20,000t when 130cm of snow, which is expected to occur every 50 years, accumulates on the roof. The retractable roof over the field is composed of one-way truss beams that utilize the arch effect of axial forces to reduce the height of the building while maintaining the interior space required for baseball. The thrust forces at both ends of the truss beams are supported by track-girders. By adopting a three-hinge structure with pin connections not only at the support points of the truss bottoms but also at the top, the structure can transmit stress through the arch effect while absorbing expansion and contraction due to temperature changes by changing the height of the roof's top. It is also effective in geometrically stabilizing the track reaction to the deformation during the opening and closing. Oil-dampers are placed on the lower chords of the truss beams to protect the pin joints at the roof tops, which is a key part of the three-hinge structure. These oil-dampers reduce the stress on the pin joints against external disturbances such as wind and earthquakes. We have developed a fail-safe mechanism that incorporates a specially designed pin shape and ensures that even if the pin shaft is broken, the roof will not collapse. The retractable roof is composed of 12 truss beams, which are supported by 24 motorized bogies at both ends. The bogies are tilted in the direction of the combined force of the roof's gravity and thrust, simplifying the structure. By providing power to the vertical wheel that receives the resultant force, it is possible to move a distance of approximately 132 meters in 25 minutes. The three-hinge parts were manufactured with a precision of 1 / 100mm to eliminate looseness in the joints, and further improve the accuracy of straightness in rail laying to reduce the driving power required to move the roof to about 1% of its weight.

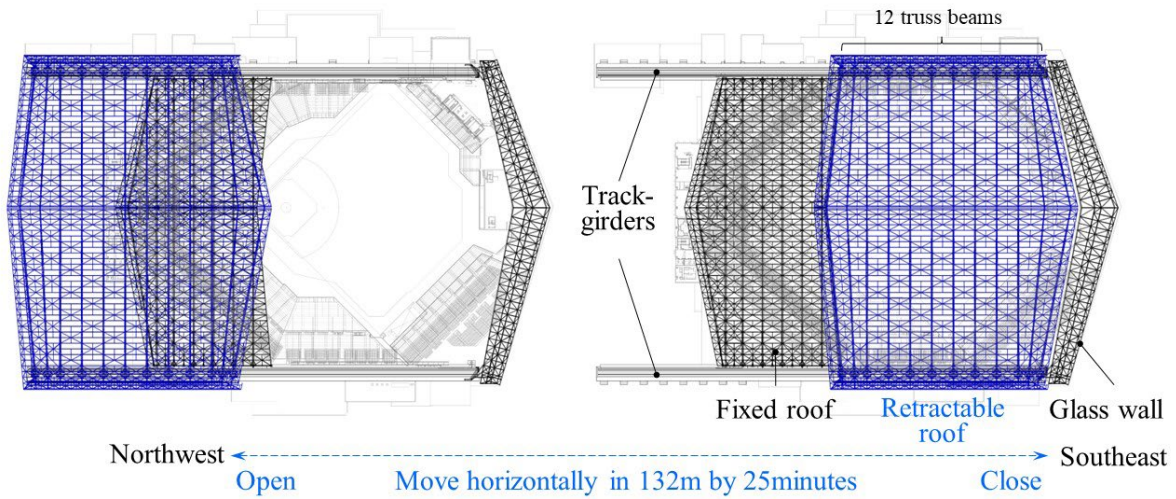


Figure 10. Opened/Closed retractable roof

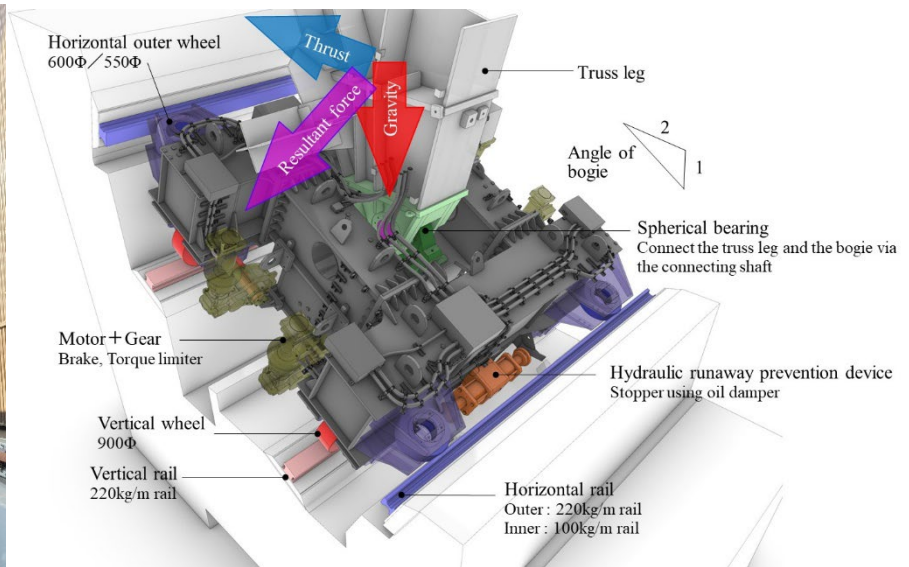
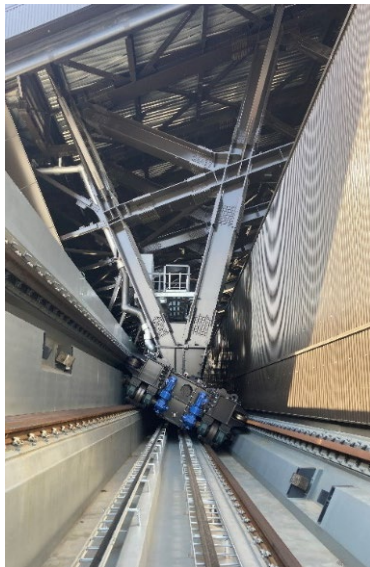


Figure 11. Motorized bogie

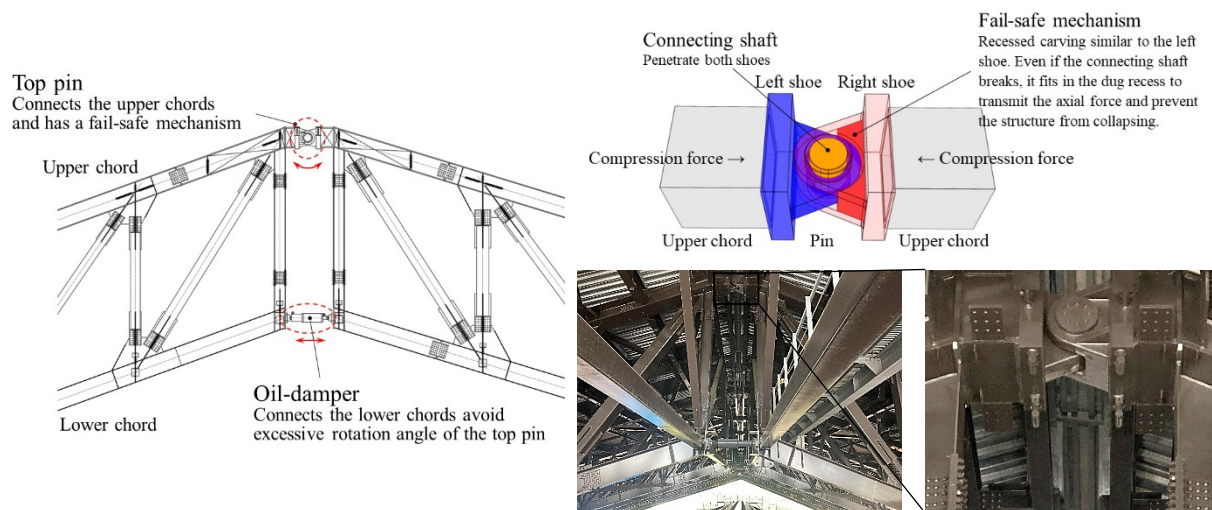


Figure 12. Details at top of retractable roof

The seismic isolated roof

The fixed roof covering the seats is also a large roof with a span of approximately 159m. The plan shape has a large cantilever only on the northwest side, and the center of gravity of the roof is offset from the arrangement of the supports. By altering the stiffness of truss beams arranged in one direction, the impact of the center of gravity shift is balanced out. The heights of truss beams at the ridge section are designed in 10m, 13m, and 16m from the field side, to ensure compatibility with the necessary volume for baseball. The fixed roof adopts seismic isolation to enhance seismic safety and accommodate the thermal expansion and contraction. Compared to non-isolated roofs, the seismic response is reduced by about 20%, significantly reducing the risk of falling for lighting and sound equipment attached to the roof. By using spherical sliding bearings as seismic isolation devices, it is possible to absorb the rotation angle of the column legs in large-span structures. In addition, even if the load doubles due to snow load, the natural period of the roof remains unchanged to maintain the seismic isolation function. U-shaped steel dampers are used as triggers for wind and absorb energy during earthquakes.

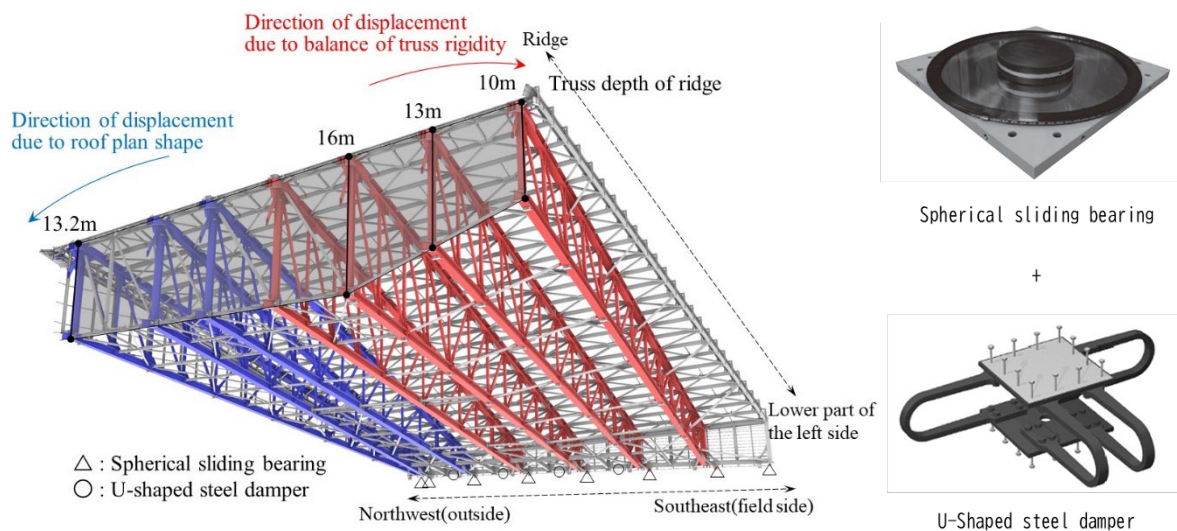
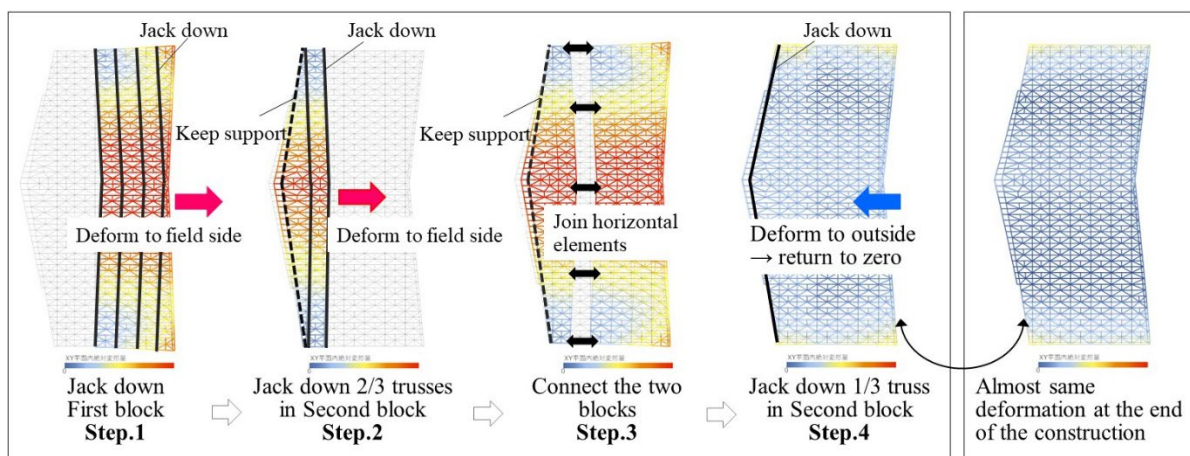


Figure 13. Fixed roof structure

Innovative sliding construction for the roofs

In order to strictly adhere to the 32-month construction schedule for opening in spring 2023, a rational plan that combined design and construction was developed from the design stage. We used the sliding construction method both the retractable roof and the fixed roof. The roof was divided into blocks and each block was constructed on vents placed outside the building and on top of the previously constructed building, and then sent out one by one to the field side, allowing the construction of the stands and roof to proceed simultaneously. The fixed roof is divided into two blocks. The fixed roof was not able to be simply connected by jacking down each construction block due to its seven different truss heights and a large overhang on the northwest side, which caused deformation in the span perpendicular direction. Therefore, we developed the following construction steps based on structural analysis.

- 1) Jack down the first block. The roof is deformed towards. In addition, the stability against earthquakes during construction was also considered, and the first block size is determined as large as possible.
- 2) Due to the large cantilever on the northwest side of the second block, when jack down, it falls over. Therefore, by jacking down only two of the three trusses on the field side, the roof is jacked down with a deformed shape on the field side.
- 3) When the deformation of these two blocks becomes almost equal, install the horizontal braces and the secondary to connect the two blocks.
- 4) Jack down the last truss of the second block. Due to its large cantilever, the entire roof is deformed and tilt backwards. This causes the roof's planar deformation to almost return to zero, resulting in a roof with stress conditions almost identical to those designed as a whole.



Horizontal deformation of each construction steps
Figure 14. Construction procedure of fixed roof structure

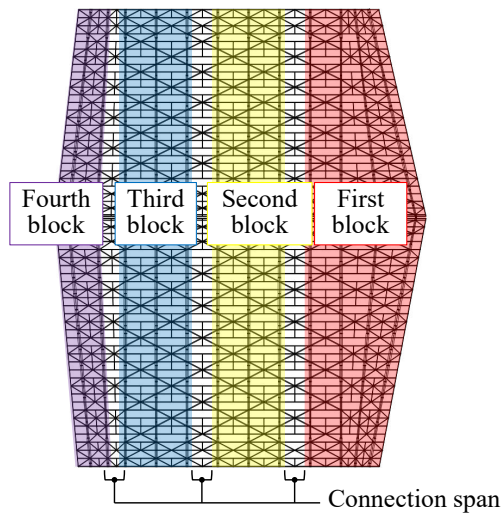


Figure 15. Construction block division of retractable roof

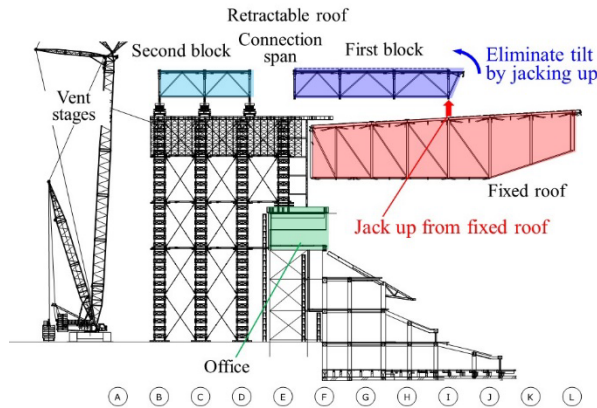
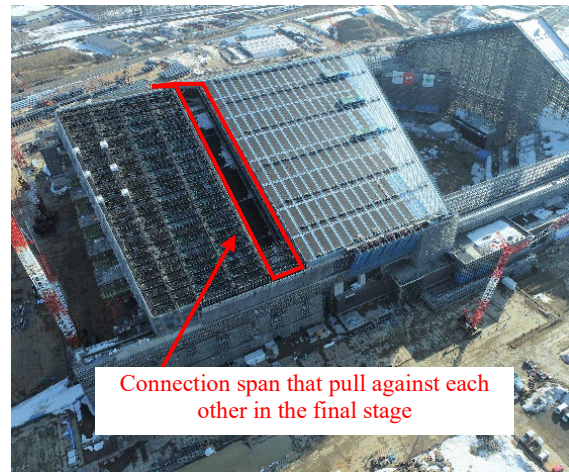


Figure 16. process of jacking up retractable roof from fixed roof



We also constructed the retractable roof which had large cantilevers by using the same construction method. The retractable roof was divided into four blocks. The effect of the overhanging parts was balanced by a construction procedure in which the two blocks on the southeast side and the two blocks on the northwest side would pull against each other in the final stage of construction. The first block was made up of blocks in the overhanging part on the southeast side, and had a tendency to displace forward. In order to eliminate the displacement perpendicular to the span when connecting to the second block, it was planned to push up the ridge part from the fixed roof with a hydraulic jack.

The displacement in construction steps and the end of the construction almost coincided with the structural analysis results successfully.

Conclusion

This report introduced the structural planning and construction of two large roofs for a natural grass baseball stadium built in a cold, snowy region of Japan. Now two seasons after opening, it is becoming an essential facility for the community.

TAMACHI TOWER ADOPTING INNOVATIVE DAMPER-CONTROLLED BUILDING SYSTEM

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Abstract

TAMACHI TOWER is high-rise steel building locating in Tokyo and adopting innovative damper-controlled building system.

System focuses on recent Japanese high-rise buildings component that occupancy is different between lower and upper level.

TAMACHI TOWER has 156 meters height with 29 stories above grade and 2 basements.

Typical office floor plans, where are assigned above level 6, is 58 x 67 meters rectangular and building core locates slight south toward floor center. Below L5, other occupancy such as retail etc. are assigned. Hence, for L2, where office entrance lobby locates, through L5 building core is a kind of void for lower plans and relatively easy to enclose with concrete core. Focusing on this, laterally sliding core arranged from B2 through L5 is maximizing damper energy dissipation.

In order to let concrete core slide, slide bearings are installed at the bottom of core in addition to assigning seismic joints between lower levels core and surrounding steel moment frame. Even though core is integrated with steel frame at L6, lateral differential displacement between core and frame accumulate downward and maximum 30 cm gap has been generated below L1. Expecting this large differential displacement viscous dampers are installed below B2 only and no viscous damper is installed above grade, whilst ordinary system expects small energy dissipation due to distributed dampers.

The innovative system has led greater disaster prevention under earthquake as well as mitigating required number of dampers to 10% of conventional system and result in providing improved floor area efficiency.

Introduction

When a Japanese tall building locates in a high seismic hazard region is designed with performance based structural design (PBSD), an efficient energy-dissipating structural system is significant to satisfy design criteria under earthquake condition. To improve building performance under earthquake, there are several options. Base-isolated or intermediate story isolated structural system is one of them and damper controlled structural system is another one. However, in the case of base-isolated structural system, excessive tensile force due to tall building overturning moment often need be addressed. Although an intermediate story isolated structural system provides better solution to mitigate such tensile force, it introduces challenges related to special vertical shaft and elevator design to accommodate large displacements at the isolation layer. Based on such reasons above, typically damper controlled structural system is adopted for Japanese tall building over 100 meters high above grade. Among damper controlled structural systems, there are also various options however, installing viscous damper on multi stories expecting single story drift under earthquake is major choice (Fig.1 left). Obviously, this is robust and reliable method having long history. But each dampers energy dissipation is not so large therefore, in order to make sure sufficient energy dissipation, large amounts of dampers are typically requested and loses floor efficiency. To address this issue, proposed system focuses on typical building component which recent mix-use building has and optimizes dampers seismic energy dissipation. This paper describes the innovative structural system having building core slide and install limited damper devices between sliding core and surrounding steel frame where large lateral differential displacement occurs under earthquake. Maximizing damper stroke under earthquake, damper efficiency is dramatically improved and system succeeds in inviting clear reduction of the number of installed damper devices in addition to optimized steel frame and sustained robust building performance under earthquake.

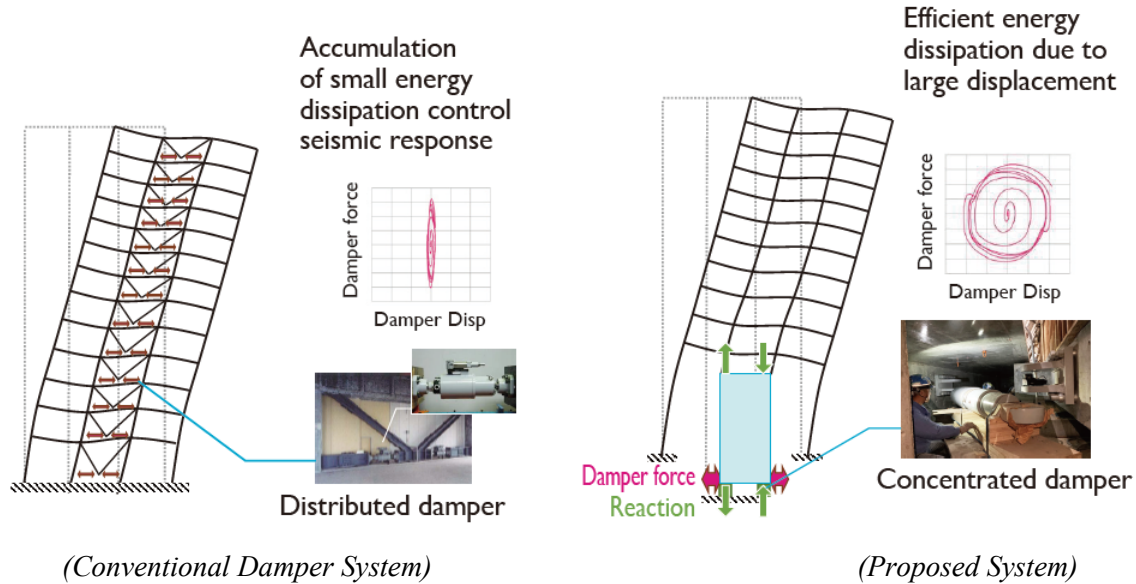


Figure 1. Comparison of damper-controlled structural systems

Building Outline

TAMACHI TOWER which adopts innovative damper system using sliding core is a new tall building locating on Tokyo Bay area. The site is an urban transportation node connecting to both JR Tamachi Station and Mita Subway Station. Although site is terminal oriented and has high potential to attracts tenant there exist a few other large-scale developments in this district. Therefore client desire the building to be as competitive as possible. To make sure competitiveness, the building aims to achieve high floor efficiency and higher building resilience after a severe earthquake, which remains a concern for central Tokyo.



Figure 2. Building Elevation

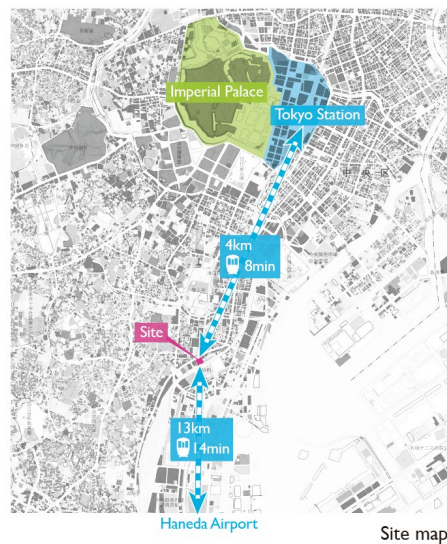


Figure 3. Building Site

TAMACHI TOWER is 156 meters high steel moment frame basis tall building having 28 stories above grade and 2 basements. Building occupancy is composed of office, retail and other occupancy where

need to manage building such as MEP floors and/or parking. Major occupancy is office locating on L6 above. L5 and B2 are MEP floors and retail locates from B1 through L4. There is also office entrance at L2 where connects to JR Tamachi station through deck above street level. Therefore, visitor to office come across L2 entrance and soon arrive in building core. Then elevator passes through retail floors and passenger reaches to L6 above directly (Fig. 4). As for plan, typical floor is 58 by 67 meters rectangular. Elevators for office floors are composed of 3 banks and assigned on slightly off from floor center. Other vertical shafts are arranged beside elevators shaft and compose a part of building core (Fig. 5). This core layout and building use are recently very typical in this scale Japanese building development.

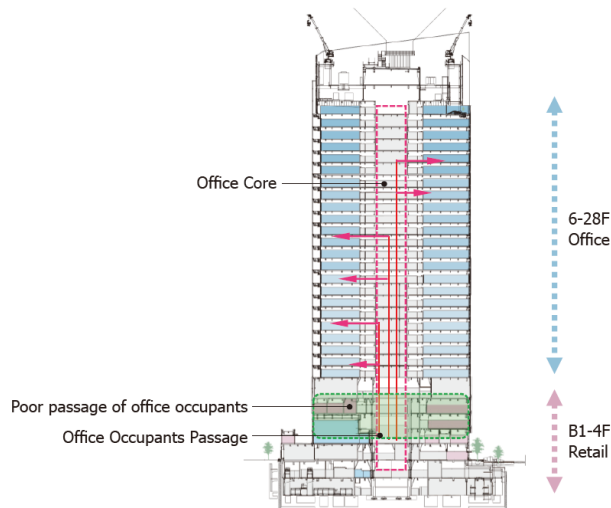


Figure 4. Building Section



Figure 5. Typical Floor Plan

Regarding seismic design procedure, Japanese building codes requests time-history analysis for over 60 meters high buildings to evaluate design earthquake load. Seismic level is divided into 2 scales, frequent earthquake and rare earthquake. Whereas it is typically said expected mean return period of rare earthquake is close to 475 years. Figure 6 shows rare earthquake design seismic acceleration response spectrum for this project, which target spectrum is defined by code and artificial ground motion records are simulated by structural engineer. When damper-controlled structure is designed, controlled inelastic deformation of the frame is permissive under rare earthquake. Therefore, typically elastic-plastic time history analysis is coordinated, and damper force including viscous dampers are directly evaluated in the time history analysis.

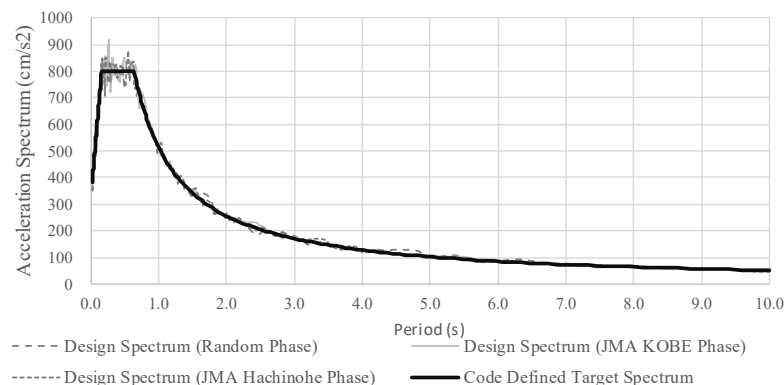


Figure 6. Design acceleration design spectrum

Table 1 describes comparison between ordinary tall buildings satisfying minimum requirements of codes and TAMACHI TOWER design criteria. When building performance under seismic is discussed in Japan, story drift limit is often adopted for criteria. For ordinary tall buildings, 1% drift under rare earthquake is defined as limit and the building satisfying 1% limit is expected to be continuous use after severe earthquake with some repairs. In other word minor damage is accepted under rare earthquake. However, TAMACHI TOWER is expected higher business continuity after earthquake. Based on engineering judgment, additional design earthquake level called Level 3 is defined as 150% amplified seismic of rare earthquake (Level 2). Under Level 3 earthquake, similar story drift limit assigned for Level 2 design criteria of ordinary tall building is set and expects mitigating building damage under Level 2 earthquake. Similar reinforced performance-based structural design is often coordinated in Japanese structural design and the upgraded design criteria is also linked to the assessment of the building's competitiveness as a tenant-focused property.

Table 1. Seismic Design Criteria (Extracted)*1

<i>Earthquake Level</i>	<i>Frequent Earthquake (Level 1)</i>	<i>Rare Earthquake*2 (Level 2)</i>	<i>Level 3 (150% amplified Level 2)</i>
Ordinary Tall Buildings	Story Drift: < 0.5 % (Frame Elastic and few damage) (Building Function Continuous)	Story Drift: < 1 % (Frame Partially Yield and Minor damage) (Building Function Recovered after repair)	Not Defined
TAMACHI TOWER	Story Drift: < 0.5 % (Frame Elastic and few damage) (Building Function Continuous)	Story Drift: < 0.75 % (Most Frame Elastic and Minor damage) (Building Function Continuous)	Story Drift*3: < 1 % (Frame Partially Yield and Minor damage) (Building Function Recovered after repair)

Notes:

*1: Descriptions in parentheses are expected building condition after earthquake

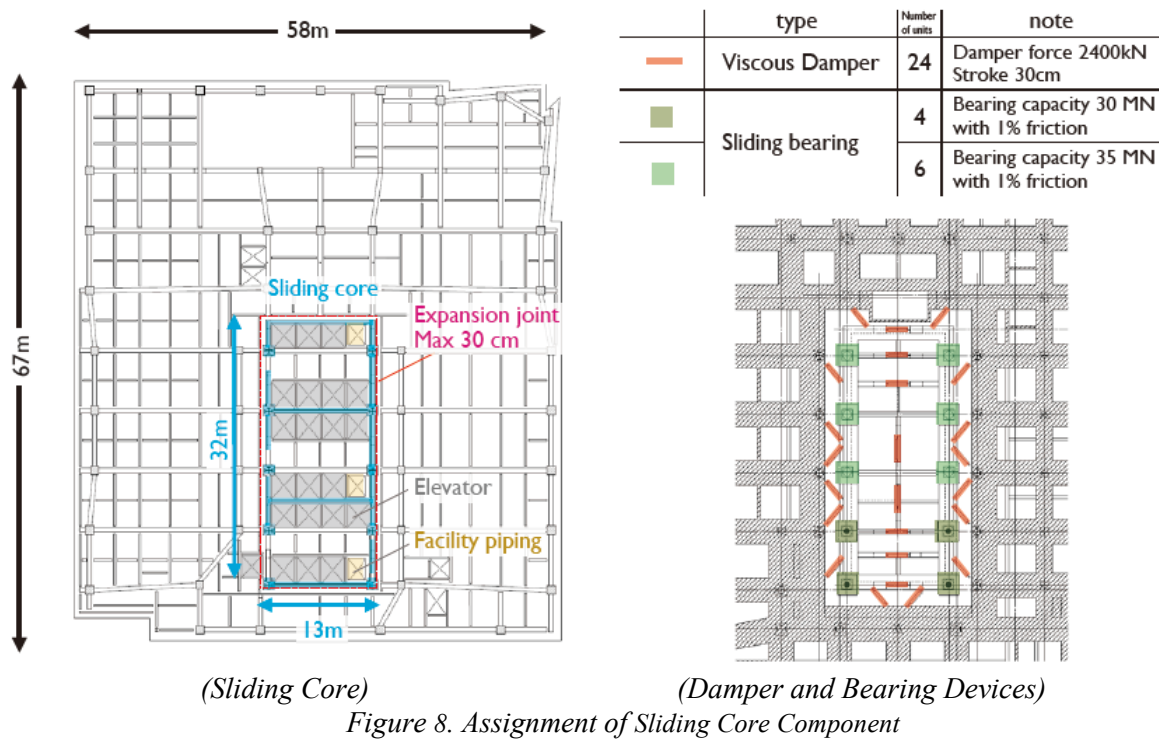
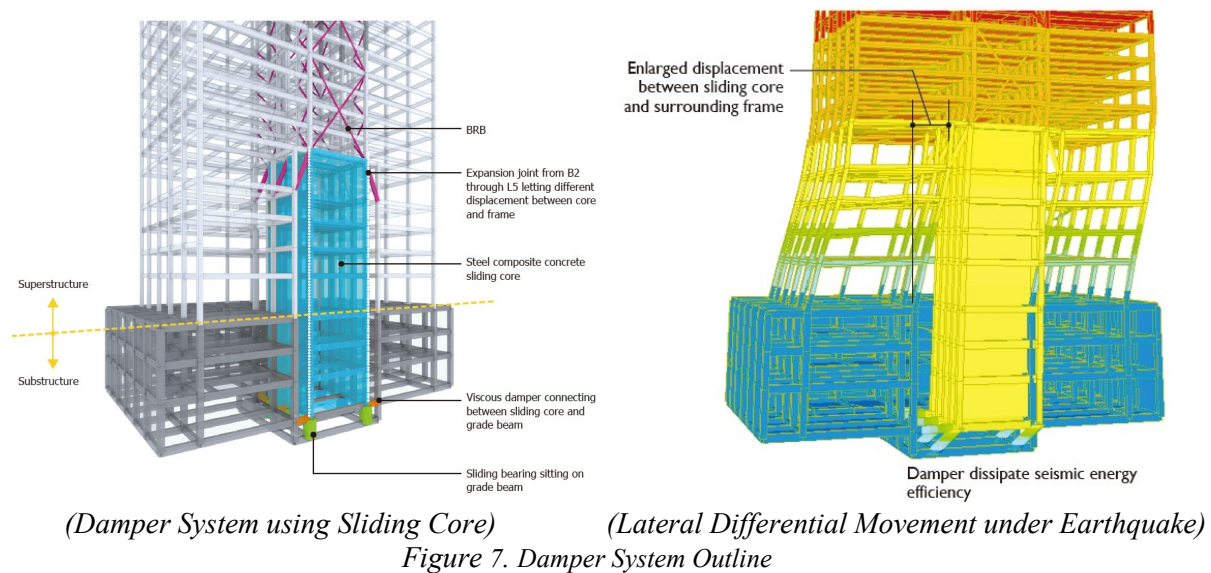
*2: Earthquake scale expected to be 475 years mean return period

*3: Shear component of story drift is set as limit.

Adopted Damper-Controlled Structural System Outline

Figure 7 shows an outline of proposed structural system. This innovative damper controlled structural system¹⁾ is developed focusing on few people across between building core for office where locates on upper levels, and lower retail. As described elevators for upper levels skip lower level, lower level of building core is easy to be closed by concrete core and is easy to install movement joints between core and surrounding steel moment frame. When this isolated concrete core sits on slide bearing, however it connects to surrounding steel frame at the top, lateral differential movement between core and frame is generated under earthquake and the displacement is amplified down-ward. To address the amplified displacement, viscous dampers are installed at the core's base to optimize energy dissipation during seismic event. Since sliding concrete core is assigned for B2 through L5 building core, both sliding bearing and viscous dampers are installed beneath B2 where the maximum lateral differential displacement is expected (Fig. 8). A clearance of 5cm to maximum 30cm between the core and surrounding frame is designed with movement joints placed at crossing points for pedestrians. Most MEP vertical shafts are assigned beside core, minimizing the need for movement joint design to enhance cost efficiency. Above L6, there are buckling resisting brace (BRB), which yielding strength is 1,000 to maximum 3000kN, are installed to gradually vary lateral stiffness from rigid concrete core to braced steel

moment frame. Without this system, large drifts would occur from L6 through L10, particularly in the few stories directly above sliding core. Additionally, L6 where sliding core and steel frame interact forces, is very significant for this system. To avoid core's over-rocking behavior which cause exceeding shear on L6 link beams, BRBs with 10,000 kN yield capacity are installed as it braces between L5 and L6 (Fig. 9).



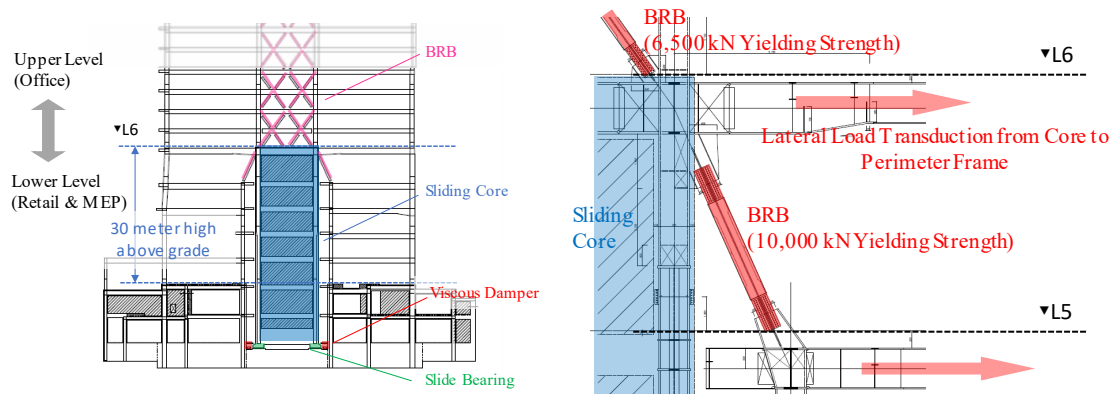


Figure 9. Section of core and frame transaction

Building Response and Damper Efficiency

Figure 10 shows time history response under Level 3 earthquake and Figure 11 shows seismic energy dissipating component under Level 3 earthquake corresponding to random phase. All maximum story drift meet design criteria. Regarding energy dissipation, approximately 35% of input seismic energy is dissipated by sliding core related devices including 24 viscous dampers having maximum 30cm stroke and including 12 low friction slide bearings, however slide bearings dissipate little energy due to its low friction.

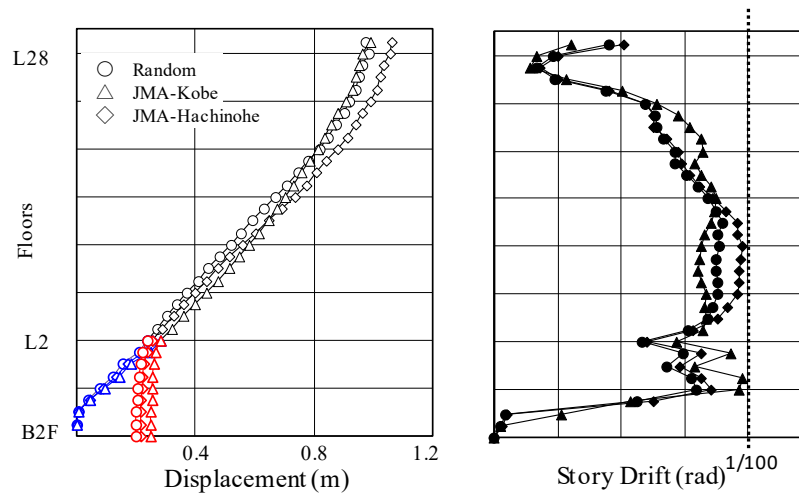


Figure 10. Time History Analysis Results

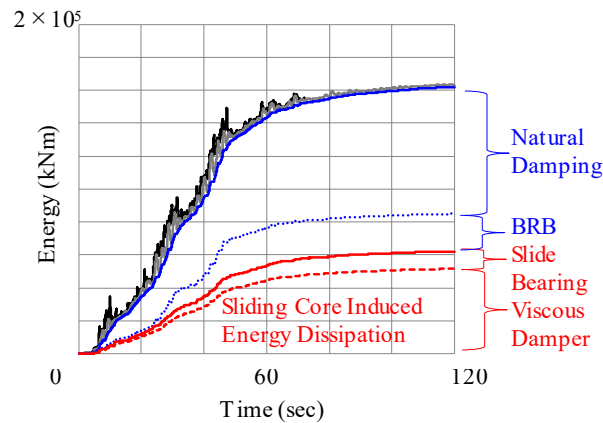
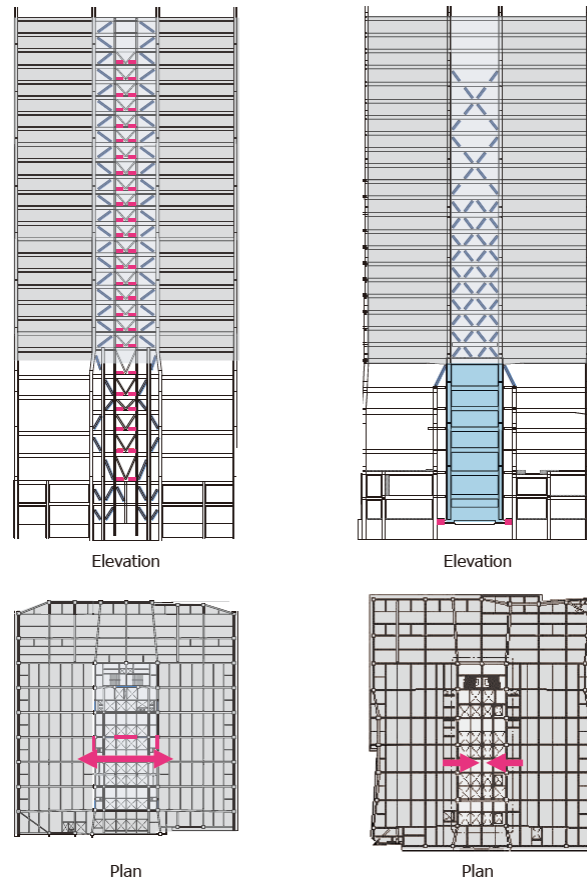


Figure 11. Energy Dissipating Component

Benefit of Innovative Sliding Core System

A comparison between conventional system and proposed system, tested in the early design stage²⁾, shows that proposed system achieves equivalent structural performance under a Level 3 earthquake with only 10% of viscous dampers required by conventional system. This optimization results in more efficient net floor area and creates additional usable space. Utilizing this buffer, additional building well-being facilities such as small kitchen at the core and semi-outdoor circumstance winter garden – an innovative feather for Japanese high-rise offices – have been incorporated (Fig.12, Fig. 13). In terms of construction cost, this innovative system shows significant savings compared to the conventional system, achieved through reductions in both steel tonnage and number of damping devices required (Table 2, Fig.14).



(Conventional System) (Sliding Core System)

Figure 12. Structural Frame Comparison



(Winter Garden)

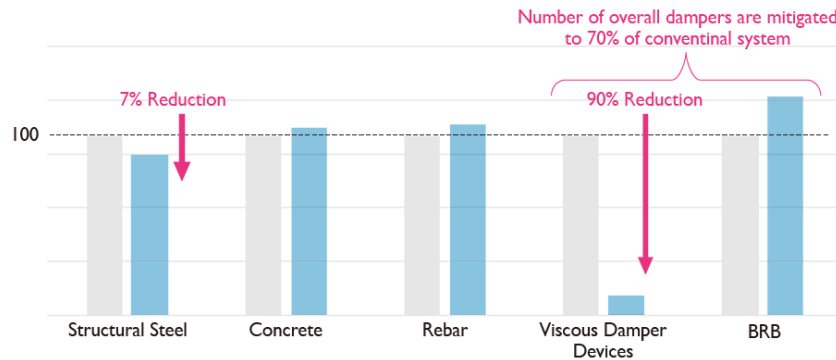
(Mini Kitchen beside Core)

Figure 13. Additional Building Function Generated by damper optimization

Table 2. Structural Comparison

<i>Structure System</i>		<i>Conventional Damper System</i>	<i>Innovative Damper System with Sliding Core</i>
Superstructure Steel Tonnage (kg/m2)		180 (100)* ¹	167 (93)* ¹
Number of Devices	Viscous Damper	220	24
	BRB	230	280
	Sliding Bearing	0	12
Damping Device Cost Index		100	60
Structural Cost Index		100	95

Notes: *1: Values in parentheses describes cost index where conventional system is assumed as 100

**Figure 14. Structural Saving**

This system also enables a more structurally efficient core wall design even in Japan. Given the high seismic hazard, Japanese buildings typically requires very stringent earthquake load designs and typically results in exceptionally thick core walls. However, this system can cap core wall design load by slide bearings induced friction and damper capacity, hence concrete core wall system can be feasible with reasonable dimensions. Replacing superstructure frame from steel to concrete can provide another benefit of embodied carbon reduction. The proposed system achieves 6.5% structural embodied carbon saving compared to studied conventional steel moment frame. The primary saving is undertaken by steel tonnage saving derived from fewer damper devices, while the steel to concrete replacement further contributes to embodied carbon savings.

Conclusion

Innovative damper system using sliding core is introduced. This system provides new possibilities to achieve reinforced structural performance under severe earthquake in addition to coping potential issue which conventional system and/or base-isolation system involves. Additionally, utilizing recent Japanese building type, i.e. vertically combined mixed use, this system can obtain further optimized damper system and provide effective devices reduction. Furthermore, this system is expected to be further developed as adopted for other Japanese typical tall buildings.

Reference

- 1) Mitsubishi Jisho Design Inc., T., 2019, “Efficient Damper System using sliding column”, Patent Number 6490862, Japan Patent Office, Tokyo, Japan, 14 pages
- 2) Taniguchi, S. et.al, 2019, “Design of high-rise building with efficient energy dissipating system using continuous column”, Summaries of technical papers of AIJ annual meeting, Architectural Institute of Japan (AIJ), Tokyo, Japan, pp.391-392

DETAILING FOR BASE CONNECTION DUCTILITY COURTESY OF REPLACEABLE ANCHORAGE

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Abstract

Column base connections are critical elements in steel structures, as they transfer the loads from the column to the foundation. In seismic design, these connections play a significant role in resisting the lateral forces generated by earthquakes. Therefore, it is essential to understand the behavior of column base connections under seismic loads and their implications on the seismic design of steel structures. This paper utilizes select results of a full-scale test program of a steel-framed modular testbed building (MTB²) tested under one, two and three degree-of-freedom base excitation at UC San Diego's Large High-Performance Outdoor Shake Table (LHPOST6) facility. The MTB² was tested with Special Moment Frames as the primary lateral force resisting system and incorporated tests with two different column baseplate detailing configurations: (1) an idealized, perfectly "fixed" base connection, and (2) a connection detailed to promote rotation and ductility at the column bases. Results from this experimental program are compared to numerical analyses to validate a modeling approach for rotationally compliant column baseplate connections. Subsequently, the implications of structural response of a low-rise, steel frame structure with varying compliant base details is evaluated.

Introduction

Exposed column baseplate (ECB) connections play a crucial role in the seismic performance of steel moment frame structures. Figure 1(a) presents the typical components in ECBs and Figures 1(b)-(c) present photographs of different configurations of steel baseplates. In traditional designs, anchor bolts often resist both tension and shear forces, but this can result in premature failures and reduced ductility under seismic loading. Integrating shear lugs allows the decoupling of these forces: anchors primarily resist tension, while shear lugs handle lateral loads. This configuration not only enhances the ductility of the base connection but also enables energy dissipation through controlled deformation. This approach gained traction after observations from the 2010 Maule, Chile Earthquake, where structures with stretch-length anchors and shear lugs exhibited considerable inelastic deformation without failure, highlighting the effectiveness of partially-restrained connections for seismic resilience (Soules et al., 2016). Although stretch length anchorage is allowed below (Figure 1(b)) or above the concrete surface (Figure 1(c)), embedded stretch length anchorage is preferred in building construction for compactness.

Key standards like ACI 318-19 and AISC Seismic Provisions address design requirements for such connections to avoid nonductile tensile failures and improve performance under cyclic axial, shear, and flexural demands. ACI 318-19, for instance, mandates that tension-loaded anchors in seismic zones meet specific criteria, including a minimum stretch length of 8 bolt diameters, which promotes ductile behavior. AISC guidelines offer flexibility in designing column baseplates, with assumptions ranging from rigid, "fixed" bases that minimize rotation to "pinned" bases that require larger columns to control drift. The "partially-fixed" option, where anchor yielding provides controlled rotation, is particularly effective in approximating realistic connection behavior while improving energy dissipation capacity. This study numerically investigates how varying levels of fixity in column baseplate connections influence the overall response of the superstructure, aiming to provide insights that support the development of more effective connection designs for enhanced seismic performance.

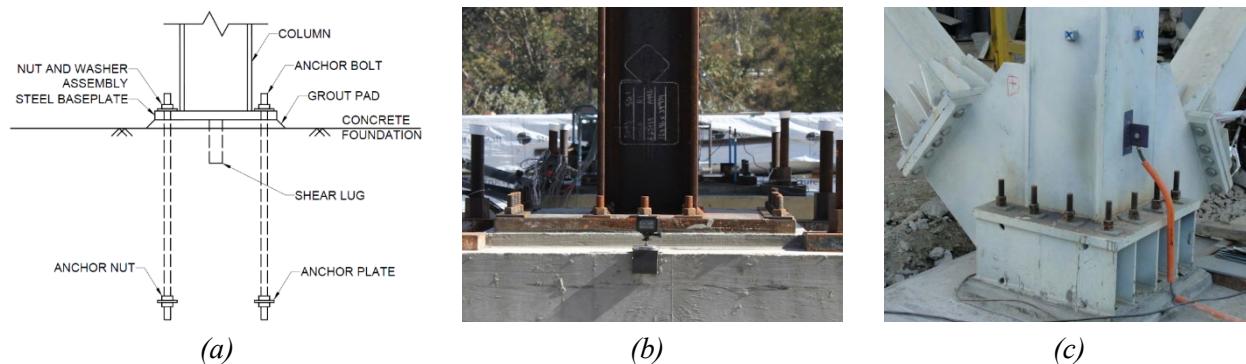


Figure 1. Column baseplate connections (a) typical steel ECB connection; (b) steel moment frame ECB with embedded stretch length anchorage (Morano et al., 2024); and (c) braced frame ECB with exposed stretch length anchorage (Soules et al., 2016).

Background

Recent studies underscore the potential of stretch-length anchors to enhance ductility and energy dissipation in seismic applications. Parks et al. (2020) demonstrated that anchors with an 8-bar diameter stretch length enable significant tensile deformation and energy dissipation without brittle failure in dry storage cask applications, especially when paired with yielding steel chairs. Similarly, Torres-Rodas et al. (2022) showed that extended anchor bolts in ECB connections concentrate ductility in the anchors, preventing damage to other components and facilitating post-earthquake repairs. Trautner et al. (2019) extended this concept to a miniature steel building with uplifting base connections, observing reduced base shear and residual drift, thanks to the energy-dissipative rocking mechanism enabled by ductile anchors. However, these studies have limitations in generalizability and scalability. The focus on specific applications, idealized loading conditions, and small-scale testing limits their applicability to larger, complex structures and highlights the need for research that addresses variable rotational stiffness and column fixity in diverse structural systems.

Recently, Morano et al. (2024a) investigated the effects of stretch-length anchorage in column base connections through full-scale shake table testing of a three-story steel-framed building at UC San Diego's LHPOST6. The test structure, known as the Modular Testbed Building (MTB²), incorporated base connections with replaceable anchor bolts with an 8-diameter stretch length beneath the concrete surface using 1.0 in. diameter F1554 Grade 36 anchors. Custom anchor sleeves allowed the flexibility to replace damaged anchors and adjust anchor number and layout, facilitating variations in column fixity and rotational stiffness. Baseplates also included cruciform shear lugs to effectively decouple tension and shear. The study examined two configurations: (1) the SMF Configuration: with fully fixed special moment frame (SMF) connections, and (2) the SMF+CB Configuration: The same as the SMF Configuration but 40% of the column base anchors removed to create rotationally compliant baseplates (CB). Figure 2 presents plan views of the interior and exterior baseplate configurations of the MTB². Findings revealed that in the SMF+CB setup, rotational movement at the base allowed ductile anchors to yield and absorb part of the roof drift, typically resisted by inelastic action in the superstructure. This partially restrained baseplate configuration also reduced interstory drift by 15% compared to the fixed SMF setup.

Numerical Analysis

Continuum-based finite element (CFE) analysis models were developed using Hilti PROFIS Engineering software to model different column baseplate configurations providing varying levels of column base fixity (Hilti PROFIS, 2024). These CFE models were used develop backbone curves which are

implemented into design grade structural analysis software, *ETABS*, models of the MTB² to provide an understanding of how varying levels of fixity affects global response characteristics when subjected to simulated seismic loading (CSI, 2023).

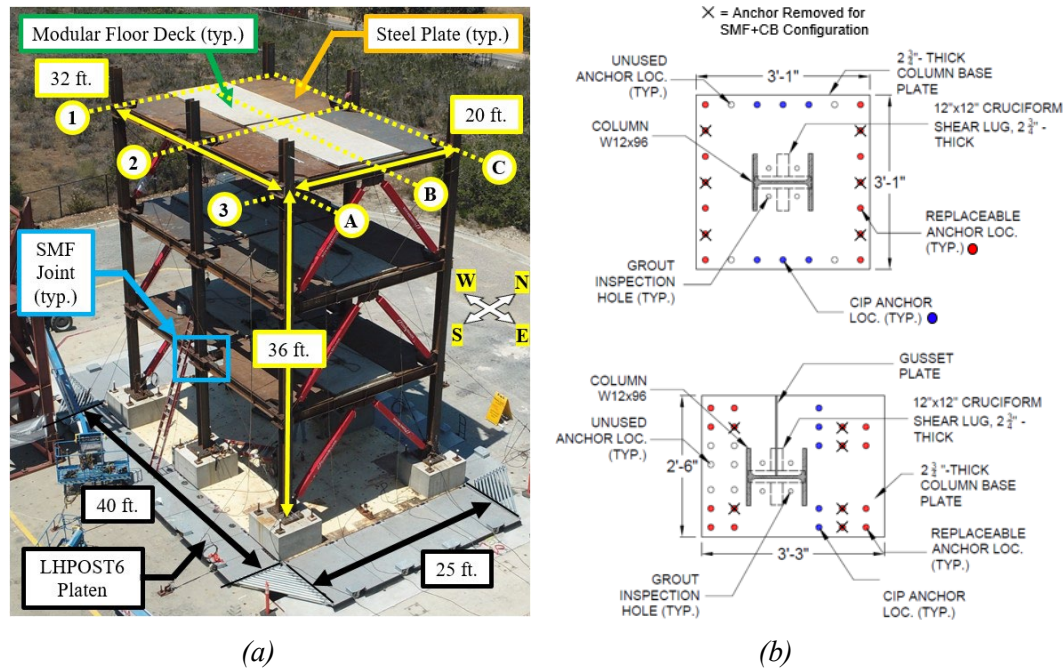


Figure 2. MTB² shakedown program specimen (a) isometric photograph of the MTB² in the SMF/SMF+CB Configuration; and (b) typical interior column baseplate (top, A2 and C2) and exterior column baseplate (bottom, A1, C1, A3 and C3).

Simulation Methodology. CFE models consist of the various components present in ECB connections, shown typically in Figure 1(a). These components include anchor bolts, steel column, steel baseplate, grout pad and concrete foundation. The PROFIS software does not allow for modeling of nonlinear material properties, thus all materials are modeled as linear. The elastic constants for all steel materials used in the CFE models are the modulus of elasticity, $E = 29,000$ ksi and Poisson's Ratio, $\nu = 0.3$. Yield strength is automatically specified though the program based on the grade of the plate, anchor, and column section selected. Anchor rods are treated as uniaxial springs with equivalent axial stiffness based on the effective net tensile area of the anchor rod and the effective stretch length. The stretch length of the individual anchors is simply specified directly in the software. Uniaxial unconfined compressive strength of the concrete is user-specified, and the elastic modulus is automatically calculated using ACI 319-19 methodology.

To establish backbone curves for the *ETABS* analysis, a constant axial load is first applied at the column section centroid on the baseplate. With the axial load held constant, uniaxial moment demands are incrementally applied, monitoring response properties such as anchor rod forces, baseplate deformation, and concrete stresses. Following Kanvinde et al. (2012), the yield rotation, θ_y , is identified at the moment demand where anchor rods reach yield strength in tension, M_y . The elastic rotational stiffness, β_y , is determined as M_y/θ_y , creating a bilinear approximation suitable for design grade analysis software, *ETABS*. The backbone curves are implemented into *ETABS* as a multi-linear plastic link property which are assigned to zero-length springs. Hysteretic behavior is assigned to the link using the pivot model. Although the pivot model is well-suited for reinforced concrete members, the model parameters can be tuned to closely represent the rocking behavior of partially fixed column baseplate connections. These zero-length spring elements are then assigned to the column bases.

Model Validation. The model is validated using experimental data from the MTB² test program (see Morano et al. (2024a) for modeling and analysis procedures). CFE models were constructed by means of the methods described above using the details of the MTB² column baseplates shown in Figure 2. Other critical details necessary in construction of the CFE models are as follows: (1) *Column Section*: ASTM A992 W12x96; (2) *Column Baseplate*: ASTM A572 Grade 50; (3) *Anchor Bolts*: 1.0 in. diameter, ASTM F1554 Grade 36 thread rod, stretch length beneath concrete surface = 8 in. (8 bolt diameters); (4) *Concrete Foundation*: Length = 66 in., Width = 66 in., Thickness = 36 in., $f'_c = 8,000$ psi; (5) *Grout Pad*: Thickness = 1.5 in. Tension tests of the stretch-length anchor sleeves were conducted prior to the shake table testing to refine anchor displacement estimates at each applied moment increment (Morano et al. 2024b). The backbone curves are integrated into the MTB² numerical model and sequential dynamic analysis is performed by imposing the as-recorded base accelerations recorded during the test program. Refer to Morano et al. (2024a) for the test matrix and details of the overall test program including test protocol and additional numerical analysis comparisons. Herein, select results of the baseplate response are provided. Specifically, Figure 3 presents a comparison of the numerical simulations of a baseplate responses compared to experimental data for three of the instrumented baseplates from the SMF+CB configuration subjected to the Northridge (RRS) motion scaled to 100% in the X-direction.

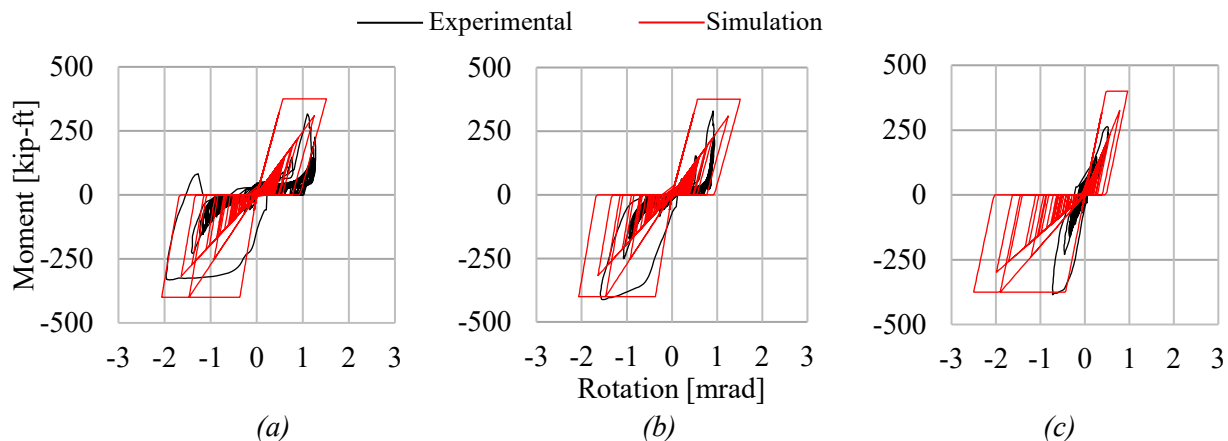


Figure 3. Comparison of experimentally derived column baseplate moment rotation response with the numerical simulations conducted using ETABS from the SMF+CB Configuration (MID 10: Northridge RRS 100% X) from (a) column A1; (b) column C1; and (c) column C3.

While the hysteretic pivot model does not perfectly match the experimental behavior, it provides a reasonable approximation of salient features including the transient stiffness and yield rotation as well as the flag-shaped hysteretic behavior. This model, compatible with commonly used design software, can be readily implemented in the analysis of steel-framed structures. The largest discrepancy is seen in baseplate C3 (Figure 3(c)), likely because the model assumes a constant axial load equal to the initial dead load on the column, but the directionality of the Northridge motion imposes a much larger axial load on this column, reducing its rotational demand. This will be investigated further in future work.

Detailing Variations. To extend the previous work and investigate how varying levels of fixity affect structural response characteristics, two additional baseplate detailing configurations for the interior and exterior MTB² baseplates are explored, as shown in Figures 4 and 5, respectively. The baseline configurations (Configuration 1) are the original SMF+CB baseplate details which provide the largest moment resistance and stiffness, described in the previous section. In the following two configurations, denoted Configurations 2 and 3, the only variable which changes is the number and layout of anchors while all other details remain consistent with the baseline configuration. The two additional baseplate configurations were detailed to provide a reduction in stiffness, moment strength, and yield rotation. In Configurations 2 and 3, the total anchor bolt areas are reduced by approximately 27-33% and 60-71%,

respectively, compared to Configuration 1. CFE simulations were conducted to approximate the bilinear, elastic-plastic backbone curves, shown in Figures 4 and 5, along with the key strength and stiffness parameters resulting from the variation in detailing and reduction in anchor bolts. Although the change in yield rotations is nearly negligible across the considered baseplate detailing configurations, there are notable reductions in strength and stiffness in the extended configurations of approximately 20-50% and 16-45%, respectively. Each baseplate configuration is implemented into the existing MTB² numerical model constructed in *ETABS* and nonlinear static and time history (NLTH) analysis is performed.

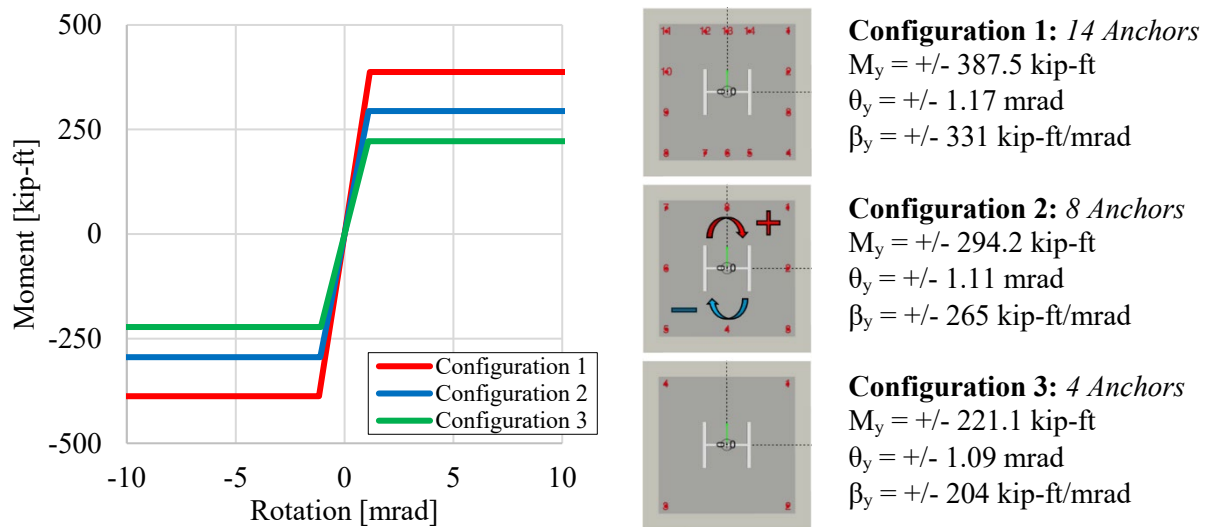


Figure 4. Three considered interior column baseplate configurations' bilinear backbone curves with key characteristics and CFE model plan views.

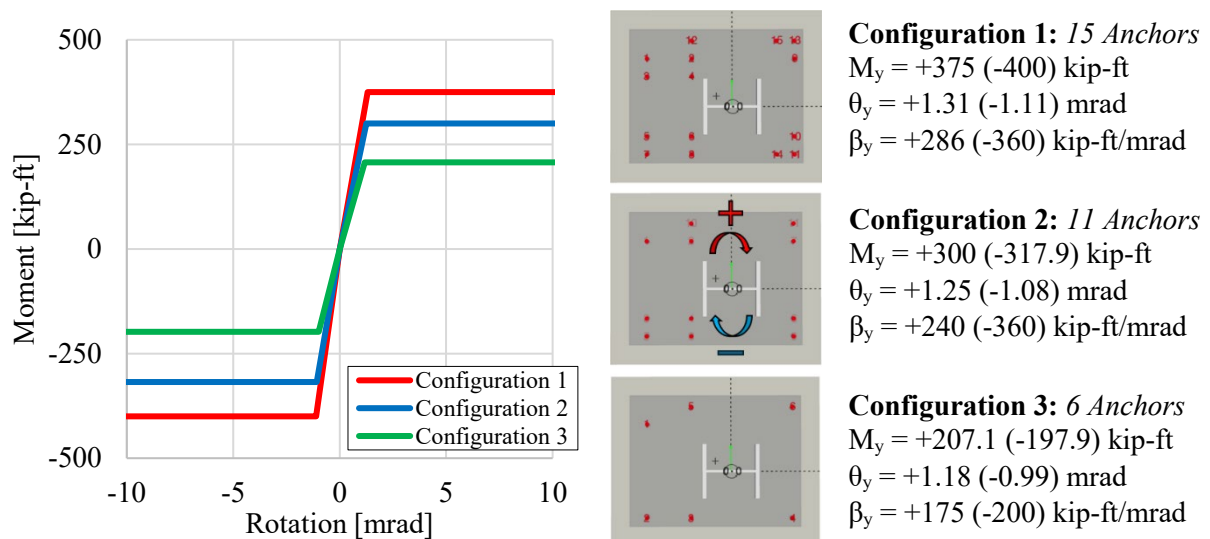


Figure 5. Three considered exterior column baseplate configurations' bilinear backbone curves with key characteristics and CFE model plan views.

Results and Discussion

Nonlinear static pushover analysis was conducted prior to conducting NLTH analysis following the provisions outlined in ASCE 41-17. This analysis allows for the determination of the overall backbone curves of the MTB² with the three different considered baseplate configurations to compare key backbone characteristics such as effective stiffness, K_e , yield displacement, Δ_y , and yield force, V_y . The three backbone curves are displayed in Figure 6. Results indicate that the change in effective stiffness is negligible across the considered configurations; however, non-negligible differences at the point of yielding is observed. As base fixity decreases from the initial baseline configuration, yield strength and displacement reduce. In fact, a maximum reduction in yield strength and displacement of approximately 45% is realized by Configuration 3 as compared to the baseline configuration.

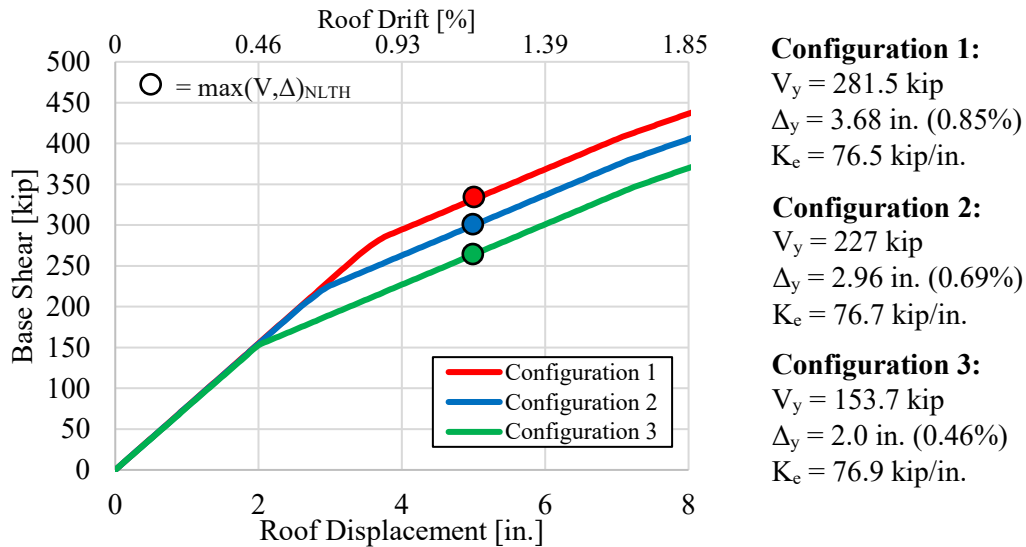


Figure 6. Nonlinear static pushover curves of the MTB2 with the three considered baseplate configurations.

Nonlinear time history analysis is run and results from one motion is presented herein: Northridge RRS 100% in the X (East-West) direction. This motion is selected for presentation of results in this workshop paper as it was the motion with the largest X-direction input during in the shakedown program of the MTB² in its SMF+CB Configuration. A simulation is also conducted with a perfectly-fixed base scenario to highlight how partial-fixity compares to fixed base results. To compare the global structural response because of the change in column base fixity, Figure 7(a)-(d) present the Peak Floor Acceleration (PFA) to Peak Input Acceleration (PIA) ratio, Peak Interstory Drift Ratio (PIDR), roof drift time history comparison, and moment rotation hysteresis comparison, respectively. In these figures the positive and negative direction profiles are shown to highlight the directionality of the input motion. PFA/PIA profiles indicate floor level acceleration amplification and are observed to generally increase along the height of the structure, with the largest accelerations reported at the roof level, as expected. Results also indicate that first floor PFA/PIA tend to increase with decreasing base fixity, while the PFA/PIA at Floors 2 and 3 tend to decrease, except for Configuration 3 in the positive direction of recording at Floor 3, albeit very small. It is evident in Figure 7(b), first story PIDRs increase as the base fixity decreases. This observation can be explained by the fact that reduced stiffness and strength defined at the base connections allows larger displacements to occur at the first floor, where the largest first story PIDRs are reported from Configuration 3. In Configuration 1, the baseline configuration, the largest PIDRs are reported in story 2, the same finding as reported from experimental data (Morano et al., 2024a). However, in Configurations 2 and 3, the largest PIDRs are reported in the first story. This may be a result of relative story stiffnesses in the extended configurations being reduced significantly enough to change the peak PIDR demand from

the second to the first story. The point of maximum base shear is indicated on the respective backbone curves of Figure 6 to visualize the level of nonlinearity achieved in each configuration under the same simulated input. This shows that although the roof displacement at peak base shear is relatively unchanged throughout the configurations, the achieved maximum base shear decreases as the strength capacity of the partially-fixed baseplates decreases, by approximately 8-12%. Figure 7(c) displays the comparison of roof drift time histories between Configurations 1 and 3. This figure indicates that although the peak displacements are relatively similar, the transient displacements post-peak are significantly reduced in Configuration 3 due to the larger rotational compliance of the base connections in Configuration 3. To highlight this effect, the moment rotation hysteresis from baseplate A1 reported from Configurations 1 and 3 are presented in Figure 7(d). It is evident in this figure that under the identical input, Configuration 3 detailing of baseplate A1 was able to plastically rotate approximately three times more than the Configuration 1 detail.

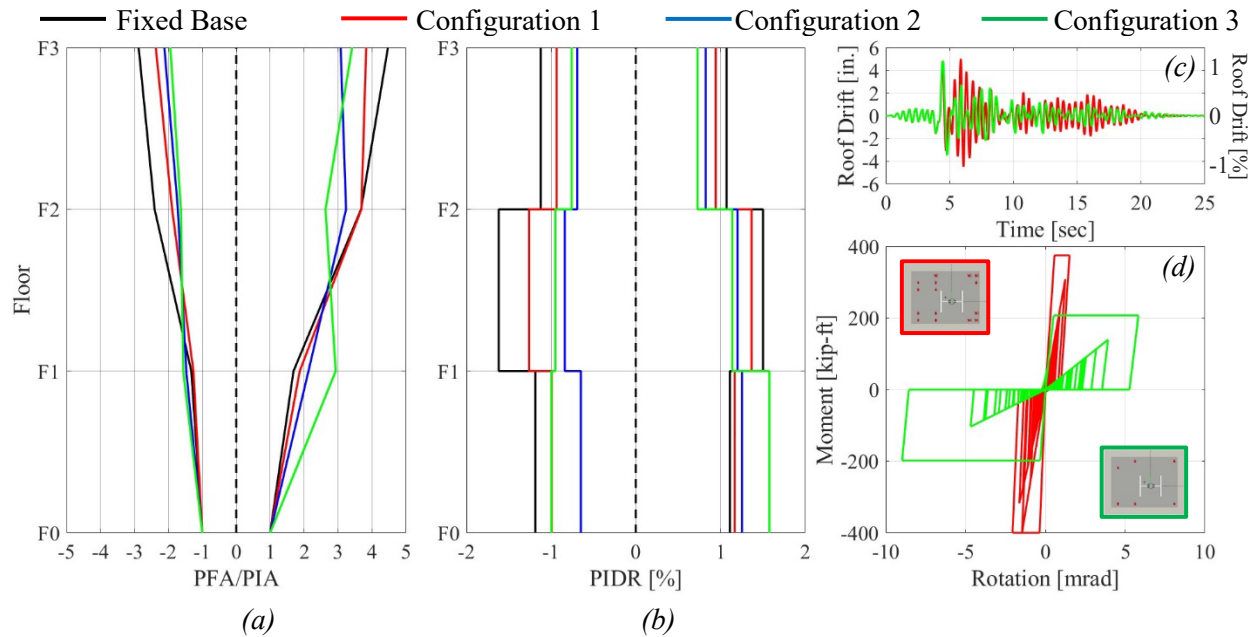


Figure 7. ETABS estimated demand profiles from the three considered configurations: (a) PFA/PIA profiles; (b) PIDR profiles; (c) Roof drift time history comparison; and (d) moment-rotation comparison from baseplate A1.

Conclusions

This study investigated the seismic performance of column baseplate connections with varying levels of rotational fixity using experimental and numerical approaches. Experimental results from the full-scale shake table tests demonstrated the potential of rotationally compliant column baseplates to enhance seismic resilience by redistributing drift demands and promoting ductility at the base connection. Specifically, configurations with partially restrained baseplates exhibited reduced interstory drifts and energy dissipation through controlled anchor yielding, emphasizing the benefits of integrated stretch-length anchorage. Numerical analyses validated the experimental findings and provided further insights into the influence of column base fixity on global structural response. Decreasing the fixity of baseplate connections resulted in reduced base shear demands and altered interstory drift profiles, indicating a trade-off between local ductility at the base connection and global deformation demands. Moreover, the backbone curves generated for varying baseplate configurations highlighted the critical role of anchor layout and strength in defining moment capacity and yield rotation. Future work will focus on extending these analyses to more complex configurations and validating numerical models under a broader range of

seismic loading conditions. By integrating such ductility-focused detailing, steel structures can achieve improved performance, repairability, and resilience during seismic events.

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E-ISOLATION –FURTHER DEVELOPMENT OF SEISMIC ISOLATION AND RESPONSE CONTROL TECHNOLOGY

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Abstract

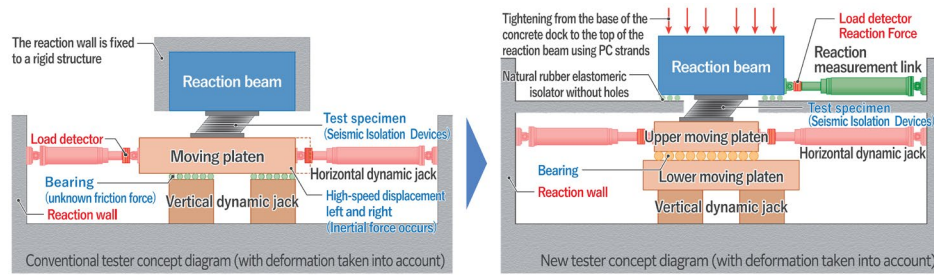
In Japan, seismic isolations and response control devices are diversifying and increasing in size due to their application to various civil engineering and building structures. However, the mechanical properties of these devices have not been sufficiently experimentally verified. Many engineers understood that dynamic tests using full-scale devices were necessary to verify and understand the dynamic characteristics of seismic isolation devices under large vertical loads. Since there was no "testing facility", verification by experiment was difficult. However, in 2023, Japan's first "E-Isolation: full-scale seismic isolation testing facility" was completed and started its operation.

Existing testing machines installed in other facilities measure the value of the load detector installed on the side of the horizontal dynamic jack. These measurements account for the force acting on the seismic isolator, as well as for the frictional force around the bearing and the inertial force of the moving platen. The latter results must be removed after the test, which is not an easy task and can take several months. The new testing machine installed at this facility measures directly the load acting on the specimen without including frictional and inertial forces, by measuring the force generated in a horizontally stiff steel pipe installed on the side of the reaction beam, towards the upper part of the specimen. This makes it possible to measure the force acting on the specimen instantaneously and accurately.

In this paper, we would like to present the mechanism and features of this testing facility.

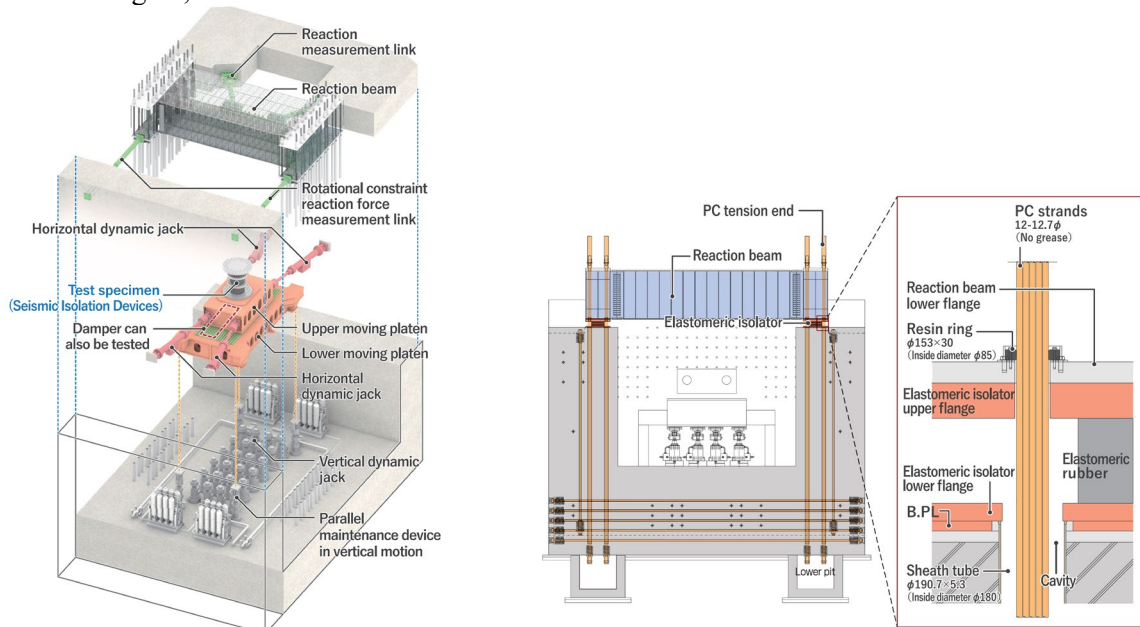
1 Dynamic testing system eliminating friction and inertial forces

1.1 The proposed system. Since the latter half of the 20th century, seismic isolation and response control technologies have been developed and put into practical use by challenging researchers and engineers worldwide. After more than 40 years, the technologies are now used in thousands of buildings, private residences, highways in many seismic areas in the world. Seismic isolation and response control methods can keep the structures undamaged even in a major earthquake and realize continuous occupancy. Among the typical rubber bearings used for seismic isolation, natural rubber-based laminated rubber bearings have a relatively linear stress-strain relationship. On the other hand, lead-plugged rubber bearings, high-damping rubber bearings, and friction-type bearings with additional damping functions are known to exhibit complex behavior under high axial forces and large dynamic deformations, and also depend on the size of the bearing. The numerical modeling of these bearings is complicated due to their size, velocity, pressure, and temperature dependence. Therefore, it is very important to verify the behavior of seismic isolation bearings using full-scale dynamic test facilities. However, there was facility in Japan that can dynamically evaluate full-scale bearings larger than 800 mm in diameter, used in large buildings and bridges. Such performance tests have been relied upon at test facilities out-side Japan, such as those in the United States and Taiwan. Fig. 1a shows the configuration of a typical seismic isolator test facility such as the Seismic Response Modification Device (SRMD) at UCSD^[2]. The moving platen that shears the specimen is moved dynamically in the horizontal direction while being subjected to vertical forces of tens of 10,000 kN. However, the frictional forces caused by the large vertical force and the inertial force of the heavy platen are mixed into the measured reaction force of the load cell attached to the horizontal dynamic jack. Although friction models that consider the effects of velocity and pressure have been proposed and provided based on the studies such as [1], [3], [4], it is still not easy to accurately reproduce a complex friction model that depends on the individual test machine conditions. The time delay involved in the calculation is a challenge for real-time hybrid simulation experiments.



(a) Conventional reaction force measurement system (b) Proposed reaction force measurement system
Fig. 1 Conventional and proposed measurement system

To solve this problem, the authors have proposed a new type of reaction force measurement system [7], as shown in Fig. 1b. A reaction beam was placed on top of the specimen, which is elastically supported with extremely low stiffness in the horizontal direction and almost rigid in the vertical direction, and the rigid reaction force measurement links are connected to the reaction beam in the horizontal direction. Most of the horizontal reaction force is measured in real-time using these force measurement links. Because the reaction beam hardly moves, almost no inertial force is generated. Furthermore, by supporting the reaction beam with elastomeric isolators, which are soft in the horizontal direction and rigid in the vertical direction, only 1–2% of the horizontal force transmitted through these elastomeric isolators which can be accurately calculated by their measured deformation. While similar measurement mechanisms have been employed in past experiments with no significant large vertical loads [8], the primary objective of our development was to realize this horizontal reaction force measurement system under immense vertical loads exceeding 10,000 tons.



(a) Isometric view (b) Section and fixing detail of the reaction beam
Fig. 2 Configurations of the planned testing facility

Table. 1 Load capacity of the new testing facility

	Vertical capacity	Horizontal capacity
Load	36,000kN (static), 30,000kN (dynamic)	6,500kN (static), 5,100kN (dynamic)
Stroke limit	250mm	±1,300mm
Velocity	70mm/sec	800mm/sec

The configuration and load capacity of the testing facility incorporating the proposed system is outlined in Fig. 2 a-b and Table. 1. In this testing system, a big steel reaction beam is placed above the specimen similar to the SRMD facility in UCSD, but in this case it is supported by laminated natural rubber bearings with low horizontal stiffness and high vertical stiffness, instead of being fixed to the RC reaction walls. The reaction beam is connected to a horizontally rigid RC reaction wall by a V-shaped measurement link with built-in load cells and a couple of rotational-constraint measurement links. The two transverse beams at both sides of the reaction beam are pulled down with horizontally-free PC strands of 14.6m length between the bottom and top anchors. The strands apply compressive force to the laminated rubber bearings so that the compression force on these bearings is not lost even when the reaction beams are subjected to lift-up effects during the experiments. As the results of the low horizontal stiffness of the rubber bearings supporting the reaction beam, majority of the horizontal force is measured through the reaction measurement link. The ratio of horizontal force transmitted through the laminated rubber bearings is less than 1%. Due to the reliable elastic characteristics of the laminated natural rubber bearings, horizontal forces in these bearings can be accurately included in the resultant reaction forces by using their horizontal deformation. Since the horizontal displacement of the reaction beam is expected to be in the vicinity of 1-2 mm and acceleration is at a negligible level, the measured forces should not be affected by the frictional or inertial forces, which have been an issue in existing facilities.

1.2 Reduced-scale mock-up experiment. To verify the practicality of the proposed reaction force measurement system, dynamic-loading experiments were performed using a reduced-scale setup. The test setup is shown in Fig. 3. This test frame was equipped with a horizontal dynamic jack that can apply loading in one direction and a moving platen supported by two linear sliders. The horizontal dynamic jack had a maximum load capacity of 500kN, maximum velocity of 500 mm/s, and maximum positive and negative amplitudes of ± 300 mm. V-shaped horizontal reaction force measurement links with the load cells were attached to the side of the reaction beam. The reaction beam was supported by elastomeric isolators (laminated natural rubber bearings) attached to the bottom beam. The reaction beam was tightly connected to the bottom beam by eight PC-steel rods, and the initially applied tension exceeded 1600kN so that the elastomeric isolators supporting the reaction beam will not lift up. The ends of the PC-steel rods were anchored using rotational nuts and remained horizontally free.

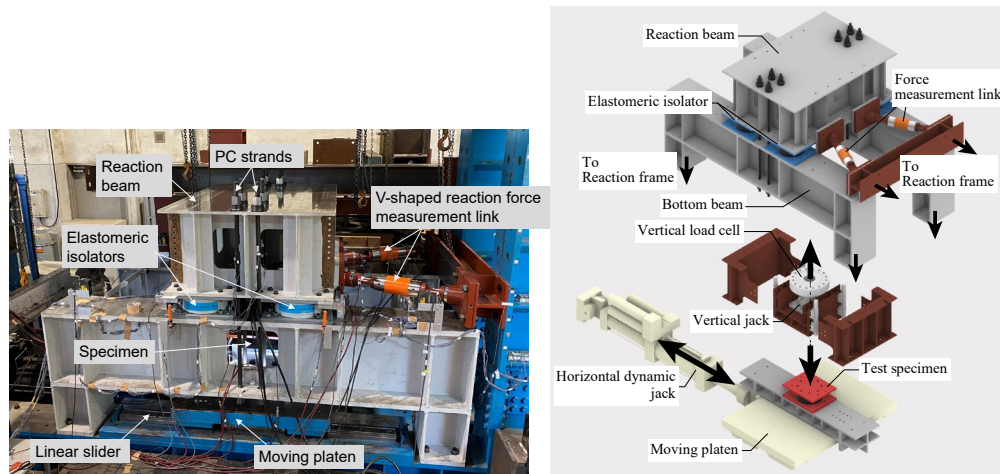


Fig. 3 Setup for reduced-scale mock-up experiment

The horizontal reaction force in the proposed reaction force measurement system defined as R_1 was obtained as the sum of the horizontal components: force acting on the load cell with V-shaped force measurement links, horizontal force calculated from the deformation of the four elastomeric isolators supporting the reaction beam, $P\Delta$ force due to the initial tension of the PC-steel bar binding the reaction beam.

The reaction force measured by the load cell attached to the horizontal dynamic jack is defined as R_2 . As a reference, the horizontal reaction force R_3 was calculated from the horizontal stiffness k of the laminated rubber specimen according to the manufacturer catalog (stiffness at 100% shear strain) and the shear deformation. Fig. 4 shows the horizontal reaction force-horizontal deformation relationship for $f = 1$ Hz, $\sigma = 15$ MPa, and amplitudes $A = \pm 2, 20$, and 100 mm, with dynamic loading and vertical load. It can be seen from these plots that there is a large error between R_1 and R_2 . Especially in the small amplitudes error is larger due to the frictional force between the moving platen and linear sliders. It can be observed from these graphs that errors increased as the sinusoidal amplitude increased. It can be explained by the inertial force being proportional to the acceleration. The measured values of the proposed measurement links are stable and unaffected by friction and inertial forces, even in a small deformation range.

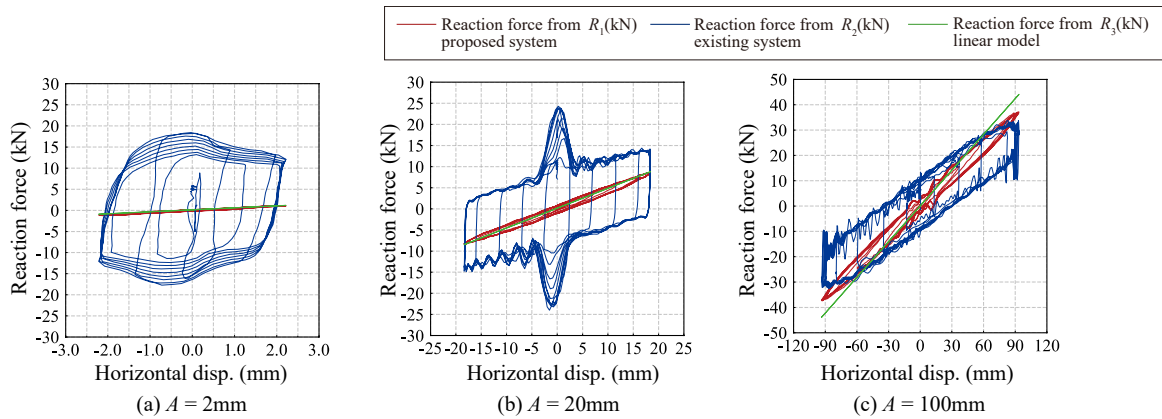
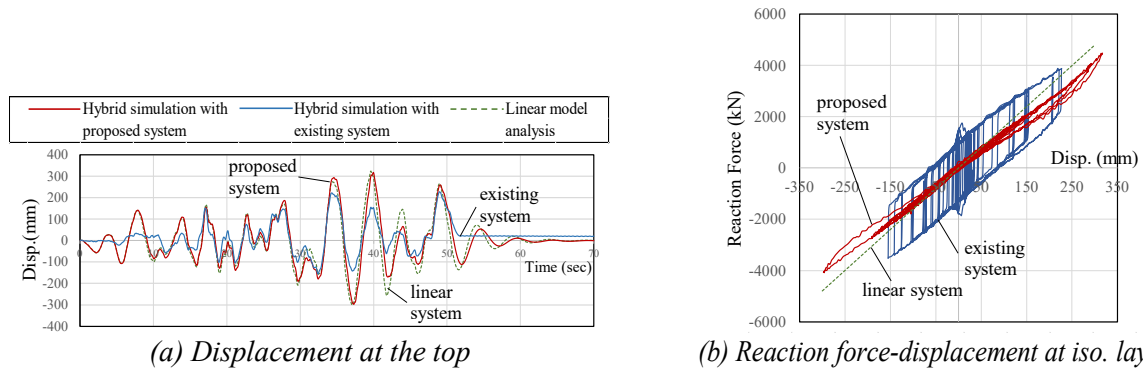


Fig.4 Reaction force-deformation relationship: dynamic loading ($f=1.0$ Hz), with 15MPa compression

Fig. 5 shows examples from hybrid simulation [5],[6] results of a seismically isolated two-story building using the reduced-scale mock-up experimental setup and a rubber bearing specimen. In the figure, the red line indicates the response results of the hybrid simulation using the proposed measurement system, and the blue line is using the load cell at the horizontal dynamic jack, which includes the friction and inertial force. The dotted green line is the pure response analysis, which estimates the rubber isolator bearing as linear stiffness. The red line is more accurate, reflecting the precise nonlinear hysteresis of the rubber bearing, which indicates a more realistic response. While the result with the existing system shows significant error due to contamination of the frictional and inertial forces.

From those reduced-scale mock-up tests and simulations, the proposed system was found to be effective through various dynamic conditions of vertical pressure and frequency with an error of less than 1%. On the other hand, the conventional dynamic jack load cell measuring the reaction force was heavily affected by the inertial force under dynamic loading and frictional force when a vertical force is applied.



(a) Displacement at the top
Fig.5 Hybrid simulation example using the proposed measurement system vs. existing system

2 Construction of the testing facility

The detailed design and construction of real-size testing facility is described in the following.

2.1 Design and construction of RC reaction wall. A part of the design drawings of RC reaction wall and construction plan are shown in Fig.6. For a future extension to 50,000kN vertical loading capacity with a two-dimensional horizontal moving platen, the RC reaction wall has been designed to withstand these conditions with 9.90-11.00m clear span. The thickness of the reaction walls is 3.5m and the foundation thickness is 4.5m. Pre-stressing is implemented in the foundation and walls, for preventing cracks. The reaction beam is placed on the RC reaction walls supported through twelve laminated natural rubber bearings with 650mm diameter. PC strand sets consisting of twelve strands with 12.7mm diameter are connecting the reaction beam onto the RC reaction walls. Four of these PC strand sets are provided for each bearing with a design pre-stress level of 1,050kN each (1,240kN in construction, considering future relaxation). Pre-stressing forces are designed to keep the support rubber bearings under compression even under the most severe testing conditions. In order to allow the expected horizontal movement of the reaction beam, PC strands have been placed in sheath tubes with 190.7mm diameter, 5.3mm thickness, allowing 47mm clearance in the design stage around the PC strands. The 11.52m tall sheath tubes are precisely installed into the RC reaction walls within 1/650 error margin, using template frameworks as shown in Fig. 7a. As a result, 30mm clearance around the PC strands is secured (Fig. 7b).

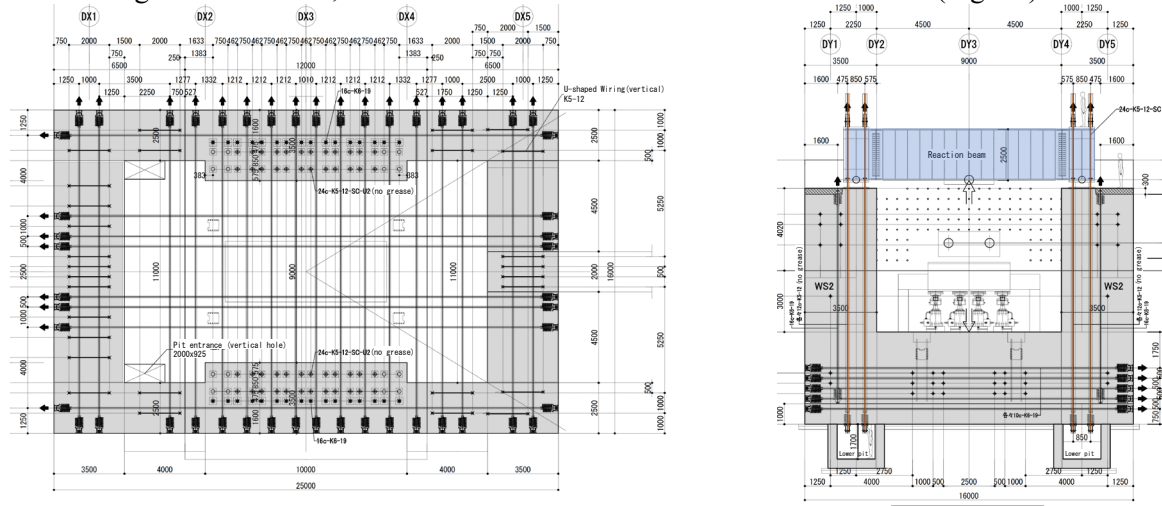
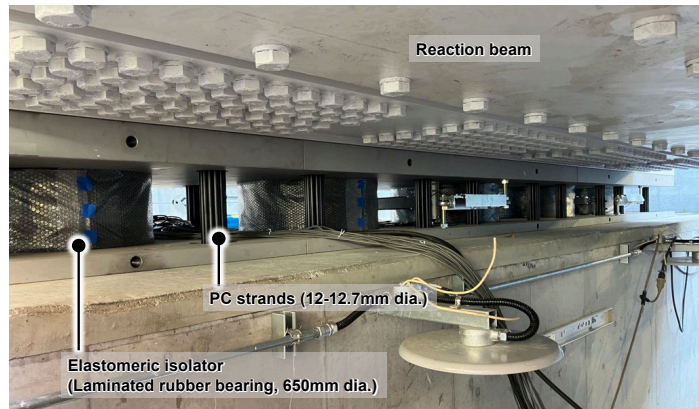


Fig. 6 Plan and section of RC reaction wall with PC strand arrangement (Courtesy of Kurosawa Kensetsu)



(a) Template framework for sheath tube



(b) PC strands and elastomeric bearing supporting reaction beam

Fig. 7 PC strands construction anchoring the reaction beam

Fig. 8 Plan and section of reaction beam and its Fabrication and installation

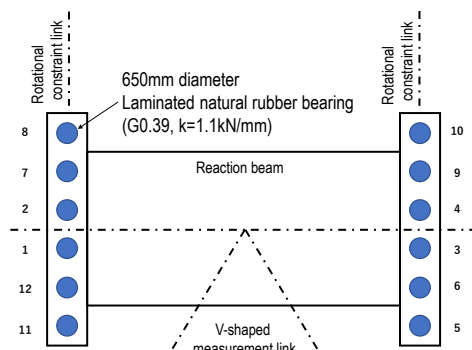


(b) Fabrication of reaction beam

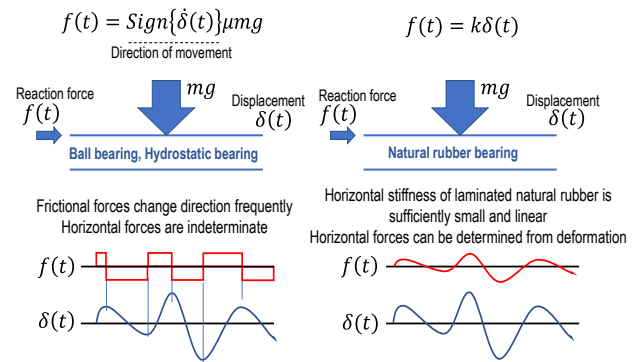


(c) Installed reaction beam on elastomeric bearings

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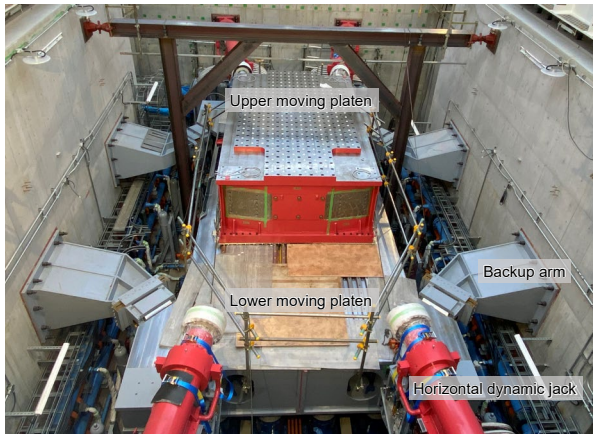
(a) Layout of rubber bearings



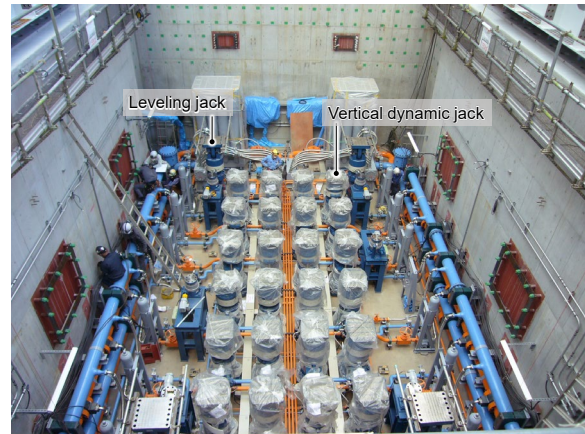
(b) Comparison of bearing characteristics

Fig.9 Elastomeric isolator supporting the reaction beam

2.4 Configuration of the moving platen. As shown in Fig. 10a, the moving platen consists of a lower layer that moves vertically and an upper layer that moves horizontally in the x-direction. The lower platen is controlled by 24 vertical dynamic jacks (Fig. 10b). The upper platen is supported on the lower layer by linear sliders with 18 rails and 14 sliders on each rail (total number of the sliders of 252), moves in the x-direction by 4 horizontal dynamic jacks. As the vertical jacks are simply supporting the moving platens, there is no restraining measure against partial uplift as a result of the rotation about y-axis and x-axis. Also, they do not resist against z-axis rotation or y-axis displacements. Therefore, 4 leveling jacks restrain x-axis and y-axis rotations, and 4 backup arms are provided to limit z- axis rotation and movement in the y-axis. Furthermore, 4 horizontal dynamic jacks are angled slightly (10° from the x-axis) to control y-axis movement and z-axis rotations. The leveling jacks are linked with cross-connected oil chambers which stabilize the horizontal level without obstructing the vertical movement of the mechanism. Total 28 numbers of vertical load cells are provided on the vertical jacks and leveling jacks, and total vertical reaction force (F_z), rotational moment around x-axis (M_x) and rotational moment around y-axis (M_y) can be obtained from these values.



(a) Moving platen and backup arm



(b) vertical dynamic jacks and leveling jacks

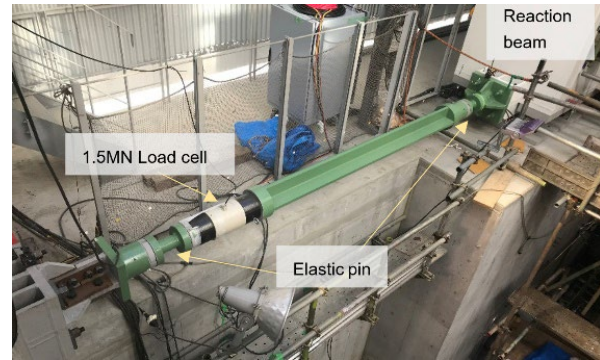
Fig. 10 Configuration of the moving platen

2.5 Design and fabrication of force measurement links. There are two types of reaction force measurement links as shown in Fig11; V-shaped measurement links intersecting the center of the reaction beam, a couple of rotational constraint measurement links. 4MN and 1.5MN load cells are mounted on the V-shaped link, and the rotational constraint link, respectively. V-shaped measurement links restrain the displacement of the reaction beam in the x and y directions, and the individual reaction forces along x and y directions can be obtained by multiplying cosine and sine components on axial forces, respectively.

To restrain the rotation of the reaction beam about the z-axis, a couple of rotational constraint links are added between both wings of the reaction beam and the RC reaction wall. Conducted analyses indicate that up to 75% of the major reaction forces will be resisted by the V-shaped measurement links and the remaining by the rotational constraint links. The additional reaction forces measured by these links are expected to be 99%. V-shaped links should be able to keep up with the vertical displacement of the center of the reaction beam under a vertical force of 30,000kN within the elastic range. Therefore, joints with reduced bending stiffness at both ends (elastic pins) are used for these links.



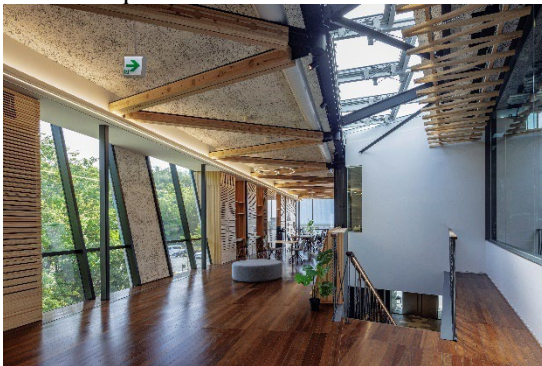
(a) V-shaped measurement links



(b) Rotational constraint link

Fig. 11 Reaction-force measurement links

2.6 Building design. The E-isolation test facility building is composed of the main test block, including the test machine and hydraulic source plant, and the office block, including the measurement room and the observatory office. The interior of the observatory office is designed with timber details, providing a relaxed atmosphere for the researchers and visitors.



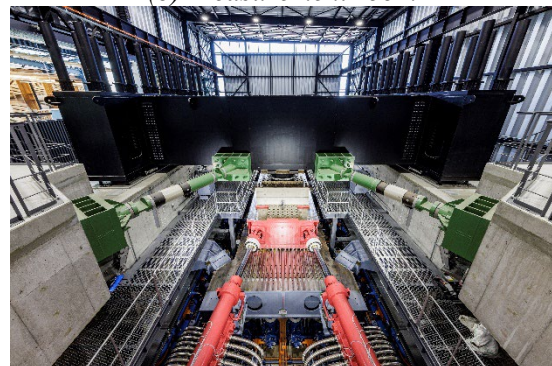
(a) Interior view designed with timber details



(b) Measurement room



(c) Japanese cherry blossom and E-Isolation



(d) main test block view

Fig. 12 Completion of E-Isolation

3 Dynamic loading tests in E-Isolation

Full-scale dynamic loading tests have been carried out after completion of E-Isolation construction in March 2023. In the following, we would like to discuss some representative test cases that attempt to evaluate the effects of the friction and inertial forces.

3.1 Experimental overview. An overview of the setup and test specimen is shown in Fig. 13. The test specimen, a natural rubber bearing with a diameter of 1200 mm, is placed between concrete attachments attached to the reaction beam and the upper moving platen. The load of the horizontal link member is obtained by summing the V-shaped measurement links, P_V installed on the east side of the reaction beam and the load of the rotational constraint links, P_R installed on the west side of the reaction beam. As with the conventional method, the load of the dynamic loading jack connected to the upper moving platen, P_A is also measured and used for comparison.

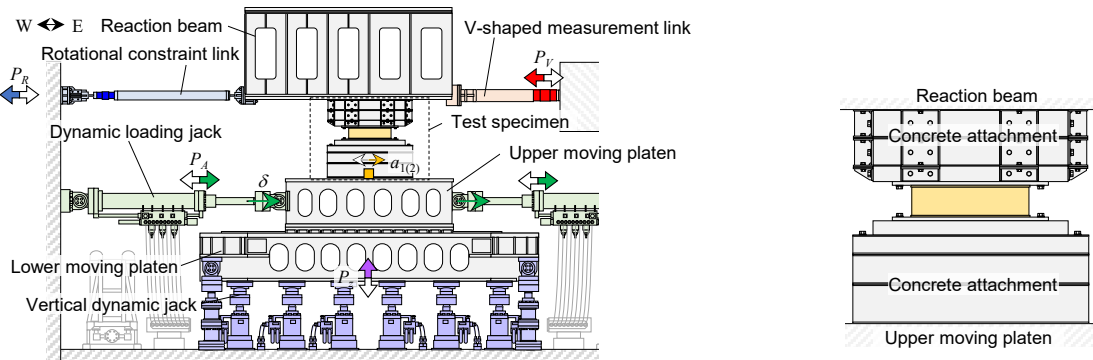


Fig. 13 Overview of test device and concrete attachment

3.2 Experimental results and discussion. Fig. 14 shows the load-deformation relationship of the 1200mm diameter natural rubber bearing under quasi-static and dynamic loading, distinguished based on a vibration frequency of 0.1 Hz. The black line is the load of the horizontal link member ($P_V + P_R$), and the green line is the load of the dynamic loading jack (P_A). The yellow line is the difference between the two ($P_A - P_V - P_R$). The history plot using the load of the horizontal link member shows linear elastic behavior. On the other hand, the history plot using the load P_A of the loading jack shows a bilinear history curve with a large bulge at the quasi-static load (Fig. 14a and 14b). This is considered to be due to the effect of frictional forces, since the difference between the two is almost constant regardless of the deformation, and the value becomes larger as the vertical load increases. In addition, a history with even greater disturbance is observed under dynamic loads (Fig. 14c and 14d). This is thought to be due to the influence of inertial forces, since the difference between the two becomes greater as the frequency increases. The practicality of the proposed reaction force measurement system, demonstrated using a reduced-scale setup, has been confirmed by its implementation in a full-scale test facility, E-Isolation.

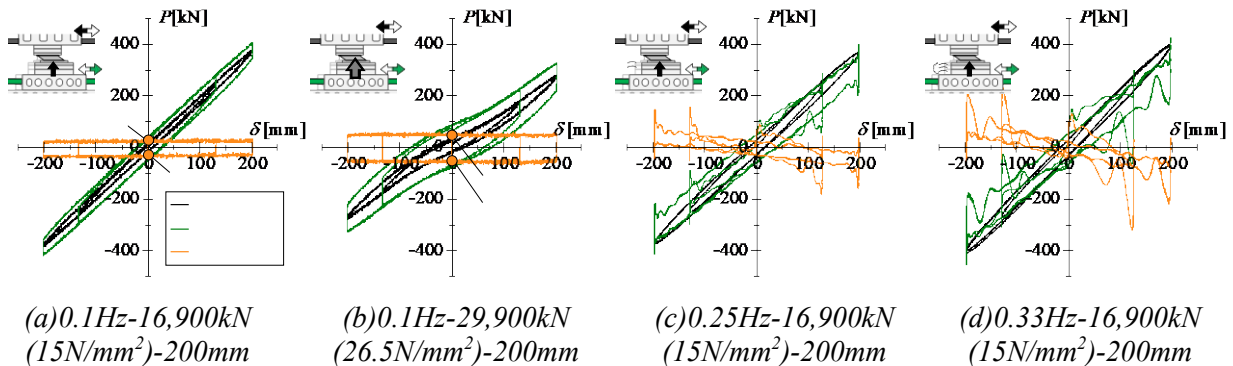


Fig. 14 Experimental results in E-Isolation

4 Conclusions

In this article, E-Isolation, a recently-constructed dynamic testing facility in Japan was introduced. It is specifically designed for seismic isolation bearing tests, eliminating the effects of friction and inertial forces. Reported discussions are summarized as follows.

- 1) How the proposed system eliminates the contamination of frictional and inertial forces mixing into the measured reaction forces in the existing dynamic test systems by using a measurement link that directly measures the reaction force.
- 2) The performance of the proposed concept including the substantial accuracy of friction and inertial force elimination method was confirmed using a reduced-scale experiment.
- 3) Design and construction of a full-scale test facility, E-isolation was completed, starting the performance tests for various seismic isolation bearings and energy dissipation devices.
- 4) The usefulness of the proposed system was also demonstrated in the full-scale facility.

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LESSONS LEARNED FROM SYSTEM-LEVEL RESPONSE OF FULL-SCALE SHAKE-TABLE TESTS OF LOW DAMAGE BUILDINGS – OBSERVATIONS AND LINKAGES TO COMPONENT TEST RESULTS

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This paper is a modified version of work presented in Henry et al (2024) that was presented at the 18th World Conference on Earthquake Engineering

Abstract

The increasing focus on seismically resilient structures that offer performance beyond the code mandated minimum has led to the development of low-damage structural systems. These systems seek to safely resist seismic demands with minimal damage that is confined to easily replaceable components. These new systems are typically tested at a component and/or sub-assembly level, and often only under pseudo-static velocity profiles prior to being implemented into real buildings. However, real buildings have complex three-dimensional interactions, load paths, and potential displacement compatibility issues that are not captured by simplified component and subassembly tests. This paper presents relevant findings from a full-scale building test undertaken at the Tongji University earthquake test facility.

To validate the seismic performance at a system level, a two-storey precast concrete building implementing low-damage structural design concepts was designed and tested on a multi-functional shake-table array. The test building used a combination of post-tensioned walls and a perimeter frame that utilised slotted beam connections. The building was subjected to both unidirectional and bidirectional ground motions in a range of different design configurations. The structure demonstrated unexpected three-dimensional interactions that may not have been detected from component or subassembly testing, or through the independent application of two unidirectional ground motions. Key outcomes include deformation compatibility when subjected to both unidirectional and bi-directional ground motions, and the implicit interactions that exist between the two orthogonal directions of the building. Validation at a system-level is crucial to identifying key design requirements and this project serves as an example of the benefits of such large-scale tests when developing new structural systems.

Introduction

Recent earthquakes have highlighted the need for increased focus on seismic resilient structural designs and a shift towards functional recovery performance requirements. Low-damage structural systems have emerged as a means of improving seismic resilience by minimizing damage and downtime to buildings following earthquakes. A range of alternative low-damage structural systems have been developed, and a summary of the development of precast concrete systems is presented by Kurama et al. (2018). The successful development of low-damage precast concrete systems, in particular jointed designs that utilize post-tensioning, has led to the implementation of such systems around the world. The implementation of low-damage precast concrete buildings in New Zealand is summarized by Henry and Rodgers (2024), showing a preference for post-tensioned walls as the primary seismic load resisting system.

The ideal process to introduce new structural systems requires thorough development that spans from an initial concept, component testing, sub-assembly testing, and system level validation, as illustrated in Figure 1. For low-damage precast concrete systems, a range of different ideas and concepts have been proposed and many of these have progressed to component testing where the design methods and calculations of response are verified and refined (e.g. test of single rocking walls by Perez et al. 2013,

hybrid walls by Smith et al. 2011, and Precast Walls with End Columns by Sritharan et al. 2015). For systems that are well proven at a component level, research has continued to consider their behavior and response when connected to other components, such as sub-assembly tests of Precast Walls with End Columns connected to floor systems by Liu et al. (2020) and bi-directional testing of a 2-storey frame using slotted beams by Muir et al. (2012). To comprehensively evaluate the seismic performance of new systems, the best available laboratory test is a large-scale shake-table test of the entire building system. Bi-directional dynamic testing of the full building system can capture complex 3D interactions, load-paths, and displacement compatibility issues between the structural frame, flooring systems, and other components, that may not be captured during component or sub-assembly tests. Large-scale shake-table tests of building systems are both time-consuming and costly but can validate the performance at a system-level when subjected to realistic earthquake ground motions.

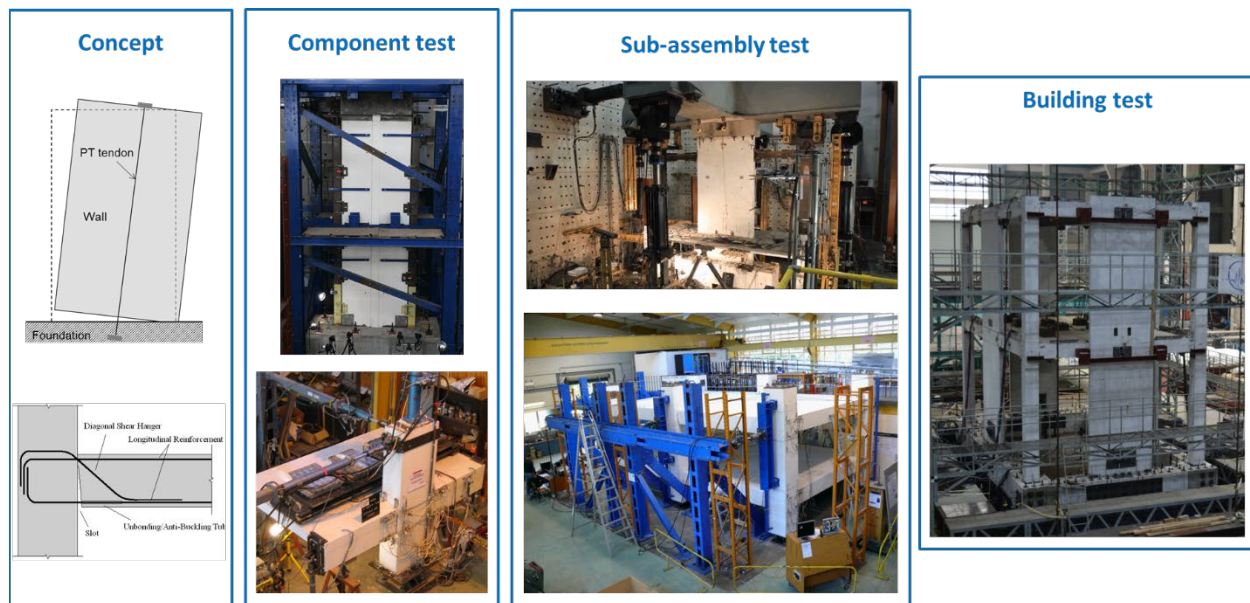


Figure 1. Development of building system from initial concept to building system test.

In New Zealand, the implementation of low-damage precast concrete buildings commenced after the successful development and testing of systems at a component level, and in some cases at the sub-assembly level. To further validate these new designs, a large-scale shake-table test of a prototype building was proposed. Prior shake-table tests of buildings that incorporated post-tensioned walls had highlighted some key system-level issues that needed to be addressed with connection detailing. A three-storey building with hybrid post-tensioned walls and slotted wall-to-floor connectors was tested at the University of California San Diego (Schoettler et al. 2009). The test primarily focused on the diaphragm response but confirmed that slotted connectors could be implemented between the wall and floor. However, the design of such slotted connectors needs careful consideration to ensure appropriate robustness, with the test highlighting the potential damage that can result to the floors when the wall uplift exceeds the connector slot travel.

Furthermore, a four-storey building with post-tensioned walls and frames was tested on the E-defence shake-table (Nagae et al. 2012, Gavridou et al. 2017). The floor was rigidly connected to the post-tensioned wall and experienced damage at the wall-to-floor interface. Numerical modelling of the E-defence building highlighted the importance of capturing the system-level interactions, including the in-plane and out-of-plane response of the floor (Watkins et al 2017).

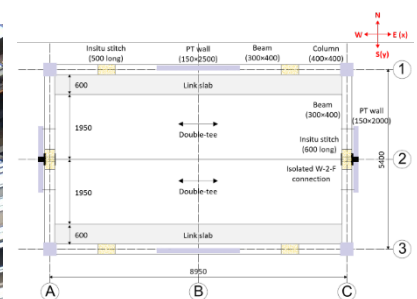
Test Building Description

A two-storey test building was subjected to bi-directional shaking, to verify the seismic response of a low-damage concrete building implementing state-of-art design concepts and construction details that were being implemented in buildings in New Zealand, as shown in Figure 2. The test was completed at the Tongji University multi-functional shake-table array as part of a collaborative research project between the New Zealand Centre for Earthquake Resilience (QuakeCoRE) and the International Joint Research Laboratory of Earthquake Engineering (ILEE). A full description of the test building design and results is reported by Henry et al. (2021) and the test dataset is published and described by Lu et al. (2022). Key features of the test building design included:

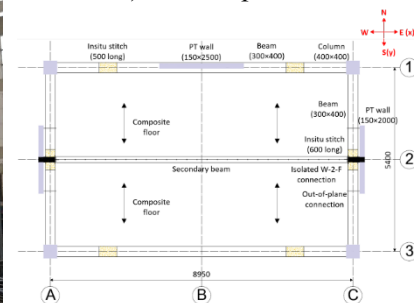
- Post-tensioned walls provided the main lateral-load resistance in both directions of the building. The perimeter frame was detailed using slotted beams, primarily designed to carry gravity loads, but provided lateral-load capacity using energy dissipation components during some test configurations.
- The Level 1 floor system consisted of long-span precast concrete double tee units. A steel tray composite floor was used for level 2 in a short span configuration with a secondary steel beam aligned through the longitudinal centre of the floor.
- Various combinations of dissipating devices were installed at both the wall bases and slotted beam joints, including steel fuses, HF2V lead dampers and non-linear viscous fluid dampers.
- Two alternative wall base details were included: 1) grouted joint detail with no shear key for the walls on grid lines 1&3 with light armouring consisting of a small steel angle; 2) recessed steel pocket detail with heavier steel armoured end toe region for the walls on grid lines A&C (refer Figures 2b and 2c).
- Three alternative wall-to-floor connection designs, consisting of a flexible link slab on level 1 in the EW (long-span) direction, flexible composite floor on level 2 in the EW direction, and an isolated steel tongue connection on both levels in the NS (short-span) direction.
- The building was designed for seven different configurations or lateral drift targets by adjusting the wall post-tensioning design, and installing different energy dissipating devices, as summarised in Table 1. Two torsional design configurations were also included where the wall strengths were altered to create a strength eccentricity in the building.



a) Photo of the building prior to testing



b) Level 1 plan



c) Level 2 plan

Figure 2. Two-storey low-damage concrete test building.

Testing was conducted over a period of six days with the building subjected to a total of 39 separate earthquake tests. The seven building design configurations were subjected to combinations of ground motions including a Serviceability Limit State (SLS) record, far field (FF) and near fault (NF) Design Basis Earthquake (DBE) records, and short (S) and long duration (L), and Maximum Considered Earthquake (MCE) records. The Design 1a building was first subjected to SLS (25%), SLS2 (50%) and DBE (100%), followed by DBE intensity level tests for Design 1b and 1c. For Design 2, DBE and MCE (180%) were both tested. The torsional configurations of Design 2Ta and 2Tb were subjected to only DBE shaking. The final Design 3 was subjected to DBE and MCE (150%). For many ground motions three individual tests were run consisting of unidirectional shaking in both x and y axis of the building, followed by bi-directional (xy) horizontal shaking.

Table 1. Test building design configurations

Design	Design lateral drift	Dissipating devices		Wall A/C strength
		Wall base	Beam-column	
1a	1%	Steel fuse	Steel fuse	Equal
1b	1%	Viscous damper	HF2V	Equal
1c	1%	Steel fuse	HF2V	Equal
2	2%	Steel fuse	-	Equal
2Ta	2%	Steel fuse	-	±10%
2Tb	2%	Steel fuse	-	±20%
3	3%	-	-	Equal

Key Results and Outcomes

System response and deformation compatibility: The building was designed using a direct displacement-based design method and guidelines for the design of jointed precast concrete systems (NZS 3101:2006 Appendix B; PRESSS Design handbook – Pampanin et al. 2010). The measured building response successfully validated this proposed design method, with peak drifts measured during testing all below the respective design drift targets, with the exception of the D1a building configuration NS direction. The unexpected large drift demands in the D1a NS direction were attributed to nuts on the steel fuse dissipaters rattling partially loose during repeated tests (including white noise tests between earthquake motions), which was not discovered until after testing of this phase was complete.

The building recentered with no significant residual drifts for all tests, despite the design slightly exceeding current recentering recommendations, proving that the self-centering design requirements are conservative as they ignore the dynamic shakedown effect. This finding highlights the importance of system-level dynamic tests for validation of such design limits that are based on static equilibrium at a component level and ignore the impacts of systems interactions and dynamic response of the building.

To achieve the low-damage performance outcomes, the building was designed with low-damage components as well as explicit consideration of component interactions and deformation compatibility when designing and detailing connections. Key findings from the test, as reported by Henry et al. (2021), that validated the design approach adopted include:

- Explicit consideration of deformation compatibility in the building design resulted in a reliable seismic performance with damage controlled at key locations, including the wall toes, slotted beam joints, and flexible floor connections. The building was deemed to have met the immediate occupancy performance objective for all tests including repeated 180% intensity maximum consider earthquake demands. This outcome is a significant improvement on the outcome of comparable test buildings.

- The two alternative wall base details represented designs used in practice and performed well with clean uplift and no significant slip, even during bi-directional demands. The grouted pocket in the NS (short-span) direction, and light armouring angle in the EW (long-span) direction were undamaged during lower drift tests and proved to be an improvement on the details used in prior tests. Concerns about the grouted pocket connection walking during bidirectional excitation due to the absence of shear dowels proved unfounded. The heavier armouring detail reduced the extent of spalling during larger drift demands, with less damage observed compared to the walls with lighter armouring.
- The frames performed well during the tests, with opening and closing of the slotted beam joints with minimal damage. No significant vertical shear slip or torsional rotation occurred at the beam joints, confirming that the design of the diagonal hanger reinforcement was adequate to provide load transfer at the ends of the beams even during bi-directional shaking demands. The slotted beam design successfully minimised beam axial elongation and prevented damage to the adjacent floor.
- Both wall-to-floor connections successfully addressed deformation compatibility. For the flexible connections, vertical deformation was primarily accommodated within the link slab or composite floor span with well distributed narrow cracks forming. The isolated wall-to-floor connections successfully allowed the walls to uplift and rotate without imposing deformations or damage to the adjacent floors.
- Careful design of the seating lengths and armouring of the support beam ledge prevented any visible damage to the precast floor supports. The potential for damage to precast floors in conventional buildings due to system-level deformation compatibility has been a significant lesson for past earthquakes and research, so the success of the implemented support details was critical to achieving low-damage outcomes.

Examples of the measured base moment-drift response for Design D2 are provided in Figure 3. The response followed a characteristic flag-shaped response. The design base moment capacity was comfortably exceeded for all building designs and loading directions, with the design calculated lateral strength represented by the dotted red line in Figure 3. The system overstrength was more pronounced in the EW direction, with the D2 building exhibiting a measured base moment capacity at the 2% design drift approximately 50% larger than the design capacity in the EW direction compared to a 25% overstrength in the NS direction.

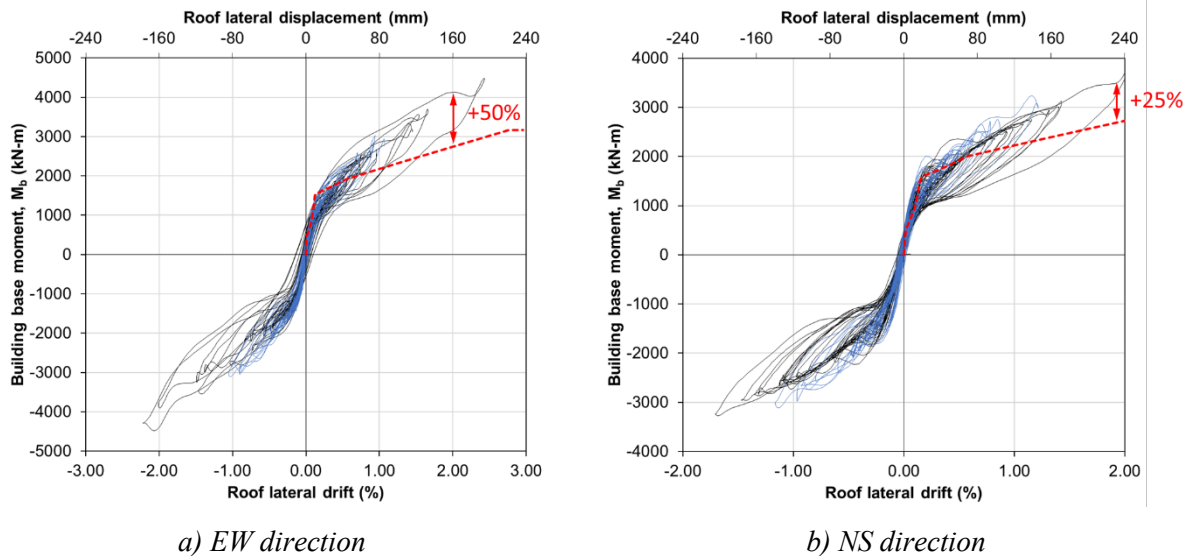


Figure 3. Examples of measured response for Design D2 compared to design estimates (dotted red line).

Although an overstrength in the actual capacity is expected when compared to design calculations where conservative approximations are made in the analysis (e.g. slotted beam joints and column bases assumed to be a perfect pin), the building was specifically designed to minimize structural interactions and ensure deformation compatibility in connections. The larger overstrength in the EW direction was attributed to the additional capacity as a result of the interaction between the walls and floor. However, it should be noted that the additional overstrength was controlled by using a flexible link slab, which greatly improved the performance and reduced the overstrength relative to the prior four-storey building tested at E-defense where a stiffer wall-to-floor connection was used (Gavridou et al. 2017).

Modelling pre-diction and post-diction: In addition to the simplified analysis methods used in design, a more detailed non-linear analysis model of the test building was developed prior to testing. The planar frame models used fiber hinge elements to model the gap closing/opening mechanism at the post-tensioned walls base, a modified multi-spring model for the slotted-beam joints with/without energy dissipation devices, and other frame components and wall panels were modelled using elastic beam-column elements. The results of this original model are described by Gu et al. (2022).

Although the model was able to capture the general test building behavior, the calculated base moment capacity was significantly less than that measured during the test, affecting the accuracy of time-history simulations. Following the analysis of the test results, it became clear that there were numerous sources of additional strength. An updated model was developed to simulate the test building response more accurately and is reported by Gu et al. (2023). The revised model included several key modifications:

- The contribution from floor interaction was quantified based on two key mechanisms that are shown in Figure 4: 1) As the walls in the EW direction of the building uplift, the flexible wall-to-floor connections still provide some additional strength due to the Out-Of-Plane (OOP) deformations of the link-slab and composite floors; 2) Moment capacity at the floor to beam connections due to the continuity of the topping concrete and floor reinforcement.
- The flexibility of the energy dissipation device connection plates was modelled based on test results, reducing the deformations applied to the energy dissipation devices, and thus the energy dissipation.

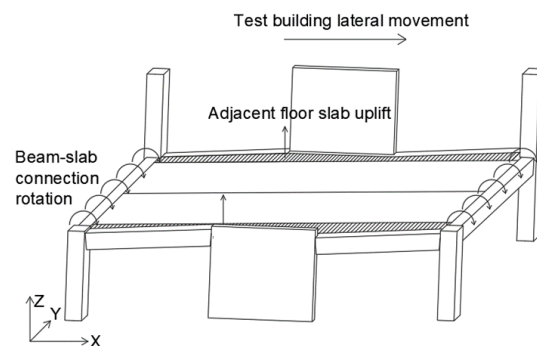


Figure 4: Mechanisms of floor strength contribution

The updated model results are compared to the original model results and the measured test response in Figure 5. The modifications to the model to account for the system-level behavior of the building resulted in an increased lateral strength that was closer to the measured strength, with the base moment strength of the models increased by up to 32% and 12% in the EW and NS directions respectively. The accuracy of the model for the lateral drift time-history response was also improved. Despite the improvement in the simulated response, discrepancies with the test response indicate that there are additional complexities that the model was unable to capture, highlighting the challenge of fully understanding the system level response of buildings.

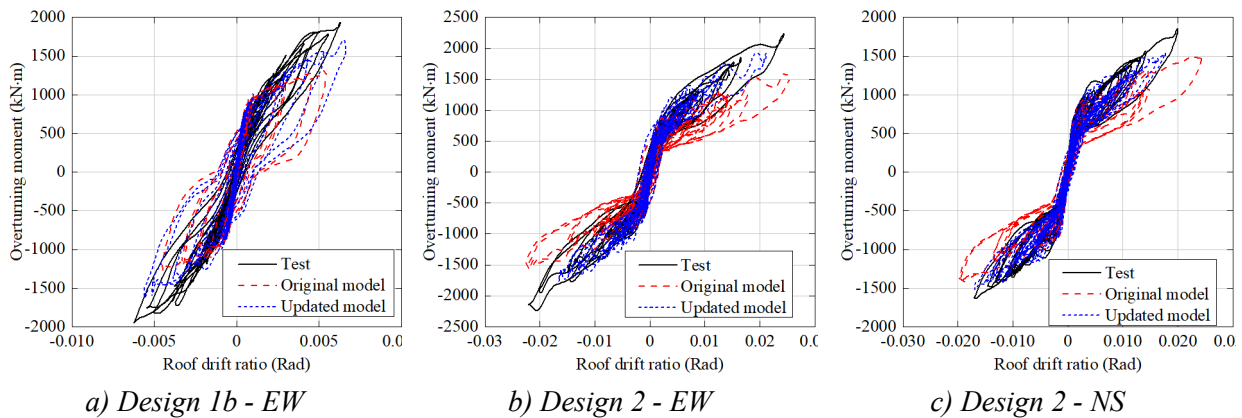


Figure 5: Original and updated calculated model responses compared to test results

Conclusions

A two-storey low-damage concrete building implementing state-of-art design concepts was designed and tested on the multi-functional shake-table array at Tongji University, as part of an ILEE-QuakeCoRE international collaborative project. The test building used a combination of post-tensioned walls and a frame that utilised slotted beam connections. The building was subjected to 39 tests consisting of both unidirectional and bi-directional ground motions in a range of different configurations. Key outcomes related to the response of the test building were presented, including the successful validation of the system level response and deformation compatibility of the test building design and component/connection detailing, and non-linear model results from an initial pre-test model and a modified model that incorporated explicit lessons from the test building response. Lessons from this test have resulted in recommendations to revise design procedures and connection detailing for such structural systems. Validation at a system-level was crucial to identifying key design requirements and the project serves as an example of the benefits of such large-scale tests for new structural systems.

Acknowledgements

The test building and subsequent research was funded by International Joint Research Laboratory of Earthquake Engineering (ILEE) at Tongji University, the Building Systems Performance branch of the New Zealand Ministry of Business, Innovation and Employment (MBIE), Concrete New Zealand, and Te Hiranga Rū QuakeCoRE, an Aotearoa New Zealand Tertiary Education Commission-funded Centre. Travel support from the New Zealand Society for Earthquake Engineering (NZSEE) to attend this workshop and present this research is gratefully acknowledged.

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RESILIENT DESIGN FOR FUNCTIONAL RECOVERY IN THE UNITED STATES: EMERGENCE OF NON-PRESCRIPTIVE RESILIENT DESIGN AND CURRENT EFFORTS TO CREATE PRESCRIPTIVE REQUIREMENTS

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Abstract

With the emergence of FEMA P-58 in 2012, elective resilient design started in the United States in 2014, and has gained traction over the past decade in the U.S. Through this use in the U.S. structural engineering industry (and abroad), FEMA P-58 has been vetted and refined, and now provides a tested and vetted tool for facilitating quantitative resilient design of buildings. This has culminated into an effort over the past ~3 years to create prescriptive requirements for resilient building design; this is currently being done as part of the U.S. National Earthquake Hazards Reduction Program (NEHRP) Recommended Provisions, led by the Building Seismic Safety Council (BSSC), and being facilitated by the 20-member Functional Recovery Task Committee of BSSC. This paper and presentation provides a summary of the FEMA P-58 Functional Recovery Module, which provides a strong technical basis for resilient design, provides an overview of elective resilient design in the U.S. over the past decade, and then summarizes the current efforts to create and codify prescriptive requirements for resilient design.

Emergence of Resilient Design over the Past Decade in the United States

Over the past decade, in the United States, it has become more widely known that the design objective of our building codes (e.g. ASCE 2022 [4]) is for buildings to be safe, but not necessarily to be functional or even repairable after an earthquake. Many have begun referring to code-compliant buildings as “safe but disposal.” In response to this, over the past several years, several efforts have been launched to focus on Building design for reparability and functionality, and “design for Functional Recovery” seems to be the working term for resilient design being used in the U.S. structural engineering community.

In parallel with this new awareness of what the building code is providing (and not providing), analytical methods have now been developed to the point that engineers are able to analytically estimate building damage and function through a building-specific engineering analysis. This was not possible 10 years ago. These supporting research efforts have occurred over the past couple decades (e.g. Moehle and Deierlein 2004, Porter 2003) and have coalesced with the 2012 release of the FEMA P-58 risk analysis method (FEMA 2021). This was then extended for repair time and downtime calculations (REDi 2013 and Terzic 2016), and led to the FEMA P-58 Functional Recovery Module (published in Cook 2022, and to be published by ATC/FEMA in 2024); the FEMA P-58 FR Module now provides the current state-of-art in seismic resilient design in the U.S. Enabling software tools are also now available to support these engineering analyses (FEMA 2012, and SP3 2024). Even as they were being developed, these new technologies have been electively used to design recent buildings to be resilient, aiming for limited building closure time and limited repair costs after the earthquake (e.g. Hata 2016, Almufti 2016). These FEMA P-58 analytical methods for resilient design have now been used, vetted, and iteratively refined within the U.S. structural engineering community for a full decade. An overview of this development is depicted in Figure 1 and is summarized below.



Figure 1. Overview of Research and Development Enabling Resilient Design Codification

The following are a fuller summary of the developments depicted in Figure 1:

- **~1997:** First “performance-based” U.S. pre-standard for retrofit.
- **2001:** FEMA initiates the FEMA P-58 project to create a performance-based design process for new building, accounting for repair costs, repair times, and safety.
- **2013:** Release of initial recovery time methods that built on top of FEMA P-58 (REDi and Terzic/PEER methods).
- **2014:** Release of the Seismic Performance Prediction Platform (SP3) software, which is a commercial software to enable use of FEMA P-58.
- **2015:** Launch of the U.S. Resiliency Council.
- **2016:** Release of the NIST Community Resilience Planning Guide
- **2018:** Major update to FEMA P-58 released.
- **2019-2020:** Large FEMA P-58 validation study conducted using ~2.2M buildings subjected to the 1994 Northridge earthquake.
- **2020-2021:** U.S. Congress sanctioned report entitled *Recommended Options for Improving the Built Environment for Post-Earthquake Reoccupancy and Functional Recovery Time*.
- **2022:** BSSC Functional Recovery Planning Committee convenes and large project is launched to create and codify prescriptive requirements for resilient design.
- **2024:** Complete draft design provision completed, and work ongoing.

The above two recent developments (societal interest and new engineering technology) have created a remarkable situation and opportunity. Structural engineers now have the technology to predict building damage and functionality with a vetted analytical method, and they can use that information to iteratively design buildings to be better (e.g. building functionality within weeks). Many levels of government are now making it clear that they desire post-earthquake functionality and smoother recovery for their

communities, rather than only safe (but disposable) buildings. This societal need, coupled with new technology available to structural engineers, puts our structural engineering profession on a good footing to make major steps toward improving our building design practices and create more resilient buildings and communities. This includes (a) continued use of elective (non-prescriptive) resilient design methods, but more importantly, (a) embarking now on a large effort to develop prescriptive and codified requirements for resilient design; the later will enable broad adoption of resilient design across the structural engineering industry (rather than resilient design being reserved for only large and special projects).

Resilient Design Using Non-Prescriptive Methods

When thinking about resilient design, there are approaches in the U.S. building code that try to make the building “better,” such as making the building stronger, stiffer, and/or enforcing the combined requirements for a higher Risk Category. However, the primary focus of the building code is to ensure safety and these requirements were created based on judgement and experience, and it has not been demonstrated that they actually deliver the desired resilience that they intend for high levels of earthquake shaking (e.g. the performance of the Oliveview Hospital building in the 1994 Northridge earthquake, Cheevers 1994). If a quantitative resilient design approach is desired, there are currently no consensus prescriptive design requirements that have been quantitatively shown to deliver a resilient building (though a current major effort in this regard will be discussed in the next section).

In the absence of prescriptive design requirement for resilience, a resiliency analysis can be conducted directly using the FEMA P-58 Functional Recovery Module (or prior versions of FEMA P-58), to demonstrate that the building meets the following goals for damage and recovery time after an earthquake. This is generally what has been done for the past decade for elective resilient design in the U.S. The FEMA P-58 Functional Recovery Module quantitatively estimates the repair cost and functional recovery time of the building, and can be used to iteratively design the building to meet stated resiliency goals (e.g. functional recovery time of one week or less). Table 1 shows some of the typical performance targets used for resilient design, typically with a requirement to meet these goals with 75th confidence.

This FEMA P-58 FR Module methodology is summarized in a later section; the methodology can be used directly for resilient design of an individual building, but can also be used to calibrate prescriptive methods for resilient design, as outlined in the next section.

**Table 1. Typical Target Functional Recovery Design Requirements
(from draft code requirements discussed in later section)**

Functional Recovery Category	Target Functional Recovery Time	
	General Expectation	Acceptance Criteria
A	Immediate	0 hours
B	Within a few days	72 hours
C	Within a couple of months	2 months
D	Within half a year	6 months

Creating Prescriptive and Codified Resilient Design Requirements – Current Work and Status

In addition to a method for non-prescriptive resilient design, the building industry also (perhaps more importantly) needs codified resilient design standards, in order to meet growing demand for earthquake-resilient buildings. The substantial growth of resilient design over the past decade has been spurred mostly by institutional and long-term owners, for which quick post-earthquake functionality is important. In parallel with this, public awareness of the need for resilient design has continued to increase, and the U.S. Congress sanctioned a two-year study on this topic, which produced a seminal report in January 2021, entitled *Recommended Options for Improving the Built Environment for Post-Earthquake Reoccupancy and Functional Recovery Time* (FEMA/NIST 2021); a key finding and recommendation of this report is that codified standard are needed to enable industry-wide of resilient seismic design methods. Additionally, major institutions like UC San Francisco have decided to “go it on their own” and are currently attempting to create their own resilient design requirements, due to lack of standardized provisions.

To meet this need for prescriptive resilient design requirements, a Functional Recovery Task Committee (FRTC) was formed under the leadership of the Building Seismic Safety Council (BSSC). This twenty-member Task Committee, with ~40-50 additional members among five task sub-committees, have worked over the past ~2.5 years to create prescriptive resilient design building code provisions (NIBS, 2023). To create these prescriptive resilient design requirements, the BSSC FRTC members are iteratively running FEMA P-58 FR Module analysis cases on a wide range of building archetypes and design cases. This work has culminating in a complete draft document, which is currently being balloted as a voluntary appendix by the BSSC Provisions Update Committee (PUC); once approved, this would then go into the 2026 NEHRP Provisions and then passed to ASCE 7-28. The next step is to resolved voting comments and finalize this voluntary appendix, and complete this current volunteer phase of work.

Creating Prescriptive and Codified Resilient Design Requirements – Needs and Next Steps

The current resilient design provisions include a full framework and foundation, and complete set of design provisions for some structural system. However, due to this building code development work being performed to date on a volunteer basis, the requirements do not yet exhaustively cover resilient design requirements for all structural systems. This presents a remaining one-year window of opportunity to further develop these resilient design provisions, primarily by evaluating as many structural systems as possible, to make the provisions as complete and comprehensive as possible in this formative code cycle. These structural system studies can then result in individual modification proposals that can add to the approved document from the volunteer phase of work, to make it more complete and comprehensive in this cycle (and to further test and validate the provisions in the process). The next opportunity to make this impact will be in the next code cycle for ASCE 7-34 (6 year delay), so we are currently organizing funding and efforts for a strong final one-year push on this work, to bring an expanded and comprehensive chapter across the finish line in this code cycle.

The FEMA P-58 Functional Recovery Module (the technical analysis basis that underpins this all)

The FEMA P-58 project was initiated in 2001, and the FEMA P-58 analysis methodology was first released in 2012 (FEMA, 2012). This methodology accounts for all of the important components of resilience – ground motion hazard, structural responses (with uncertainties), assessment of damage to building components (with uncertainties), identification of which component damage inhibits functionality, and assessment of repair cost and repair time to building components and resulting repair time for the full building (with uncertainties), and consideration of the effects of residual drifts. An

update to the methodology was released in 2018, and then the FEMA P-58 Functional Recovery Module was built between NIST and FEMA funding from ~2020-2024. This methodology has been documented by Cook et al. (Cook, 2022), has been released by the ATC/FEMA project in draft report form, and the final project report is expected to be released by ATC/FEMA this year.

Figure 2 provides an overview of the FEMA P-58 analysis method, including the FEMA P-58 Functional Recovery Module, and the new Functional Recovery Module has been made publically available by Cook et al. on GitHub (<https://github.com/OpenPBEE/PBEE-Recovery>). The FR Module includes a full construction scheduling sub-module, as depicted in Figure 3, which includes both direct repair times as well as impeding times that delay the onset of repair. The FEMA P-58 Functional Recovery Module include numerous fault trees that related component-level damage to system-level function, with Figure 4 providing an example for the electrical system in the building. This provides a simple and quick overview, and much additional technical detail is available in the Cook et al. paper, the draft and forthcoming final ATC/FEMA report, as well as all the code being available and documented in the GitHub repository.

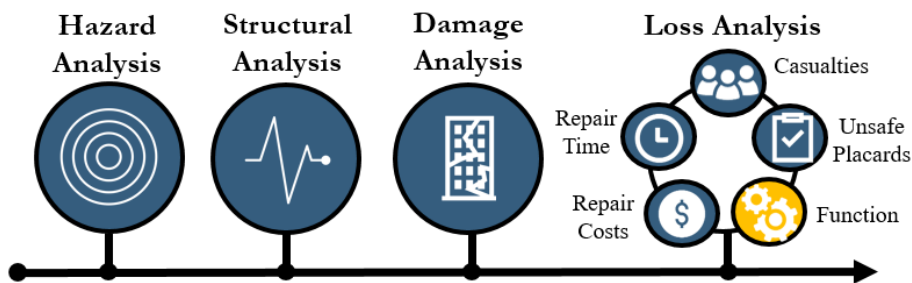


Figure 2. Overview of the Steps in FEMA P-58 analysis, including the P-58 Functional Recovery Module

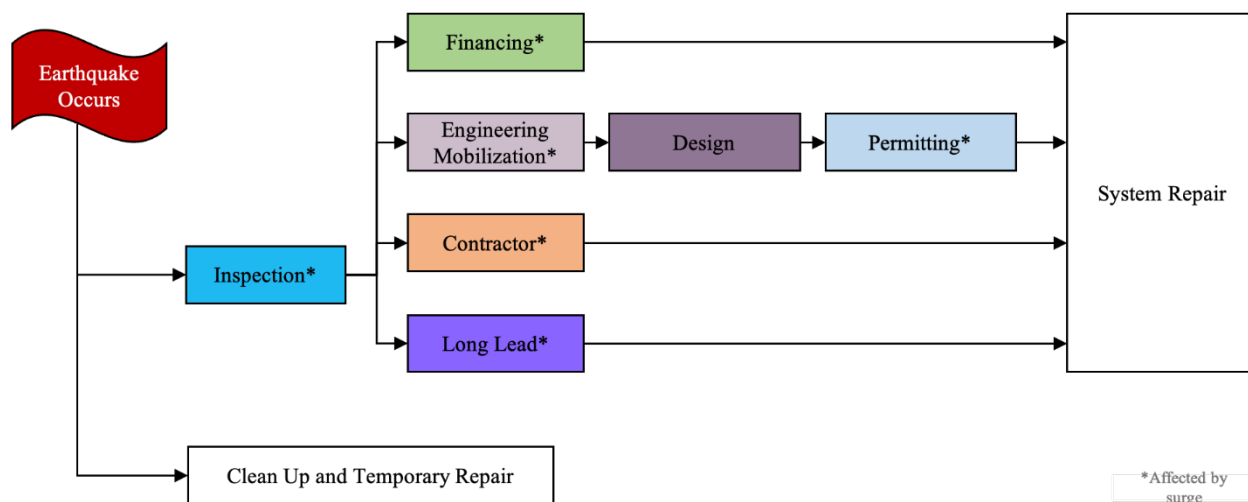


Figure 3. Overview of the Repair Time Scheduling in the P-58 Functional Recovery Module

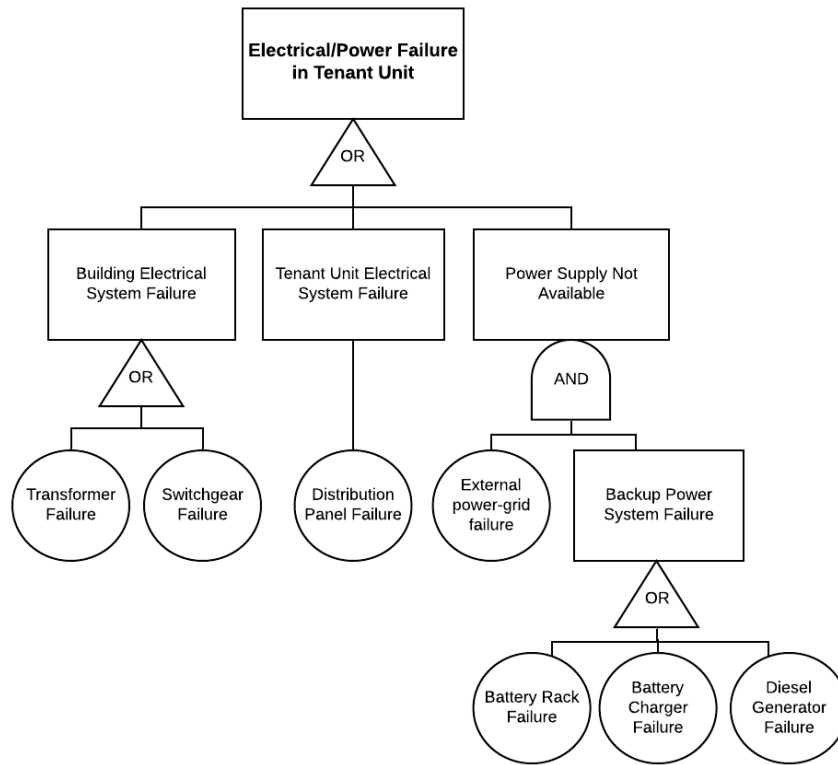


Figure 4. Sample Fault Tree for Electrical Subsystem

Figure 5 provides sample results from an analysis done using the FEMA P-58 Functional Recovery Module. This shows the median recovery times as a function of peak group acceleration, including three stages of recovery – re-occupancy, functional recovery, and full repair. These results are characteristic of a code-minimum building design, where the building was just designed for safety and not for a quick functional recovery.

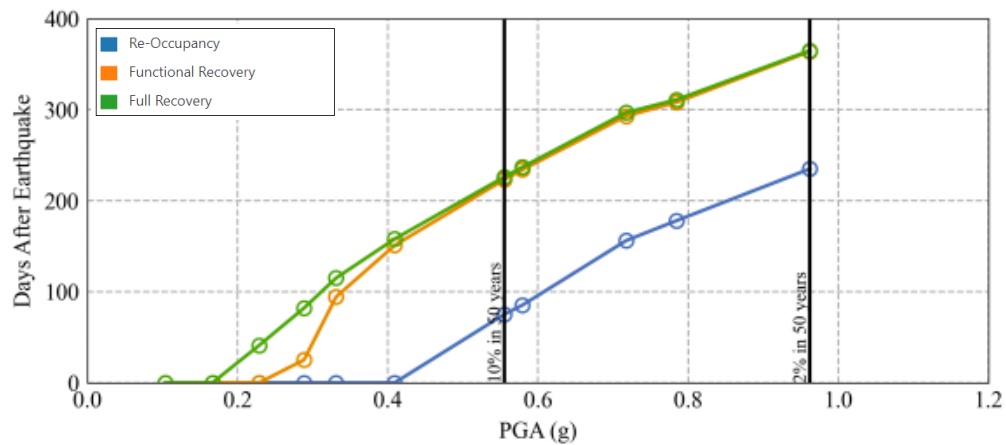


Figure 5. Sample Results from the FEMA P-58 Functional Recovery Module

Summary and Next Steps

The primary next steps are to make as much progress as possible in the next ~1 year, within this current U.S. code cycle. The current resilient design provisions are on their way toward a complete voluntary appendix, and additional development efforts would be useful to make the provisions are complete as possible in this code cycle (so as to not have a ~6 year delay to the next cycle).

Acknowledgements

This paper provides a broad overview of work being done in the United State to push forward resilient design for functional recovery. This overview includes work by hundreds of contributors, all of which cannot be named here. Specifically notable contributions to this body of work include the papers already cited in this report, as well as the FEMA P-58 (ATC-138) project funded by the U.S. Federal Emergency Management Agency (FEMA) and run by the Applied Technology Council (ATC), as well as the FEMA-funded work of BSSC and the related Functional Recovery Task Committee (and all participating subcommittee members).

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QUALITY ASSURANCE OF NONSTRUCTURAL COMPONENTS FOR FUNCTIONAL RECOVERY DESIGN

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Abstract

It has been long recognized that seismic performance of nonstructural components plays a significant role in functional recovery of structures. Current life-safety codes treat many nonstructural components without regards for post-earthquake performance. Furthermore, many engineers suggest, based on anecdotal and physical evidence, that nonstructural components and systems may not be code-compliant when installed into structures.

As part of an ongoing project under FEMA's National Earthquake Hazards Reduction Program (NEHRP) Architect-Engineer Contract, the *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures* has formed a Provisions Update Committee (PUC) under the management of the Building Seismic Safety Council. The PUC is developing potential code language for the groundbreaking concept of functional recovery. As part of this effort, a working group on quality assurance is developing provisions to ensure proper design and construction of nonstructural components and systems.

Workshops with structural engineers, architects, fabricators, and building officials informed issues to be addressed. The goal was to assess current state of construction quality and assurance and propose improvements and modifications to current procedures for structures targeting functional recovery objectives. The subsequent provisions include requirements on documentation, coordination and review of nonstructural seismic design during the design and construction phase, special inspection requirements and procedures, and requirements for engineering observations. It further suggests additional considerations for shake-table seismic certification in non-mandatory language. These requirements are set to ensure proper design and installation of nonstructural components and systems, enhancing the post-earthquake functional recovery of the structure. Final results, conclusions, and recommendations are still pending.

Introduction

Modern seismic design codes and standards, such as those in the United States, generally target life-safety goals for most buildings by providing a margin against collapse. To achieve acceptable performance margins, designers generally seek to dissipate the earthquake's energy through structural system ductility, resulting in damage to the structural components. Similarly, the codes also treat nonstructural components as replaceable and only target life-safety, with limited protection against component damage for most structures. Observations from past earthquakes, both in the U.S. (Loma Prieta, 1989; Northridge, 1994; Napa, 2014) and internationally (New Zealand, 2010/2011; Turkey, 2023), have demonstrated that many buildings, even designed to modern seismic design codes, may incur severe damage, rendering the asset unusable and often requiring significant repair or replacement.

The repair or reconstruction of buildings following earthquakes can take months or even years and can severely delay community recovery. As such, it has been proposed that the seismic design of buildings should target functional recovery performance in addition to life-safety goals (NIST & FEMA, 2021).

Functional recovery here is defined as a state in which the building is ready to support its basic intended function (NIST & FEMA, 2021).

Currently, there are no codified approaches for the seismic design of buildings targeting functional recovery objectives. Even design guidelines such as Arup's Resilience-based Design Initiative (REDi) still rely on performance-based approaches with explicit quantification of downtime using methodologies such as ATC-138 (Cook et al., 2022), REDi (Almufti and Wilford, 2013; Paul et al., 2018), TREADS (Molina-Hutt et al., 2022), or F-Rec (Terzic et al 2021). In general, the design of buildings to achieve functional recovery objectives quickly after an earthquake typically focuses on reducing a building's response to the ground shaking. For example, reducing floor accelerations and interstory drifts and increasing component capacities reduce the likelihood of severe building damage that leads to loss of function. Relative to life-safety design, the design for functional recovery requires more attention to the performance of the structural components at lower ductility demands, and the design and selection of the nonstructural systems and components.

Building Seismic Safety Council (BSSC) Provisions Update Committee (PUC) established a Functional Recovery Task Committee (FRTC) that is currently developing first generation seismic design provisions for functional recovery performance. The intent of this effort is to include these provisions in 2026 NEHRP Recommended Seismic Provisions for New Buildings and Other Structures. These provisions will inform the development of the 2028 edition of ASCE 7: Minimum Design Loads for Buildings and Other Structures, and ultimately, the 2030 edition of the International Building Code (IBC).

During the development of the provisions, it has been recognized that successful implementation of a functional recovery design objective – targeting low damage of structural elements and operational performance of nonstructural systems – may require a higher level of construction quality. Current material standards and the building code (such as IBC) primarily focus on the quality of structural elements and the prevention of element failure. However, designing for functional recovery requires an increased focus on the performance of structural elements at the onset of damage and prevention of damage to gravity and nonstructural elements. This design shift may require new approaches to construction quality assurance, with additional considerations for nonstructural systems and components. In California, the California Department of Health Care Access and Information (HCAI) and Division of State Architects (DSA) pioneered increased quality requirements for the construction of hospitals and public schools. These requirements serve as the basis for suggested changes to construction quality assurance practice within this paper.

The work presented in this paper was undertaken primarily by FRTC Task Subcommittee 4 and QAQC Task Group. Within this effort, the groups first identified that the shortcomings of the current construction quality assurance in United States especially as they relate to nonstructural components, and then proposed several new requirements to address these issues. This paper first summarizes the current construction administration process in United States, then highlights the shortcomings of this process, and lastly introduces proposed changes.

Current Construction Administration Practice in United States

Following section is adopted from Valigura et al. (2024) to summarize construction administration practice in United States.

At the end of the structural design process (usually corresponding to the end of the Construction Documents phase), a set of structural drawings, calculations, and specifications related to structural design is issued by the Structural Engineer of Record (SEOR) and submitted as part of a permit submittal

to the building department for approval and permit issuance. It should be noted, that while the structural drawing set is complete with respect to the primary structure, there are still components, such as cladding, stairs, MEP (mechanical, electrical, and plumbing) supports, and seismic bracing that may be deferred for design after the permit is issued and/or delegated to other parties for design.

Once the permit is issued, the project moves into the construction phase. During this phase, several entities are involved from the structural engineering perspective. The SEOR enters the Construction Administration phase. This phase usually involves coordination with respect to the construction of the primary structure, such as response to Requests for Information (RFIs) and change orders, issuance of construction sketches where necessary, structural observations, and review of submittals. In parallel, the contractor(s) perform the work, and, in many cases, hires component engineer(s) of record (CEOR) to perform delegated design. The CEOR is responsible for the design and oversight of proper installation of the designated component or system and its seismic restraints, bracing or anchorage. The owner is required to hire a special inspector to verify the structure is constructed according to contractual documents.

The design and construction requirements are governed by state or local jurisdiction building codes. In the United States, these codes are usually based on International Building Code (2021 IBC, hereafter referred to as IBC). However, jurisdictions can selectively adopt and/or amend different sections of the IBC. The following list highlights requirements in the IBC pertaining to construction administration processes and the construction phase as related to comments and observations from expert panel discussions highlighted later in the paper.

Chapter 1 Scope and Administration, among others, provides the requirements for Code Compliance Agency, Permitting, and Construction Documents. Specifically:

- Sections 103 and 104 define Building Officials and their duties. These duties include, but are not limited to, interpretation and enforcement of the code, review of construction documents, issuance of permits, and inspections.
- Section 107 summarizes construction documents that need to be included with a permit. These include construction documents, statements of special inspections, geotechnical reports, and any other data.
- Section 107.3.4 defines Registered Design Professional (RDP) in Responsible Charge. In many cases, this is the Architect of Record (AOR).
- Section 107.3.4.1 requires deferred submittals to be reviewed by RDP prior to sending to the building official. Work shall not be installed until approved by the building official.
- Section 107.4 requires any changes made during construction that are not in compliance with approved construction documents to be resubmitted for approval by the building official.

Chapter 17 Special Inspections and Tests describes the requirements for the testing agency, nature of special inspections and testing that need to be provided during the construction, and structural observations. Specifically:

- Section 1704.2 requires the owner or the owner's representative, other than the contractor, to employ an approved agency to provide special inspection.
 - Per Section 1704.2.1. the approved agency needs to provide documentation to the building official about competence, training, and experience of the special inspectors.
 - Section 1704.2.4 requires the approved agency to submit reports of special inspection to the building official and RDP in Responsible Charge.
- Section 1704.3 requires the RDP in Responsible Charge to prepare a statement of special inspection that includes:

- The materials, systems, components, and work required to have special inspection or tests.
- The type and extent of each special inspection, and whether it is continuous or periodic.
- Additional seismic or wind requirements.
- Identification of designated seismic systems (1704.3.2).
- Section 1704.4 requires the contractor to acknowledge the additional wind and/or seismic construction requirements.
- Section 1704.5 provides additional submittals that the approved agency needs to submit to the building official.
- Section 1704.6 provides requirements for structural observations.
- Section 1705 provides requirements for special inspections and tests, including but not limited to special inspections and tests for:
 - Concrete construction (Section 1705.3)
 - Seismic resistance (Special inspection – Section 1705.13, Testing – Section 1705.14), including requirements for special inspection of Designated seismic systems, architectural components, and MEP components, and testing (where required) of nonstructural components and designated systems.

In California, healthcare facility and public-school construction is under the oversight and jurisdiction of the Department of Health Care Access and Information (HCAI) and the Division of State Architects (DSA), respectively. These entities amend and enforce the California Building Code to provide a higher standard of design and construction. Both HCAI and DSA provide enforcement of the code regulations and act as the building official. For simplicity and due to the often more stringent requirements, only HCAI requirements/amendments are referenced in this paper. A selection of HCAI requirements/amendments pertaining to the scope of this paper are highlighted below. Additionally, HCAI recognizes several facility types (OSHDP 1 through 5), each with different requirements. However, this differentiation is not made in this paper.

HCAI regulations and design and construction requirements are governed by the California Administration Code (CAC 2022, hereafter referred to as CAC) and the California Building Code (2022 CBC, hereafter referred to as CBC).

- CAC Section 7-141 provides requirements on construction administration, including the Testing, Inspection, and Observation (TIO) program and the responsibilities of the engineer to develop and administer the TIO and provide observations.
- Testing, Inspection, and Observation (TIO) Program requirements are provided in CAC Sections 7-141, 7-145, 7-149 and 7-151.
 - The TIO Program must be approved prior to building permit approval.
 - The TIO specifies the manner, frequency, duration, and reporting of all required testing, inspection and observations activities.
 - The architect or structural engineer in charge of the TIO program administration.
 - Intervals or specific milestones when the design professional, contractor, approved agency, and inspector of record verify and document compliance with construction documents using Verified Compliance Reports (CAC Section 7-151).
 - The engineer responsible for the preparation of construction documents is required to observe the construction of work for their portion of the project.
- The Inspector of Record (IOR) (one or more) is provided for the project according to CAC Section 7-144.
 - CAC Section 7-204 provides a classification of hospital inspectors and minimum qualification for each class.

- CAC Section 7-144 requires that the IOR is approved by the structural engineer and HCAI.
- CAC Section 7-145 requires that the IOR has personal knowledge of the project and provides continuous inspection of all parts of the construction work at all stages.
- CAC Section 7-145 requires that the IOR maintains field records of construction progress for every day they are on site.
- The list of other duties of the IOR is provided in CAC Section 7-145.
- CAC Section 7-153 provides requirements for changes to the approved work. The section clearly highlights what changes are considered as changes that do not materially alter the work. These changes do not need to be submitted to the Building Official for approval and can be approved by HCAI field staff. Changes that materially alter the work, as determined by HCAI field staff, need to be submitted for approval.
- CAC Chapter 6, Section 11.2.3. requires the development of an Operational plan stating, among other information, areas needed for continuous operations and the nonstructural systems that are serving them (note Operational plan is only required for NPC 4D retrofit).
- CBC Section 1704A.2 requires the inspector to act under the direction of the architect or structural engineer or both (note only applicable to OSHPD 1 and 4).
- CBC Section 1704A.3.2 requires identification of equipment/components that require special seismic certification (modification to IBC that requires identification of designated seismic systems).
- CBC Section 1705A provides many changes to the specifics of special inspection and testing. Listing all of these changes is beyond the scope of this paper. Many of the subsections extend the special inspection and testing on components that are otherwise exempt in IBC. More focus is given to nonstructural components, where active or energized equipment is required to be certified by shake-table testing (1705A.14.3), equipment that must be tested is explicitly listed (1705A.14.3.1, only for OSHPD 1 and 4), and post-installed anchors for nonstructural components needs to be tested (1705A.3.4 – note that testing requirements are then in 1910A.5).

Identified Deficiencies

The group identified several deficiencies in current practice as outlined in Table 1. This table also links the deficiencies to suggested provisions highlighted in next section. The overarching concern is about proper enforcement of design, review, and inspection requirements on nonstructural components, especially components that are deferred and delegated design items.

Specific to special inspection of nonstructural components, the major shortcomings were found in the fact that while for major structural materials, the code presents the special inspection requirements in table form that can be readily copied on contractual documents, that is not the case for nonstructural components. Further, the group agreed that there are currently no clear measures to track and verify that required special inspection was performed.

Often, there is not a clear coordination between different trades on location of different nonstructural systems and their seismic restraint, again especially for deferred and/or delegated design. What happens in practice is often that the systems are installed uncoordinated, which can significantly affect their seismic performance.

The group also pointed out couple of specific issues with current requirements, such as vague definition of continuous and periodic special inspection, and requirements on field testing of post-installed anchors in concrete and masonry.

Table 1. Construction QA deficiencies and their solutions

<i>Deficiency</i>	<i>Requirements</i>
Lack of review of changes during construction	Functional recovery coordinator
Lack of special inspection tracking	Acceptance reports Functional recovery statement of responsibility
Lack of nonstructural component coordination and ownership for performance	Functional recovery coordinator System coordination
Need for construction team “buy-in”	Pre-construction functional recovery conference Functional recovery statement of responsibility
Lack of ownership of observations	Final written statement of observations
Unclear requirements on construction quality assurance	Quality assurance plan
Vague definition of periodic and continuous special inspection	Required special inspection and testing
Difficulty with prescribing special inspection requirements for nonstructural components	Special inspection of nonstructural components, systems, equipment and their anchorage
Testing of post-installed anchors for nonstructural components not required	Testing of post-installed anchors
Lack of planning for required post-earthquake actions	Post-earthquake functional recovery plan

Suggested Provisions

The entire requirements for seismic design for functional recovery are being written as a stand-alone chapter. Within the chapter, the sections can be separated into four types: general requirements, structural design requirements, nonstructural design requirements, and associated criteria. The construction QA requirements are located in the associated criteria part of the chapter spanning three different sections. Following are the highlights of the proposed associated criteria requirements.

Functional recovery coordinator. The provisions identify a new role within the design team – functional recovery coordinator. Within this role, the functional recovery coordinator is responsible for review, acceptance, and coordination of the documentation, observation, quality assurance, and recovery plan for all elements, components, equipment, distribution systems. The functional recovery coordinator provides a holistic perspective of the entirety of the functional recovery aspects of a project and can prevent items necessary for functional recovery from “slipping through the cracks”. In general, it is expected that on many projects, the structural engineer will act as the functional recovery coordinator.

Among some of the specific duties of this role belongs: 1) Review and acceptance of construction documents for functional recovery requirements, delegated design submittals, equipment seismic certification, quality assurance plan, acceptance reports, and others, 2) leading pre-construction functional recovery conference and post-construction functional recovery walk-through, 3) development of schedule of milestones when acceptance reports need to be submitted.

Acceptance reports. To help with ensuring that the project is constructed according to construction documents, including nonstructural components and systems, the provisions require submittal of acceptance reports at several construction milestones. The acceptance reports are signed statements by general contractor, structural engineer, component engineers (where applicable), and special inspector(s)

that the construction has been performed according to construction documents. The acceptance reports are based on “own personal knowledge” defined within the provisions.

Contractor responsibility. Contractor responsibility includes functional recovery statement of responsibility, and system coordination.

Functional recovery statement of responsibility is a signed statement by general contractor where they acknowledge the awareness of added requirements brought by functional recovery performance objective, that these requirements were communicated to the subcontractors. The general contractor is also responsible for developing and maintaining special inspection schedule.

In addition, general contractor is responsible for coordinating the construction of nonstructural systems. The provisions provide three options: 1) coordination drawings (or BIM model) to detect any clashed between the trades, 2) “zoning drawings”, which essentially provide zones for each of the systems within the system must be installed, or 3) other means as approved by the functional recovery coordinator

Pre-construction functional recovery conference. The pre-construction functional recovery conference is organized by functional recovery coordinator. The pre-construction functional recovery conference(s) serve to ensure that the functional recovery requirements are passed from the design team to the construction, observation, and inspection teams. For example, pre-construction conference(s) may include reviewing together the seismic certification requirements for HVAC equipment or helping educate the contractor on expected additional functional recovery observations required (e.g., more frequent visits by architect to observe partition wall metal stud construction matches detailing).

Quality assurance plan. The functional recovery coordinator is responsible for coordinating the development of the quality assurance plan. In general, the quality assurance plan is expected to include the list of components required for basic intended function, requirements on special inspection and testing (type, frequency, distribution of reports etc.), requirements on observations, and others.

Special inspection of nonstructural components, systems, equipment and their anchorage. The proposed requirements expend current provisions on special inspection of nonstructural components. Table 2 shows illustrative example of special inspection requirements for potable water and sanitary waste plumbing systems. The table in the provisions is further expended on large number of nonstructural components and systems.

Table 2. Illustrative Example for Special Inspection Requirements for Nonstructural Components

Item	Special Inspection		Reference	Functional Recovery Category			
	C	P		A	B	C	D
Potable water and sanitary waste plumbing systems							
Potable water distribution piping		X	24.14.4.1.4	X	X	X	X
Pumps, valves		X	24.14.4.1.5	X	X	X	
Sewer distribution and vent piping		X	24.14.4.1.4	X	X	X	X
Drain distribution and vent piping		X	24.14.4.1.4	X	X	X	X
Wet wells, sumps, pumps		X	24.14.4.1.5	X	X	X	
Others		X	24.14.4.1.10	X	X	X	X

In addition, the provisions clearly prescribe the actions that the special inspector needs to perform during the inspection. For example, following are minimum actions require for special inspection of post-

installed anchors in concrete or masonry: 1) confirm anchor type (mechanism, material, diameter, embedment), 2) confirm conformance with design document (edge distance, anchor spacing, hole diameter in anchored component is appropriate for anchor diameter, use of washers, etc.), 3) Confirm installation procedure is in conformance with Manufacturer's Printed Installation Instructions (MPII).

Conclusions

This paper highlights the increased and added requirements on construction quality assurance within the seismic design for functional recovery. The added requirements primarily focus on nonstructural components and systems, and enforcement of their design, coordination, special inspection and observations. The provisions introduce new design team role – functional recovery coordinator, means to track special inspection, and specific requirement on special inspection of nonstructural components and systems.

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DETAILING FOR SEISMICALLY RESILIENT STEEL STAIR SYSTEMS: VALIDATION IN THE MASS TIMBER (10 AND 6-STORY) PROGRAMS

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Abstract

A series of shake table tests were recently conducted on full-scale 10-story and 6-story mass timber buildings at the 6-DOF Large High-Performance Outdoor Shaking Table facility at the University of California San Diego. Stairs, providing the primary egress in and out of a building during and after an earthquake event, were incorporated in each of these building test programs. To ensure they support the immediate recovery of building function, a variety of drift-release details were incorporated. Previous earthquake events and experimental studies have shown that stairs are among the most drift-sensitive nonstructural systems and are prone to damage, therefore relieving interstory drifts is paramount to improving their performance. To this end, the designed drift-release connections within the stairs considered the test buildings response during earthquake motions scaled at various hazard levels with expected minor and repairable damage under large earthquake loading. This paper provides an overview of the shake table test programs from the perspective of the design and performance of resilient steel stairs.

Introduction

The robust performance of a building during an earthquake and its recovery to functionality after the event strongly depend on its nonstructural systems, particularly the egress system. Previous earthquake events have documented the significant damage and often total collapse of stair systems. For example, Li and Mosalam (2013) summarized the damage to reinforced concrete stairs during the 2008 Wenchuan earthquake. Bull (2011) reported damage to both concrete and steel stairs during the 2011 Christchurch earthquake. Both studies highlighted the poor performance of stairs with fixed connections. In addition, experimental investigations of the seismic response of stair systems, such as Simmons and Bull (2000), Higgins (2009), and Wang et al. (2015), have also revealed that the performance of these systems under lateral load strongly depends on the flexibility of their connections. Indeed, it has been consistently reported that stairs fixed at each floor-to-floor connection results in damage to both the stair system and potentially the supporting structure. To this end, ASCE 7 (2016 and 2022) requires that egress systems, when not part of the lateral force resisting system, be designed such that they accommodate interstory drift without loss of gravity support. Therefore, Black et al. (2017, 2020) studied the performance of several types of drift-release connections incorporated into a full-scale single-story straight-run stair under quasi-static and dynamic loadings. The drift release connections in these studies included slotted connections with a single degree of freedom, free connections with multiple degrees of freedom (DOF), and drift-compatible (*DriftReady*TM) connections with three DOF. The stair system with a fixed-free configuration received significant damage to its connection plate. The stair system with slotted connections performed well under small to medium levels of shaking, but experienced binding when subjected to very large earthquake loading. On the other hand, stairs with drift-compatible connections showed superior performance. Nevertheless, the performance of these drift-release connections at a system level was still untested. Ultimately, the promise of such connections (as well as other types to facilitate drift-release) is best validated courtesy of full-scale testing. To this end, 10-story and 6-story prefabricated steel stair towers were incorporated into the shake table testing of three different full-scale

mass timber buildings. This paper provides a brief overview of these programs, summarizes key building responses with respect to the seismic demands imposed on the stair system; and, ultimately, aims to shed light on the performance of stair systems.

Shake Table Test Programs

Mass Timber Buildings. In 2023 and 2024, a series of mass timber building shake table test programs were conducted at UC San Diego. Initially, a 10-story mass timber building was constructed and tested as part of the NHERI TallWood project (Figure 1a, Pei et al., 2024). In this project, a pair of post-tensioned rocking walls made of Cross Laminated Timber (CLT) and Mass Ply Panel (MPP) were used as the lateral load-resisting system in the east-west (X direction) and the north-south (Y Direction) directions, respectively. The building, which had a height of 34.1 m, floor plan of 10.5×10.4 m, and total weight of approximately 328 metric tons, including 51 metric tons of foundation, was built on top of the 6-DOF Large High-Performance Outdoor Shake Table (LHPOST6). In this project, U-shaped Flexural Plates (UFPs) were used to dissipate rocking-induced energy. For more details on the design of the structural system, refer to Busch (2023), Huang (2023), Wichman (2023), and Pei et al. (2024). Vertically distributed nonstructural systems were also incorporated into the 10-story building. These included a full-scale operable 10-story stair tower with various drift-release connections (Sorosh et al., 2022a, 2024), exterior and interior cold-formed steel wall subassemblies (Ji et al., 2024 and Roser et al., 2024), and a two-story fire-rated curtain wall subassembly (Wynn et al., 2024). Subsequently, as part of the NHERI Converging Design project, the bottom 6 stories of the TallWood building were preserved and tested in multiple phases (Figure 1b). The 6-story building had a height of 20.7 m and a total weight of approximately 224 metric tons, including 51 metric tons of foundation. In the first phase of the Converging Design project, the UFPs and PT bars and tensioning force were replaced to return the lateral force resisting system closer to an undamaged state after testing of the 10-story building. In the second phase, the MPP walls (Y-direction) were replaced with new MPP walls detailed to receive buckling restrained boundary elements (referred to as BRBE in this paper) at the base of the rocking walls. In addition, during this phase, the nonstructural walls were removed. For more detail on the design and performance of the two phases of the 6-story building refer to Barbosa et al. (2024), and McBain et al. (2024).

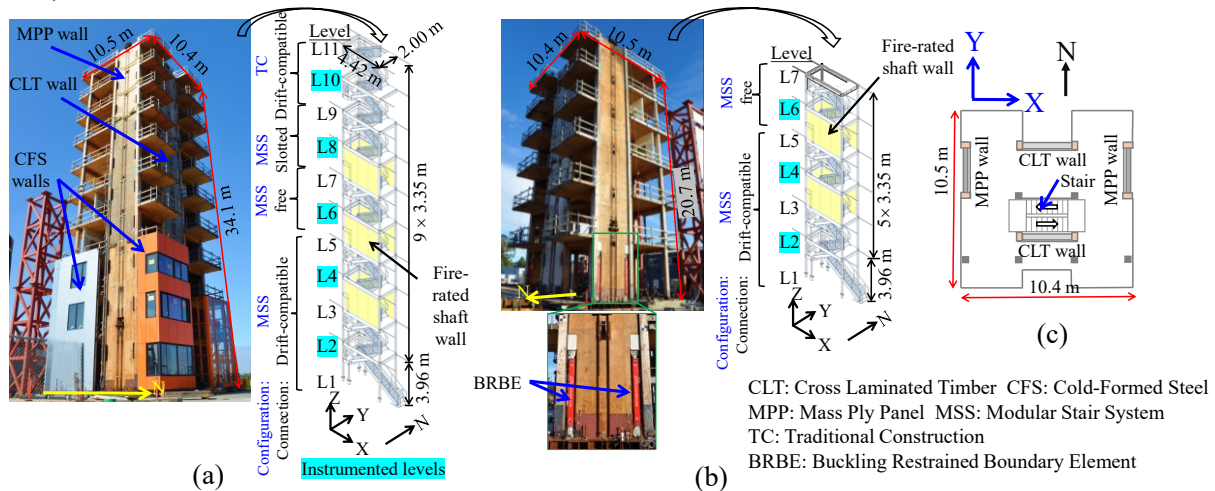


Figure 1. (a) NHERI TallWood building and the 10-story stair tower, (b) NHERI Converging Design building (phase 2: BRBE in Y-direction) and the 6-story stair tower rendering, and (c) floor plan

Stair Systems. The stair tower was placed within the central region of the floor plan such that its strong axis (longitudinal) was in the east-west direction (X Direction), and its weak axis (transverse) spanned in the north-south direction (Y Direction). The stair system incorporated into the 10-story building included the bottom eight stories of the Modular Stair System (MSS) and the top two stories of stairs built with Traditional Construction (TC), see Figure 1a. This test program featured three types of drift-release

connections, namely, *drift-compatible (DC)* connections with three translation degrees of freedom (DOF), and one rotational DOF, *free connection (FC)* with three DOFs, and *slotted connections (SC)* with one DOF. Each of these drift-release connections was placed such that they were subjected to 50% of the interstory drift. Figure 2 depicts various connections details, featured in these test programs, with degrees of freedom shown in green arrows. Sorosh et al. (2022b and 2024) provides a detailed discussion of the design details of the stairs in this test program. Subsequently, as shown in Figure 1b, in the two phases of the 6-story building configuration shake table test programs, the bottom 6 stories of stairs were preserved with no modifications to their configuration. It is notable that the fundamental period of the 10-story building was 1.84 sec in the X-direction and 2.03 sec in the Y-direction; while that of the 6-story (in phase 1) was 1.14 and 1.04 sec, in X- and Y-directions, respectively.

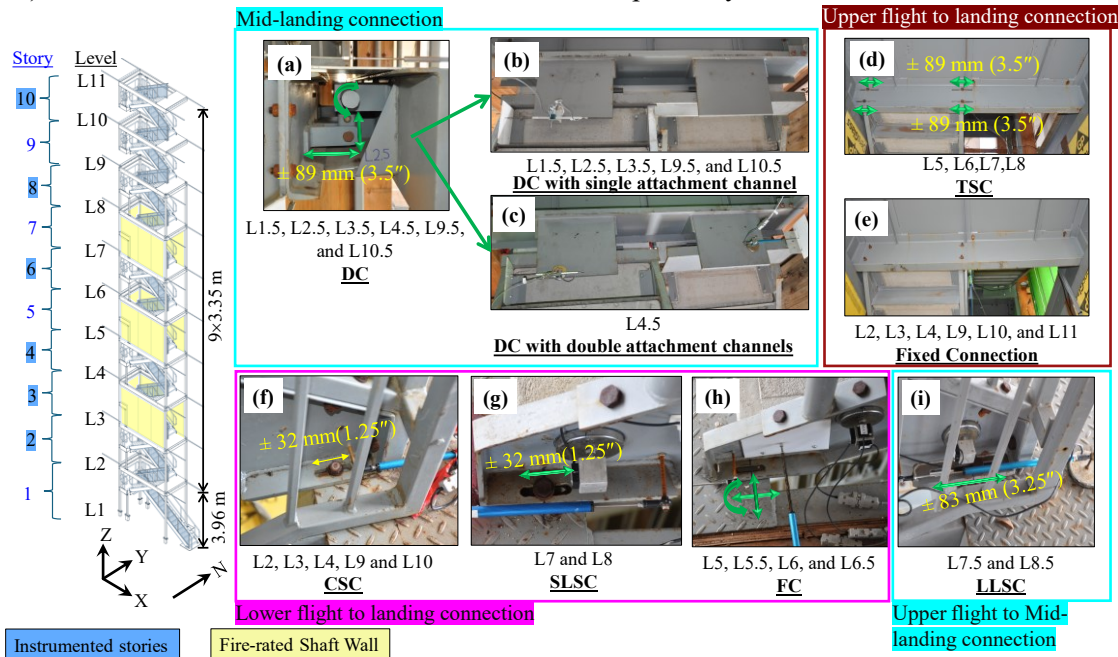


Figure 2. 10-story stair connection variations (a), (b), and (c) drift-compatible (DC), (d) transverse slotted connection (TSC), (e) fixed connection, (f) construction slotted connection (CSC), (g) short longitudinal slotted connection (SLSC), (h) free connection (FC) and (i) long longitudinal slotted connection (LLSC). [note that (1) details for L1-L7 were preserved when the building was in its 6-story configuration and (2) instrumented stories are highlighted in blue in the stair tower rendering].

Selected Input Motions. The TallWood (10-story) building was subjected to 88 earthquake motions scaled to various risk-targeted levels of intensity. These ground motions were scaled for site class C, risk category 2 with a selected site in downtown Seattle, WA. These motions were selected from past earthquake events representing distinct seismic sources. The selected motions were amplitude-scaled to risk-targeted events with return periods (RP) of 43 years, 225 years, 475 years, 975 years, and a risk-targeted maximum credible earthquake (MCE_R). Refer to Wichman (2023) for a detailed discussion on ground motion selection and scaling method for the 10-story building program. Similarly, for the shake table tests of the 6-story building, ground motions were selected for a site class C and a different location from that of the 10-story building. The selected motions in this test program also represented various seismic sources. For the 6-story building, the motions were also amplitude-scaled to various risk-targeted levels of intensity consistent with those selected in the 10-story building test (Barbosa et al., 2024). For the BRBE configured 6-story building, four ground motion records corresponding to crustal and subduction earthquakes were selected. The selected motions were then scaled to the MCE_R design spectrum. Subsequently, the selected motions were down-scaled to represent service level earthquake (SLE) and design earthquake (DE) targets (Simpson et al., 2024). The 475 year RP motions of the 10-

story program are equivalent to DE motions. This paper focuses on the response of the building and stair system during DE motions, as this offers a common design scenario across all three programs, see Figure 3. In this figure dashed vertical lines show periods corresponding to the first three translational modes in each orthogonal direction of the buildings at their initial (pre-test) state. To determine the modal properties of the building, the response of the building was represented using the Frequency Response Function (FRF), which acts as a transfer function between the input signal on the shake table platen and the output signal on the test specimen.

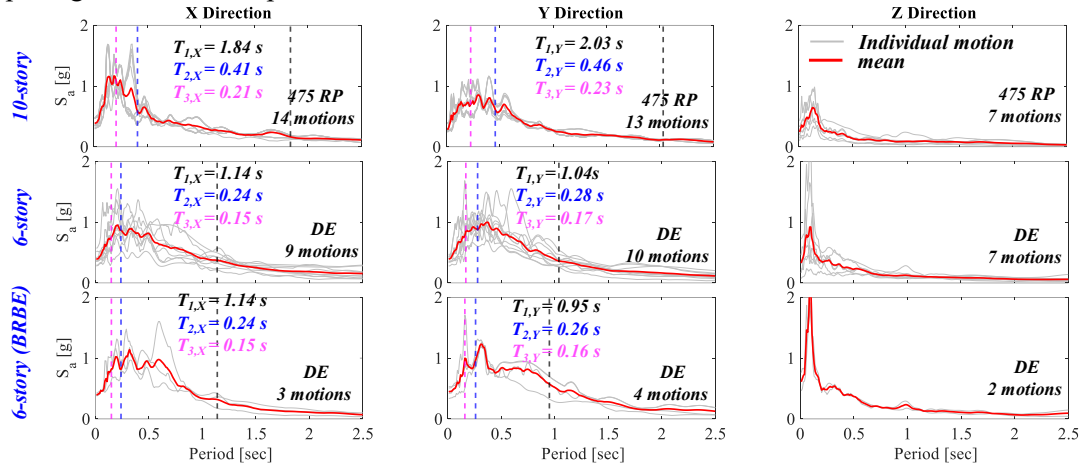


Figure 3. Elastic spectral acceleration (S_a) of select input motion (damping, $\zeta=5\%$)

Building Response

The peak floor acceleration (PFA) and peak interstory drift ratio (PIDR) profiles of each building specimen during the suite of DE motions are shown in Figures 4 and 5, respectively. The acceleration data are filtered using a 4th-order Butterworth bandpass filter with cutoff frequencies of 0.1 and 50 Hz. The floor displacements, that are used to determine PIDR, are obtained by double integrating the acceleration data using the 4th-order Runge-Kutta method. Table 1 summarizes the mean (μ) and standard deviation (σ) of |PIDR| and |PFA| based on Figures 4 and 5. On average, under DE motions, the mean of |PFAs| are between 0.25g to 0.87g with the maximum floor acceleration generally occurring at the roof. However, due to the participation of higher modes, PFAs at levels 4 and 7 of the 10-story building are larger than those at other levels except the roof. This effect is also observed at level 3 of the 6-story buildings. Observing the PIDR profiles of the 10-story building, the lower stories have a smaller PIDR, while stories at mid-height (i.e., stories 6 and 8) have a larger PIDR. However, the PIDR profile of the 6-story building is more uniform with the height of the building.

Table 1. Mean (μ) and standard deviation (σ) of PIDR and PFA

Building	Mean PIDR [%]		Average standard deviation of PIDR [%]		Mean PFA [g]		Average standard deviation of PFA [g]	
	X Dir.	Y Dir.	X Dir.	Y Dir.	X Dir.	Y Dir.	X Dir.	Y Dir.
10-story	0.38-0.99	0.38-0.94	0.15	0.11	0.33-0.75	0.25-0.71	0.11	0.09
6-story	0.69-1.23	0.91-1.20	0.28	0.50	0.34-0.67	0.34-0.65	0.09	0.10
6-story (BRBE)	0.50-0.84	0.70-1.15	0.28	0.24	0.33-0.63	0.29-0.87	0.09	0.06

Stair Response

Connection Displacement Response. Stair connections were detailed to accommodate a drift of up to 2.5% at the MCE_R target scale, thus it is anticipated that during a DE scaled event lower displacement demands will occur. Note that each connection was placed such that they are subjected to 50% of the interstory drift. Therefore, Figure 6 shows a scatter plot of peak connection displacement versus 50% of

the peak interstory drift. It is important to note that these responses are uncorrelated. In general, it is observed that irrespective of the building configuration, the stair connections accommodated the interstory drifts effectively. The standard deviation in PIDR of the 6-story building is larger than that of the 10-story building. Therefore, since stairs are drift-sensitive systems, a larger deviation in the stair connection response in the 6-story building is observed as well. Free connection (FC) shows larger displacement responses compared to the drift-compatible (DC) connections. Differential movements between stair components due to having drift-release connections result in minor damage to the stair components, such as the handrails, that were continuous through these interfaces. Sorosh et al. (2024) discuss the observed damage to the stair system during the shake table test of the 10-story stair system. During the test of the 6-story and 6-story (BRBE) buildings, no new damage instances were reported.

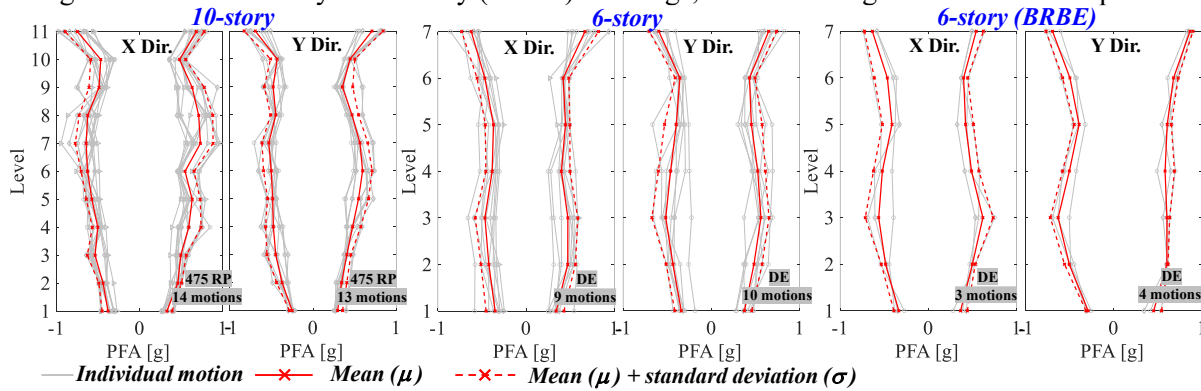


Figure 4. Peak floor acceleration (PFA)

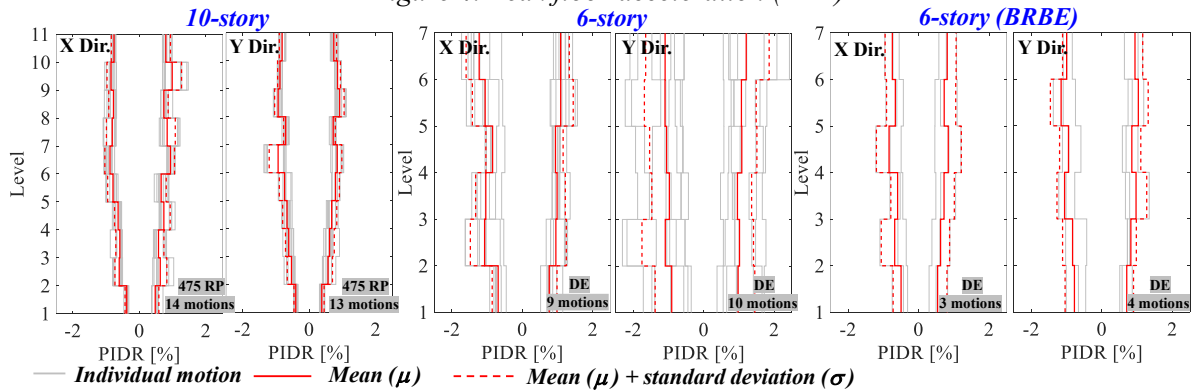


Figure 5. Peak interstory drift ratio (PIDR)

Component Acceleration Response. Peak component accelerations (PCA) normalized by peak floor acceleration (PFA), namely the component acceleration amplification factor, are used to predict the seismic design force of non-structural components and systems. In ASCE 7-22 (2022), the acceleration amplification of nonstructural components and systems is considered in a single coefficient defined as the resonance ductility factor (C_{AR}). In ASCE 7-22 (2022), a C_{AR} of 2.2 is proposed for egress stair and ramp fasteners and attachments, while a value of 1.0 for stairs not part of the building seismic force resisting system is required. Figure 7 shows the PCA/PFA of stair systems at levels 2, 4 and 6, compared to these codified values. During the shake table test of the 10-story building, the acceleration amplifications at the midlandings, more often in the weak axis of stair system (Y Direction), exceeded the code proposed value ($C_{AR} = 2.2$). The accelerations recorded at stair flights, mostly, were within the range of code-based predicted values. However, during the test of 6-story and 6-story (BRBE) buildings, the acceleration amplification in the majority of stair components, particularly in the direction parallel to the stair's stiffer axis (X Direction) exceeded the code proposed value. Stair systems with free connections, more often, showed acceleration amplification exceeding the proposed code value of 2.2 in both X and Y directions. The acceleration amplification in the stair system at level 2 that features the drift-compatible connections

with a single attachment channel had a lower acceleration amplification, compared to other levels. The stair system at level 4 featuring the drift-compatible connections with double attachment channels, throughout the tests of three buildings, observes consistent acceleration amplification response in the Y direction. However, the acceleration amplification in the X direction during the test of 6-story buildings is significantly higher than that of the 10-story building. It is evident that drift-release connections aid in avoiding extensive damage to the stair systems. These connections allow free movement of stair components that eventually increase the acceleration response of components.

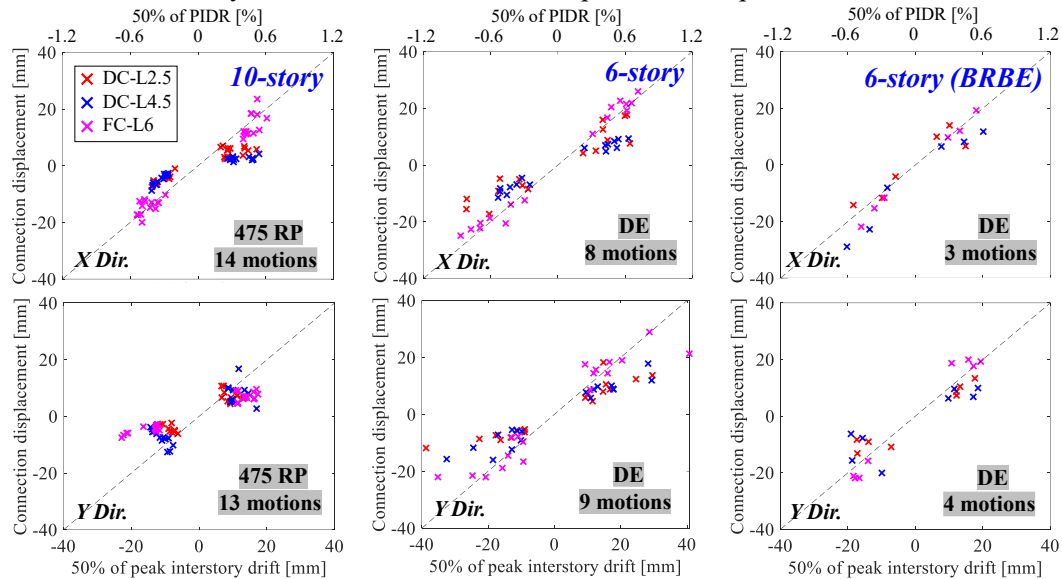


Figure 6. Peak connection displacement vs 50% of peak interstory drift (DC: Drift-compatible, FC: Free connection)

Conclusions

Previous field reconnaissance studies and experimental investigations have revealed that stairs with fixed types of connections are prone to damage under earthquake loading. Therefore, modern design codes such as ASCE 7 (2016, 2022) require egress stairs to be detailed such that they accommodate the interstory drift without loss of gravity support. Recently, stair systems with various drift-release connections were integrated as part of three full-scale mass timber buildings, offering a unique opportunity to characterize their response in a variety of (height and detailed) buildings. In total, over 150 earthquake motions were utilized throughout the three shaking table test programs. This paper provides a brief overview of these test programs, the global response of the buildings, and the seismic design and response of the stairs under the design earthquakes. It was found that drift-release connections consistently avoided extensive damage to the stair systems. However, the movement of stair flights relative to the landings may result in minor damage to continuous members such as handrails. In addition, the free movement of stair flight increases the acceleration response of the stair system. In the shake table test program of the 10-story mass timber building, only the mid-landing of the stair system observed acceleration that exceeded the ASCE 7 (2022) design amplification value. However, testing of both 6-story building configurations presented herein, the acceleration amplification in the majority of stair components in all levels measured exceeded the ASCE 7 (2022) proposed value. Although minimal damage was observed and complete return to functionality was preserved courtesy of the various drift-release connections explored; the observed acceleration amplification in excess of codified values requires further study to ensure that essential connections within the gravity support of a stair are not compromised.

Future Investigations: CFS10

Findings from the stair system investigations conducted during the tall and mid-rise mass timber building programs document very good performance, including continued functionality post-earthquake, of a

variety of drift-release connections within stair systems. To expand understanding of these details and continue to improve their robustness, particularly for a wider range of building lateral force-resisting systems, the authors have implemented a similar modular stair system with drift release details, and enrichments to select features (notably flexible handrail details, locking mechanisms, and improvements to stair shaft details) into an upcoming 10-story cold-formed steel building shake table program, also at the UC San Diego outdoor shake table. At the time of preparation of the current article, construction of this specimen, coined CFS10, was underway. Interested readers are encouraged to note progress and findings to be presented on the project website as they become available: cfs10.ucsd.edu.

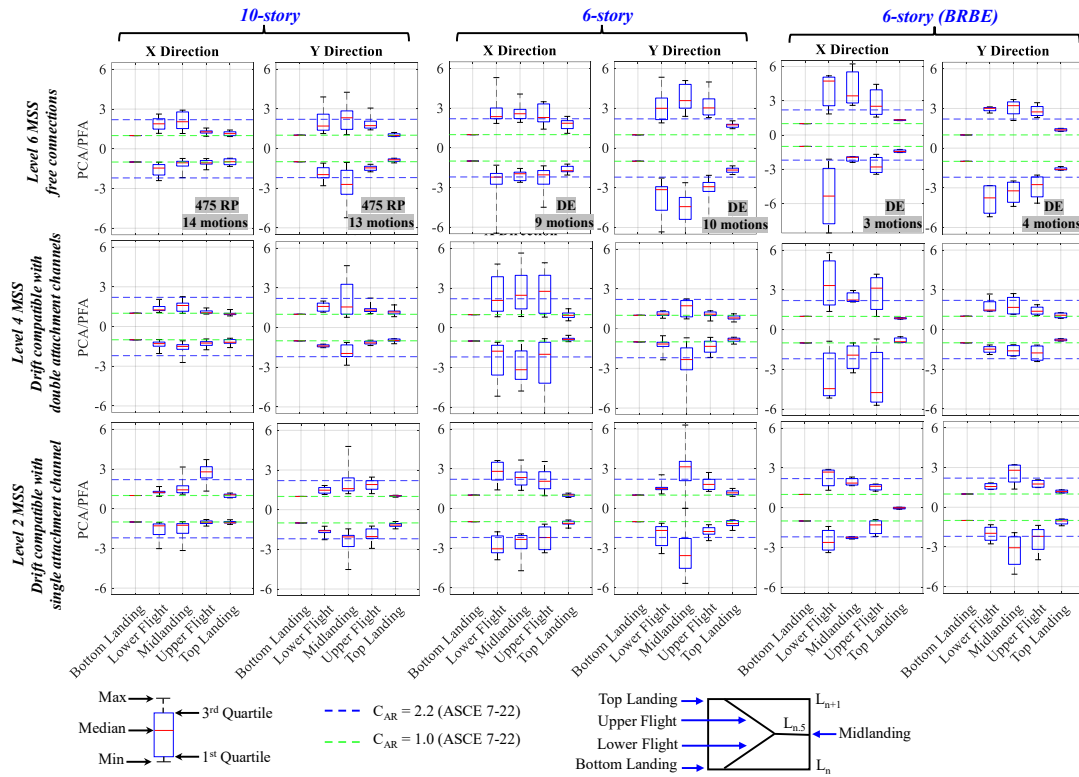


Figure 7. Peak component acceleration (PCA) normalized by peak floor acceleration (PFA)

Acknowledgments

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SEISMIC PERFORMANCE TARGETS FOR CLADDING AND CURTAIN WALL SYSTEMS IN SAN FRANCISCO HIGH-RISES: LESSONS LEARNED SINCE LOMA PRIETA (2024 UPDATE)

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Abstract

In order to obtain necessary regulatory approval to build high-rise structures in San Francisco after the Loma Prieta earthquake (1989), the design team for most owners had to demonstrate to the satisfaction of a peer review panel (“SDRT”) that the constructed high-rise would sustain only minor damage, suspend operations for a very short time and perform virtually elastically during (and after) a foreseeable service level earthquake. This performance prediction specifically applies to cladding and curtain wall systems. For each such post-1989 high-rise, this performance prediction may become the starting point for future legal claims akin to the “performance gap” theories of liability developed in the *Millennium Tower* litigation. The sinking and tilting that triggered the *Millennium Tower* litigation abated by the second half of 2023, by which time several underlying lawsuits had been settled and settlement amounts in excess of \$100 million had been funded. Several lessons derived from this litigation cycle can assist design professionals to better manage seismic risk in the legal arena.

Most of the designs for post-1989 San Francisco high-rises utilize non-structural curtain walls or a functionally equivalent cladding system. It is probable that some of the post-Loma Prieta high-rises will sustain significant damage to those non-structural systems during foreseeable earthquakes. The resulting down-time and costs to repair will almost certainly lead to legal claims against some of the owners and some of the design professionals, based on variations of the “performance gap” theories employed in the *Millennium Tower* litigation. Long before the next significant earthquake, owners and design professionals should develop the testimony that will demonstrate that they acted reasonably in assessing seismic vulnerabilities and developing mechanisms that provide satisfactory seismic capacity for the primary structural system, the curtain wall systems and other non-structural elements of their high-rise facility.

Introduction

Some design professionals solving problems with high-rises in urban centers (such as San Francisco) can improve their methodologies by considering the following question. Before construction commences, do they know what their sworn testimony would be in the event there is a gap between (i) field performance of the as-built high-rise during a foreseeable earthquake and (ii) pre-construction performance predictions by members of the design team? This paper will explain why better practice requires the answer to be in the affirmative. Rigorous consideration of that testimony before construction will often improve the management of seismic risk in the legal arena.

1. Performance Gap Litigation: The Obstinate Legal Ostinato

In the American legal system, the breach of a duty to protect the interests of another party increases the legal exposure of the breaching party. For instance, California Civil Code section 1714(a) requires an owner

of real property to use ordinary care in the management of that property to prevent injuries to third parties; failure to honor that duty can result in civil liability. *See, e.g., Myrick v. Mastagni* (2010) 185 Cal.App.4th 1082, 1087 (liability for two fatalities caused by failed roofing system during an earthquake). This pattern plays out in those lawsuits predicated upon allegations that the breaching party exposed the victim to unnecessary *seismic risk*. For instance, if a developer of a high-rise chooses to install a curtain wall system that will most likely perform below performance targets applicable to the project during a foreseeable earthquake, the legal risk profile of the developer is higher than if its design team had been authorized to take steps (usually leading to additional expense) to assure that the system would meet (or exceed) its curtain wall performance targets.

In the latter scenario, before the commencement of construction, members of the design team can influence the developer's management of its seismic risk by providing technical information that usually cannot be developed by an untrained and unlicensed layperson. Best practice would include advising the developer as to which design options would tend to reduce the developer's risk profile by protecting third parties from interference with their legally cognizable interests. In our hypothetical scenario, the design team may explain to the owner that a more conservative (and more expensive) curtain wall system will remain intact during a foreseeable MCE seismic event, while a cheaper and less-complicated system most likely will detach and place nearby pedestrians at risk of personal injury or death during a foreseeable earthquake. This comparison of scenarios enables the developer to manage its risk profile by, for instance, consciously choosing to spend more resources in the short run to meet elevated performance targets.

After construction commences on a high-rise project, legal claims may be prosecuted against the developer or members of its design team (or both) when (i) the field performance of the facility falls short of preexisting performance targets related to its architectural and structural integrity and (ii) that failure harms the legally cognizable interests of third parties. Here, the pattern will be referred to as "performance gap" litigation. During the last 125 years, as seismic performance targets have become more protective of third parties, the corresponding risk profiles for high-rise developers and members of their design teams have increased. This necessarily has increased the incentives for design professionals to discuss with their developer clients how the frequency of "performance gap" litigation can be managed and minimized.

The types of injuries and dislocations that can be caused by the unreasonable earthquake performance of high-rise curtain wall systems (often referred to as "death, destruction and downtime") include the following:

- Personal injuries to building occupants, visitors and those near the compromised curtain wall system (e.g., pedestrians and vehicle passengers). In extreme cases, the compromised elements of the curtain wall system may cause fatalities. *Compare Myrick v. Mastagni, supra* (roof system).
- Reduced functionality of the compromised curtain wall system itself.
- Impairment of the use of high-rise units previously occupied by tenants and owners.
- Impairment of the use of common areas and other spaces owned and operated by homeowner associations of residential high-rises, and by owners of non-residential high-rises.
- Impairment of use of neighboring public spaces (e.g., roadways and transit facilities) and private facilities arising from orders of governmental authorities.
- Economic losses (including reduced market values) sustained by owners of the above-referenced facilities.

How can design professionals reduce the frequency of "performance-gap" litigation arising from these injuries and dislocations?

2. Lessons Learned from the Millennium Tower Litigation

By the summer of 2023, more than \$100 million had been spent to arrest unexpected settlement and tilting of the 58-story Millennium Tower, located in downtown San Francisco. Many lawsuits were based on the assertion that the Millennium Tower had settled (“had sunk”) significantly more than originally predicted by members of the project design team before construction. *See, e.g., Dennis and Alexandra Lenehan v. Millennium Partners I, Inc. et al.* (San Francisco Sup. Ct.; “*Lenehan*”) Case No. CGC-17-557038 (owners of a residential unit alleged that they would not have purchased their unit if the seller/developer had disclosed “that the building had sunk at a rate greater than designers originally forecast”) (Compl. par. 1; *see also* par. 5). Many other owners made similar claims (including former 49er quarterback Joe Montana, Case No. CGC-17-558649); those cases were consolidated with the “lead case” *Laura S. Lehman, et al. v. Transbay Joint Powers Authority, et al.* (San Francisco Sup. Ct.) Case No. CGC-16-553758. Also consolidated was the lawsuit brought by the homeowner’s association, *Millennium Tower Association v. Mission Street Development LLC, et al.* (San Francisco Sup. Ct.; “*Association*” case) Case No. CGC-17-557830, which, in its fifth amended complaint (filed 4/16/2018), alleged that, among other things: the Millennium Tower “was originally predicted to sink no more than 4 to 6 inches over its lifetime” (par. 181); but its actual settlement was 16 inches as of April 2018 (par. 2); “The Tower was constructed on an inadequate foundation system and has experienced vertical displacement of over 16 inches” (par. 39); “The foundations of the Tower and Podium are experiencing settlement at a rate and depth well beyond the design parameters” (par. 91(d)).

The steps taken to resolve these legal claims and reduce the rate of settlement of the Millennium Tower illustrate types of challenges for design professionals that will arise when certain curtain wall systems perform unreasonably during the next significant earthquake in San Francisco. They can be divided into 5 phases.

Phase One: Identification of the pre-construction performance targets. According to pleadings in the *Association* case, Treadwell & Rollo, Inc. (“T & R”) “was the geotechnical engineer of record for the Millennium Tower.” (Fifth Amended Complaint, par. 40). The prevailing view among those involved in analyzing the settlement problem at the Millennium Tower is that before construction, T & R made predictions of future settlement of the tower, three of which are discussed here.

Predicted settlement at the time of completion of project construction.

According to the EDRT (see footnote 1, *infra*), T & R’s “original design anticipated 1 inch of settlement under the Tower by the time of construction completion” (EDRT (7/28/2017) p. 2). According to the *Lenehan* complaint, a February 14, 2005 T & R memo predicted that “Settlement” would be on the order of “1.8 to 2.6 inches” by August 1, 2008, the date when the “Project [is estimated to be] completed.” (Complaint, pars. 36-37).

Predicted settlement at 5 and 20 years after completion of project construction.

According to the EDRT, “The original design anticipated . . . additional long-term settlement due to compression of the underlying clay layers of 5 inches.” (EDRT (7/28/2017) p. 2). According to the *Lenehan* complaint, a February 14, 2005 T & R memo predicted that “Settlement” would be on the order of: “3 to 4 inches” by “August 1, 2013,” which was estimated to be “5 years after completion”; and “4.5 to 5.5 inches” by “August 1, 2028,” which was estimated to be “20 years after completion.” (Complaint, pars. 36-37).

Phase Two: Comparison of actual field performance to pre-construction performance targets: the *Performance Gap*.

The EDRT and other interested entities investigated actual settlement experienced by the Millennium Tower in the field. Three sets of such data are discussed here.

Actual settlement at the time of completion of project construction. According to the EDRT, the “project’s completion” was August 2009, and by that time “total settlement [had] increased to about 9 inches.” (EDRT (7/28/2017) p. 5). Accordingly, the *gap* between T & R’s predicted settlement and the actual settlement measured in the field at the time of project completion was ***more than 6 inches***.

Actual settlement 5 years after completion of project construction. According to the City’s monitoring engineers (Slate), by August 2014 (5 years after the “project’s completion”), total settlement had increased to more than 12.5 inches. (City, Monitoring Reports). Accordingly, the *gap* between T & R’s predicted settlement and the actual settlement measured in the field five years after project completion was ***more than 8.5 inches*** relative to the 5 year prediction and ***more than 7 inches*** relative to the 20 year prediction.

Phase Three: Assessment of impairment of structural integrity and seismic capacity.

The EDRT ultimately concluded that the elevated settlement did not make the Millennium Tower more susceptible to collapse in a foreseeable MCE event. “SGH’s engineering team investigated the building’s performance by analyzing the structural system and foundations using nonlinear response-history analysis (NLRHA) under severe earthquake ground motions. **This approach to establishing acceptability of seismic safety is consistent with current requirements and standard practice for tall buildings in San Francisco that are designed using non-prescriptive seismic design procedures, as described in the San Francisco Department of Building Inspection’s (SFDBI) Administrative Bulletin AB-083 (2011) and associated guidelines and standards (PEER, 2010, ASCE 7-10).**” (EDRT (7/28/2017) p. 7; **bolding added**).¹

“The SGH study has further demonstrated that in its current condition, the structure generally meets the performance-based design criteria for the seismic design of new tall buildings using non-prescriptive seismic-design procedures. The nonlinear structural analysis procedures employed in this study have been calibrated to consider the effects of degradation that may occur to important structural components under strong ground shaking. **Results of these analyses indicate that the structure meets the earthquake**

¹ Administrative Bulletin 083 (“AB-083”) was promulgated by the City of San Francisco to require design team members to demonstrate satisfaction of certain performance characteristics for “New Tall Buildings” that were designed using “Non-Prescriptive Seismic-Design Procedures.” “Tall Building” is defined as a structure with an “*hn* greater than 160 feet above average adjacent ground surface.” (City, AB-083, section 1). Service-level ground motion is defined as “having a 43-year mean return period (50% probability of exceedance in 30 years).” (Id. at section 4.2). Adherence to the requirements of AB-083 is obtained through a Peer Review process denominated “Structural, Geotechnical, and Seismic Hazard Engineering Design Review” spelled out in DBI’s Administrative Bulletin 082 (“AB-082”). (City, AB-082). The extent to which members of the “SDRT” team require the design team to justify the substance of their compliance with applicable performance targets has increased since AB-082’s original adoption in 2008. These refinements were, in part, the results of lessons learned from the complex civil litigation in San Francisco Superior Court involving unexpected settlement at the *Millennium Tower*. Similarly, an Engineering Design Review Team (“EDRT”) was formed for the PPU retrofit project. The senior member of the EDRT for the Millennium Tower is Professor G.G. Deierlein of Stanford University.

building drift criteria of AB-083 (2011) under the Maximum Considered Earthquake ground motions, as specified in the 2016 San Francisco Building Code.” (EDRT (7/28/2017) p. 12; bolding added).

Thus, in 2017, both SGH and the EDRT concluded “the settlements experienced by the 301 Mission tower have not compromised the building’s ability to resist strong earthquakes and have not had a significant effect on the building’s safety.” (EDRT (7/28/2017) p. 7; *see also* p. 12 (“our professional opinion is that the foundation settlement experienced by the Tower has not appreciatively affected the safety of the building at this time”)).

Phase Four: Development of a plan to mitigate performance gap problems.

Simpson Gumpertz & Heger (“SGH”; Ronald Hamburger was the Engineer of Record (“EOR”)) developed a plan (including a detailed design) to mitigate the problems arising from elevated settlement of the Millennium Tower, including unchecked sinking. “The **goal** of [the SGH] design is to relieve a portion of the stress on the Old Bay Clay soils along the building’s north and west sides, and in so doing, **arrest further settlement at the building’s northwest corner**, while recovering a portion of the building’s tilt through continued secondary compression of the Old Bay Clays located to the south and east of this corner. The upgrade will also **enhance the seismic resistance** of the building’s foundation.” (SGH (12/28/2021) p. 2; bolding added).

The EDRT approved the SGH design, both in its original and amended forms. “The approved permitted retrofit plan entails the installation of 52 new piles, extending down to the Franciscan Complex bedrock, along the north and west perimeters of the building, which are to be attached to the building through an extension to the existing mat foundation. As stated in the EOR’s basis of design, the structural upgrade is designed to meet the requirements of Section 403.9, Voluntary seismic improvements, of the 2016 San Francisco Existing Building Code (SFEBC), with **the intent to reduce future building settlement and improve the seismic performance of the foundation**. As summarized in our letter dated August 19, 2019, the EDRT previously reviewed and offered our professional opinion that the originally permitted retrofit is expected to meet the EOR’s stated objectives and section 403.9 of the San Francisco Existing Building Code.” (EDRT (6/15/2022) pp. 1-2; bolding added). (See also EDRT (9/13/2023) p.1).

This conclusion was not changed when the SGH design was reduced from 52 piles to 18 piles. “The EOR and team have further stated that the 18-pile foundation retrofit will accomplish the original project design objectives.” (EDRT (6/15/2022) p. 2). “*Structural Safety*: Structural analyses of the building and foundation systems indicate that under the current and estimated future building settlement and tilt with the revised 18-pile PPU (through 2060), the systems meet the structural safety requirements of the San Francisco Building Code to resist gravity loads and extreme earthquake ground motions. **The extreme earthquake ground motions are consistent with the Maximum Considered Earthquake (MCER) as specified in the 2016 San Francisco Building Code and Administrative Bulletin 83 (March 25, 2008; updated 1/1/2020) for new tall buildings**.” (EDRT (6/15/2022) p. 4; bolding added). Accordingly, the EDRT stated “in our professional opinion the revised 18-Pile PPU meets the requirements of section 403.9 of the 2016 San Francisco Existing Building Code.” (Ibid.).

Phase Five: Articulation of post-remediation expectations and performance targets.

Construction implementing the SGH mitigation plan and design was completed around August of 2023. *See, e.g.,* City Monitoring Reports. During late 2021, “The Department of Building Inspection has requested that prior to resumption of production piling, the project present a **projection of expected additional building settlement** and tilt through project completion.” (SGH (12/28/2021) p. 12; bolding added). In response, SGH “project[ed] building performance” to include “additional building settlement”

as follows: “**total settlement, at the end of December 2022 of approximately 17 in.**, with total tilt to the west of 27 in. and total tilt to the north of 11 in.” (SGH (12/28/2021) p. 13; (bolding added)). This is very close to the “Planar Average” actual field settlement for December 2022 and for completion of remediation work in the field (August 2023), as reported by Slate Geotechnical Consultants in its July 29, 2024 report. (City, Monitoring Reports).

Shortly after substantial completion of the PPU project, the EDRT (9/13/2023) made a series of findings, including the following: “the design revisions and quality assurance measures implemented by the EOR conform to accepted engineering and construction practice in accordance with the San Francisco Building code” (p. 2); and “The monitoring reports that the EDRT has reviewed to date indicate that the foundation retrofit has been successful in stabilizing further settlement in the north-west corner of the building” (p.3).

In his Zoom interview with the author on October 6, 2024, Ron Hamburger stated the completed Perimeter Pile Upgrade meets the project goals of “stopping the sinking and tilting” of the Millennium Tower. **He expects overall settlement of the Millennium Tower to increase by approximately one inch by calendar year 2060** (i.e., the Planar Average would thus be approximately 18 inches of settlement in 2060).

In the same interview, Ron Hamburger stated that he expects the Millennium Tower to avoid collapse during a foreseeable earthquake in the next 40 years. During a design level earthquake (say PGA on the order of 0.40), Hamburger expects the outrigger elements of the frame system to sustain repairable damage and the curtain wall system to remain intact. During a targeted MCE event (say PGA in excess of 0.50, up to 0.70), Hamburger believes the outrigger elements would be significantly damaged, making repair possible, but problematic (e.g., the owner would need to consider demolition as an alternative to repair). In such an MCE event, Ron Hamburger believes the Millennium curtain wall system will perform well, and not detach from the superstructure of the tower.

3. Managing Seismic Risk Related to Curtain Wall and Cladding Systems within the Framework of San Francisco’s AB-083

In San Francisco, since 2008, through the administration of Administrative Bulletin 083 (“AB-083”), the Department of Building Inspection (“DBI”) has conditioned permission to construct most new high-rises on a pre-construction demonstration by the design team that both (i) the “primary structural system” and (ii) “critical non-structural elements,” including “exterior curtain wall and cladding systems,” will remain intact during and after a foreseeable service level earthquake. (City, AB-083, sections 4.1 and 4.2). In AB-083, DBI mandated virtual elasticity as the performance target in service-level earthquakes for “New Tall Buildings” that were designed using “Non-Prescriptive Seismic-Design Procedures.”

With regard to the “primary structural system,” section 4.2 of AB-083 requires the project design team “*to demonstrate acceptable, essentially elastic seismic performance at the service-level ground motion.*” (Italics and emphasis added.) The design team must demonstrate no more than “minor yielding,” requiring no more than “minor repair.” (City, AB-083, Commentary at p. 83-5). Specifically, “*essentially elastic seismic performance*” includes a prediction of no worse than “minor yielding of ductile elements of the primary structural system, provided such results do not suggest appreciable permanent deformation in the elements, strength degradation, or significant damage to the elements requiring more than minor repair.” *Ibid.*

With regard to “exterior curtain wall and cladding systems,” DBI expects “that the building cladding will remain undamaged and that egress from the building will not be impeded when the building is subjected to the service-level ground motion.” (City, AB-083, section 4.2 (Commentary)).

An essential lesson of the Millennium Tower litigation is that there will be a significant risk that either the owner/developer or members of the design team (or all of them) will be named in “performance gap” litigation if there is a gap between (i) the performance targets demonstrated by the design team to the SDRT (pre-construction) and (ii) the actual performance in the field. One way to reduce this risk profile before construction commences is for each member of the design team to consider whether their sworn testimony would demonstrate that he or she took reasonable steps to assure that the performance targets would be met for both structural and non-structural systems of the project.

For instance, if he or she worked on the “primary structural system,” how would his or her sworn testimony show that design choices were made to assure that it would demonstrate “essentially elastic performance” in a service-level earthquake, sustaining insignificant damage, only requiring “minor repair.” Similarly, if he or she worked on “exterior curtain wall and cladding systems,” how would his or her sworn testimony show that design choices were made to assure that in a service-level earthquake the curtain wall system “will remain undamaged and that egress from the building will not be impeded.”

Conclusion

A valuable lesson derived from the Millennium Tower litigation is that peer review panels (SDRTs) evaluating designs for “New Tall Buildings” in San Francisco that employ “Non-Prescriptive Seismic-Design Procedures” will likely require the sponsoring design team to demonstrate adherence to the performance targets spelled out in Administrative Bulletin 083. The same is true when the project involves a seismic upgrade to an existing high-rise. For those members of the design team working on the “primary structural system” or on non-structural systems (such as curtain walls) in the high-rise design, before construction, they should develop their own testimony that will demonstrate that they acted reasonably in assessing seismic vulnerabilities and developing corresponding mechanisms that, in the field, will provide satisfactory seismic capacity to meet the demands of foreseeable service, design and MCE earthquakes.

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NEW ZEALAND CODE OF PRACTICE FOR THE SEISMIC PERFORMANCE OF NON-STRUCTURAL ELEMENTS

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Abstract

Recent studies on non-structural elements (NSEs) in New Zealand highlight a significant gap in knowledge regarding the seismic behaviour of NSEs and particularly, the ability of NSEs to function post-earthquake and whether the building will continue to operate as a result. Whilst the seismic restraint of Non-Structural Elements (NSE) predominantly involves design to achieve life-safety performance, the seismic performance of NSEs is integrated in the overall building design, necessitating deliberate design considerations and actions from multiple disciplines, including the structural engineer, architect, fire protection engineer, building services engineers and others. The New Zealand Code of Practice for the Seismic Performance of Non-Structural Elements (NSE CoP) is a document that provides guidance for clients to determine the seismic performance they require of their building and guidance for the design and construction team on how to achieve that performance. The NSE CoP is a foundation guidance document for New Zealand, it provides a base level to determine the seismic performance for NSEs and how the seismic performance of NSEs impacts the overall building seismic performance. The NSE CoP highlights where research is needed and supports investment and innovation by providing pathways to incentivize NSE suppliers to improve the seismic performance of their products. Use of this Code of Practice is expected to establish consistency for the seismic design of NSEs across the industry and improve the overall seismic performance and resilience of our building stock.

Introduction

Many buildings suffered minor structural damage as a result of recent New Zealand earthquakes (2010-2011 Canterbury earthquakes, the 2013 Seddon earthquake, and the 2016 Kaikoura earthquake), however many of those buildings were unable to be used and re-occupied due to damage and failure of non-structural elements (Dhakal, 2010; Baird, 2017). In these instances, the damage to non-structural elements (NSEs) had implications to building functionality and caused major disruptions to businesses and our communities (BIP White Paper, 2020). The performance of our building stock during these earthquakes underlined the need for improved seismic performance and resilience of our buildings.

Purpose and Motivation for NSE CoP

Over the past decade, the New Zealand building industry has increasingly recognised the need for a guidance document to address issues related to NSEs, such as inconsistencies in design practices and allocation of roles and responsibilities (BIP White Paper, 2020; Rashid, 2022). Problems also exist around the transfer of risk to contracting teams, who are traditionally tasked with the design and coordination of NSEs during construction phase. This timing led to challenges in meeting the extensive and varied requirements related to both the functionality and seismic performance of NSEs. For example, ensuring services that penetrate fire walls are both fire-rated and capable of accommodating seismic movements can prove difficult. These challenges frequently result in complications during the review and sign-off stages of construction. The recently published Code of Practice for the Seismic Performance of Non-Structural Elements (NSE CoP) offers guidance for industry professionals, addressing the seismic performance and functionality of NSEs.

The primary purpose of the NSE CoP is therefore to “*Provide guidance on the seismic design, coordination, and construction of non-structural elements to improve practices, and establish consistency in approach across the industry.*”

In addressing the seismic performance of NSEs, the NSE CoP could have focused on numerous potential issues. After gathering industry feedback, four key issues have been identified as primary areas for focus, which align with the document's purpose above. These are explored further in the context of the NSE CoP's first version to industry, with future stages anticipated to broaden its scope. The four main areas of focus are as follows:

1. Roles, Responsibilities and Scope Gap. A crucial challenge within the industry has been the lack of clarity in roles, responsibilities, and effective management of scope between stakeholders. The NSE CoP seeks to address this by providing a framework that outlines recommendations on ‘what’, ‘who’, ‘how’ and ‘when’ multidisciplinary collaboration should be undertaken. It emphasises the importance of early-stage engagement and coordination between disciplines—including structural engineers, architects, building services, fire protection, and others—which is crucial for improving seismic performance outcomes. This proactive engagement is essential for achieving and maintaining consistent seismic performance standards across a variety of projects.

2. Early Coordination and Design Integration. One of the key objectives of the NSE CoP is to optimise seismic performance by promoting early discussions and decisions during the planning phases of construction projects. The NSE CoP has intentionally aligned with the New Zealand Construction Industry Council Guidelines (NZCIC, 2023), which define and communicate responsibilities and outputs to all parties involved in a project through all project stages. The NZCIC (2023) guidelines was developed with similar industry-wide concerns related to inconsistency in the New Zealand construction industry. The NSE CoP provides a systematised approach to design and coordination but does so in a scalable framework that is versatile enough to be applied to a variety of building types and project sizes. This approach not only improves the integrity and quality of construction documentation but also enhances the consistency of seismic design practices, ultimately leading to better overall outcomes in terms of building resilience and safety.

3. Clarifying Seismic Performance Objectives. A central focus of the NSE CoP is the establishment of clear performance objectives, accompanied by detailed guidance on how to meet these. A notable example is the seismic performance of concrete anchors for seismic restraint in services, partitions, and ceilings. Despite significant attention and research into aspects like seismic rating, testing protocols, and potential prying actions, failures in the application of these anchors are rare. This suggests a potential mismatch between the areas where resources are concentrated and the actual issues that need addressing. Another illustrative case is the seismic verification of plant and equipment, where New Zealand currently lacks a definitive seismic verification methodology. New Zealand often relies on international testing standards, which might not align completely with New Zealand's specific performance requirements, resulting in a gap in ensuring both compliance and effectiveness.

4. Fostering Innovation and Research. The NSE CoP serves as a catalyst for innovation and research by identifying and addressing existing knowledge gaps within the industry. By clearly highlighting these gaps, the NSE CoP can prioritise future research. The NSE CoP supports innovation by establishing a baseline level of performance that allows for meaningful advancements to be made. With a clear understanding of baseline performance, the industry can focus on developing enhancements in critical areas.

The NSE CoP serves not only as a guideline but as a baseline reference for other forthcoming guidance documents currently being developed. These include the structural, geotechnical and building services design guidance for hospitals, which is summarised in an accompanying workshop paper (Brunsdon et al, 2024) and the low damage seismic design of buildings guidance document, which is also summarised in an accompanying workshop paper (Sullivan et al, 2024).

Coordinated Seismic Design process to achieve good seismic performance of the building as a whole

Historically, the structural system options are considered alongside the architectural aesthetic and layout. Little consideration has typically been given to the implications of the structural system response on building functionality and seismic performance of NSEs. As the structure often dictates the seismic response of the NSEs, there can be potential additional project complexity and costs, that are not considered during the structural system selection. Further to this, the design of building services, passive fire, ceilings, and partitions has typically been undertaken by different designers, often operating in contractual silos not considering the wider seismic performance of the building.

Overall, this design process rarely led to best-for-project outcomes as it focused only on seismic life safety not functionality and building resilience. The ability to achieve the required seismic performance of all NSEs was often compromised due to issues relating to disconnected timing of design work by various disciplines, poor or limited coordination between disciplines during the main building design, inconsistency in design approach and confusion in scope between disciplines.

The NSE CoP takes a different approach. It is based on holistic design of all disciplines from the start of the project. It provides guidance on how to work through the holistic design process from project briefing through to design undertaken during the construction phase. The design process described in the NSE CoP is scalable and can be used on large complex buildings through to smaller simpler buildings.

The design process includes development of a Building Movement Strategy where the design team consider all aspects of building movement. The Building Movement Strategy will consider requirements for movement and gaps versus locations and zones where movement needs to be restricted and restrained, e.g. thermal movements of components required for day-to-day function of the building versus seismic restraint of components, and/or gaps for acoustic/vibration control. It is acknowledged that there are often competing goals, with potential cost and performance implications, and compromises are likely to be needed. Nevertheless, the NSE CoP is clear that what is important is that the design team should agree what aspects are to take precedence and which can be compromised, and the agreement is to be clearly documented including why the decision was made.

The NSE CoP provides design task responsibilities and coordination guidance between disciplines for the various project phases. The NSE CoP identifies and acknowledges the challenges that occur at the interfaces where multiple disciplines work and that these interfaces are typically poorly coordinated. The NSE CoP provides guidance on various design elements and discipline responsibilities for six specific interfaces including inter-storey drift interfaces, vibration isolation interfaces, thermal expansion interfaces, ceiling and partition interfaces, equipment/system section interfaces and passive fire interfaces.

Figure 1 provides an example from the NSE CoP of the conceptual design process. It shows the iterative process to consider the implications of the proposed building form and layout, structural system options, locations of heavy plant, fire engineering, building services reticulation strategy and the overall strategy for how building movements will be accommodated.

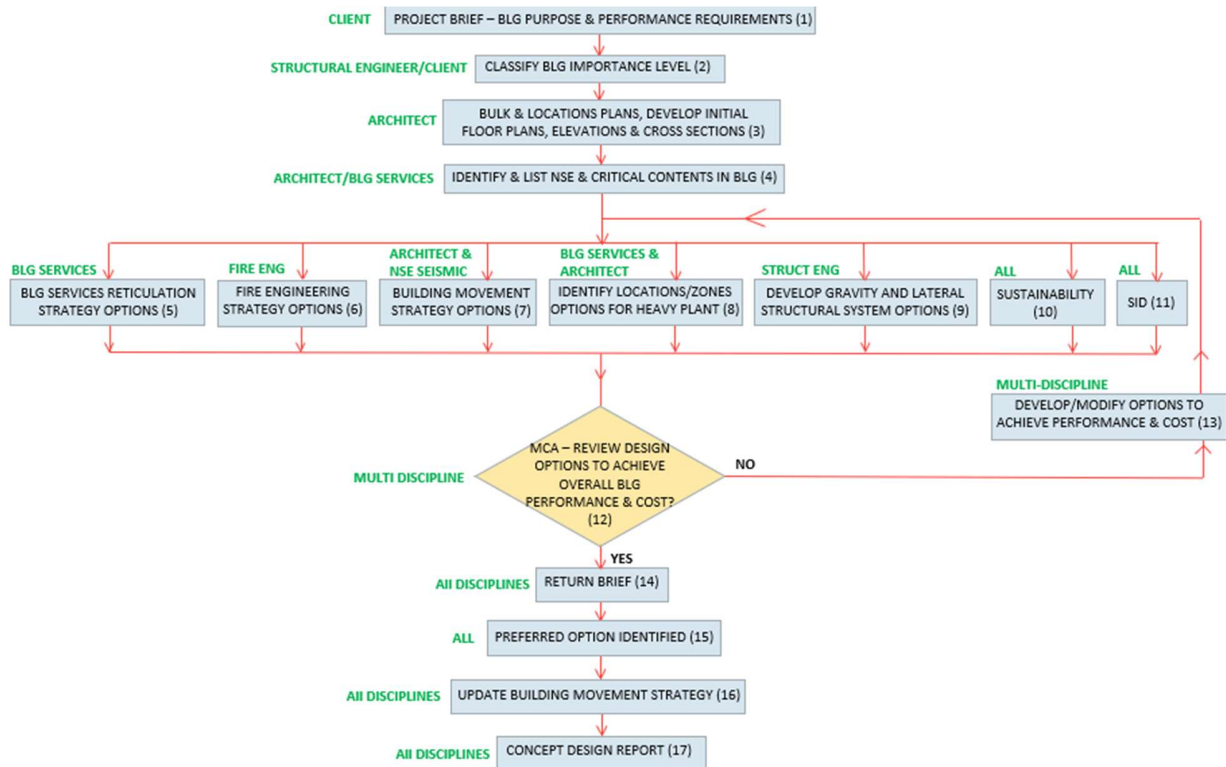


Figure 1. Concept design process flowchart

Seismic verification of plant and equipment

The New Zealand Building Code (via NZS 1170.5 and NZS 4219) requires that all plant and equipment that are a life safety hazard should be verified for structural integrity. In New Zealand a life safety hazard is defined as a component that can fall either 3m or more or is 7.5kg or more. In addition, there are a number of components that are known to be inherently rugged where those pieces of plant and equipment have been consistently shown to remain undamaged and function after a design level earthquake. The NSE CoP lists inherently rugged equipment based on international research and feedback from industry.

Furthermore, the scope for seismic restraint designers frequently does not include a review of the selected equipment. As a result, there is often a critical gap between the design of the restraints and the connection points on the equipment, as well as the overall structural integrity of the equipment itself. This oversight can lead to failures in ensuring that both the equipment and its restraints can withstand seismic forces. Additionally, a clear distinction between structural integrity and operational integrity is often not well understood or properly addressed. Structural integrity only confirms that the plant and equipment will not, in whole or part, create a falling hazard during ULS seismic shaking¹. Operational integrity verifies that the plant and equipment can continue to operate following a designated level of earthquake where operation is required.

¹ ULS = Design Earthquake = 2/3 of the corresponding risk targeted maximum considered earthquake MCE_R

The NSE CoP lists different component types and the minimum seismic verification requirements for structural integrity and operational integrity of plant and equipment associated with each component type, refer Table 1.

Table 1. Minimum Seismic Verification* Requirements for Plant and Equipment

	<i>Component Description</i>	<i>Structural Integrity Seismic Verification¹ Required?</i>	<i>Operational Integrity Seismic Verification¹ Required?</i>
P1, P2, P3	Component that represents a hazard to life outside of the building or within the building and is not inherently rugged.	Yes (ULS ²)	Not required ³
P4	Component necessary for the continuing function of the evacuation and life safety systems within the building and is not inherently rugged.	Yes (ULS ²)	Yes (operation following ULS ² event)
P5	Component of a system required for operational continuity of the building and/or designated space and is not inherently rugged.	- ⁴	Yes (operation following SLS2 ⁵)
P6	Component for which the consequential damage caused by its failure is disproportionately great and is not inherently rugged.	- ⁴	-
	Component is inherently rugged	Not required	Not required
P7	All other components	Not required	Not required

Table notes:

1. Seismic verification includes a) analysis, b) testing, or c) experience data as described in ASCE 7-22 Clause 13.2.1.
2. ULS = Design Earthquake = 2/3 of the corresponding risk targeted maximum considered earthquake MCE_R.
3. Client may choose operational continuity verification
4. Refer to category P1, P2, P3 requirements for structural integrity verification if the component is a life safety hazard.
5. SLS2 = is the limit state where the building and/or designated space maintains operational continuity after the SLS2 earthquake. NZS 1170.5 Amendment 1 requires SLS2 performance for buildings with special post-disaster functions, e.g. hospitals, and defines the SLS2 earthquake to have an annual probability of exceedance of 1/500 with a design life of 50 years. NZS 1170.5 also provides operational continuity requirements for lower importance level buildings e.g. 1/100 and 1/250 AEP.

We understand that the ASCE 7-27 working group is considering providing a limit for floor accelerations below which plant and equipment would be exempt from needing to achieve seismic verification. We believe this may be practical but as different plant and equipment have different vulnerabilities to being shaken it may be that various types of plant and equipment would be better categorized into Acceleration Classes e.g. 0.5g, 1.0g, 1.5g, > 1.5g.

Should a future version NSE CoP recommend a plant and equipment Acceleration Classification System, it would ideally be based on an international rating system for plant and equipment as New Zealand uses

plant and equipment from around the globe including USA, Japan and Europe. The holistic design process included in the NSE CoP includes early consideration of the accelerations expected in various locations in the building. This could be used alongside plant and equipment Acceleration Classes to motivate the design team to design the structure and/or consider the location of critical plant and equipment in the building to reduce accelerations such that the need for seismic verification of plant and equipment can be reduced/eliminated.

Options for improved seismic performance and resilience

There are different means of providing a building with improved seismic performance and resilience. One option, being developed as a deemed-to-comply approach for low-damage seismic design (LDSD) (Sullivan et al. 2024), is to stiffen the building and thereby limit the storey drift demands. By limiting the storey drift demand, damage to structural elements and drift-sensitive non-structural elements can be reduced. On the other hand, demands on acceleration-sensitive non-structural elements may increase, but there is an expectation that the design of such elements would recognize this and hence, adjust seismic restraint bracing requirements accordingly.

A recent study by (Sullivan et al. 2024) found that application of the LDSD deemed-to-comply approach to a set of case-study buildings in the main cities in New Zealand, led to significant reductions in expected annual losses (associated with direct repair work). However, in low seismic regions (such as Auckland) the likely damage and losses were not high even without LDSD and for high seismic regions, such as Wellington, damage to acceleration sensitive non-structural elements could not be completely mitigated as some damage states do not depend on the amount of bracing but rather, the level of acceleration demand. This finding is partly due to a lack of mechanical and electrical equipment that has undergone seismic qualification in New Zealand.

As an alternative to the use of seismic qualification of plant and equipment, the structural design team could aim to limit the acceleration demands on the NSEs. Base-isolation, either of the whole building or of the components themselves, offers one means of limiting acceleration demands on NSEs in high-seismic regions.

A second general strategy for improved seismic performance and resilience is to design and detail structural and NSEs for larger storey drift capacities. The benefits of good detailing on the deformation capacity of structural elements have long been recognized and is reasonably well reflected in current material standards. Over the past few decades, a number of efforts have also been made to develop detailing approaches for common types of NSEs that improve their drift capacity. Furthermore, Arifin et al. (2021) showed that by adopting low-damage detailing just for plasterboard partition walls, the expected annual losses in a 22-storey building could be reduced by around 40% (from 0.10% to 0.06% the building replacement cost).

While the benefits of detailing NSEs to provide larger drift capacity may be appreciated, there is currently no strong incentive for practitioners or suppliers to do so. To this extent, it is expected that the NSE CoP could play an important role in facilitating this via the NSE Drift Classification System described in the NSE CoP. The NSE Drift Classification System provides a simple five class drift naming convention to describe the amount of deformation a drift sensitive NSE can sustain to achieve the required performance requirements. It is expected the Drift Class System would be used by engineers to communicate the required drift performance by way of reference to the NSE CoP and stating the required Drift Class. This is expected to provide incentive to suppliers to provide products that can achieve performance at higher drifts.

The Future

The first version of the NSE CoP is the distillation of good engineering practice and academic research, but it is a long way from addressing all issues in the industry. The performance of some NSEs is relatively well understood whilst others, such as suspended ceilings, still have large unknowns. In the first instance, collating performance data and ongoing testing are essential to fill in the gaps in the performance framework. Further iterations of the NSE CoP are expected to include specific guidance for the residential sector. It is also recognised that this first release of the NSE CoP, by necessity, is mainly focused on design and process. Later phases will have greater emphasis on the role of the contractors and subcontractors.

The largest gap in our knowledge is the comparative performance of components and systems from different suppliers. New Zealand is a small market and so relies upon imported building components to a large degree. To date, there is no consistent way of verifying the seismic performance of NSEs brought in from overseas. This equivalency problem is a focus of the next stage of the NSE CoP. A desktop exercise will be undertaken to compare codes, loading standards and testing protocols to develop seismic verification framework. We will draw on national and international performance data to define testing protocols for drift and acceleration sensitive systems. We will also review overseas definitions of ‘robust’ products and ‘experience’ data to provide a consistent system to allow engineers and contractors to have a common language and confidence in component selection.

Implementation of the NSE CoP is also a major piece of future work. Developing the guideline is only the start. Initially, promotion will be undertaken by a standard approach of sharing at conferences and in industry journals and other communications. For the NSE CoP to have real impact however, there are a couple of different approaches that need to be taken. The first of these is to build a community of practice around holistic design. This will leverage other guidance such as the Low Damage Design Guidelines that are currently being developed (Sullivan, 2024), and which reference the NSE CoP. These guidelines should have impact as they are being developed by the Ministry of Building, Innovation and Employment (MBIE, the building regulator) and Engineering New Zealand, the organisation responsible for administering Engineering Chartership. Forming special interest groups within the Structural Engineers Society New Zealand and the New Zealand Society of Earthquake Engineering will be a first step.

The challenge of creating new industry roles such as the NSE Seismic Designer are likely to be a longer-term challenge. Whilst forward-thinking designers and contractors are likely to create these roles, it is reliant upon the clients and their procurement teams to stipulate the importance and, of course, to fund them. The best levers for this are within Central Government procurement. Current changes in the way the New Zealand Government support the procurement of major projects through the creation of the National Infrastructure Agency may provide a pathway to making these changes.

Research will also play a major role in the future development of the NSE CoP, where the NSE CoP, in effect, sets out a research agenda. Questions such as: interactions between systems, balancing conflicting performance requirements, safe-to-fail components, defining the performance of systems that are both drift and acceleration sensitive, etc., are all in need of urgent answers. Further to this is the need to understand and improve how Building Information Modelling (BIM) can be used to enhance coordination and reduce risk.

Conclusions

The NSE CoP has applicability across our building stock, including design and construction of new buildings as well as applicability to existing buildings. By applying these principles, the industry can accelerate improvements in the seismic performance and resilience of both new and existing buildings.

The NSE CoP sets out to provide practical guidance to all parties involved in the planning, procurement, design, and construction of buildings. By encouraging a holistic view of building design, the NSE CoP is a critical step toward elevating New Zealand's building practices. Its role in fostering a collaborative and integrated approach ensures that all stakeholders, from architects to engineers and contractors, contribute to the seismic resilience of structures. This not only addresses the immediate needs posed by known vulnerabilities but also paves the way for future developments tailored to the specific demands of New Zealand's unique seismic landscape.

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RESILIENCE AND SUSTAINABILITY CONSIDERATIONS IN PERFORMANCE-BASED SEISMIC DESIGN OF STRUCTURES

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Abstract

Traditional seismic design approaches prioritize life safety but pay limited attention to a structure's upfront embodied carbon footprint, post-earthquake functional recovery, and the environmental impacts of post-earthquake intervention (i.e., repair, retrofit, or replacement). As performance-based seismic design of civil infrastructure evolves, there is an increasing need to incorporate and balance resilience and sustainability objectives. This paper presents a holistic multiobjective design framework for integrating resilience and sustainability objectives in the seismic design of civil infrastructure. The proposed approach considers resilience objectives through the seismic design for damage control philosophy, aimed at minimizing the probability of requiring safety-critical repairs following a design-level earthquake. Sustainability objectives are considered by setting limits on two primary metrics – upfront embodied carbon and carbon associated with post-earthquake interventions. This study suggests thresholds for these sustainability metrics. The proposed approach is demonstrated using a case-study reinforced concrete (RC) building in California. Results indicate that current code-conforming RC structures may not meet both resilience and sustainability objectives. In contrast, designing structures for damage control may result in a minimal embodied carbon footprint associated with post-earthquake intervention following design-level seismic events.

Introduction

Recent seismic events have shown that while modern design approaches (e.g., ASCE 2022; Standards New Zealand 2004) have ensured a high probability of satisfying life safety objectives following earthquakes, they often lead to high repair costs, significant building downtime, and substantial environmental effects from pollution associated with building demolitions and intervention-related carbon impacts (Brown and Milke 2012; Gotoh et al. 2002; Marquis et al. 2017; Pan et al. 2014). It has also been highlighted that a significant gap exists between codified design provisions and desired performance objectives by building owners and society at large (Brown et al. 2022; EERI 2019). Furthermore, the rising urgency of addressing climate change requires a systematic design approach that minimizes embodied carbon while maintaining desirable seismic performance.

With an interest in fostering the design of resilient and sustainable civil infrastructure, policymakers and stakeholders are advocating for improved seismic design provisions (EERI 2019; NIST-FEMA 2021; Senate Bill 1768 2018), which can help to reduce post-disaster functionality loss and improve recovery rate (including reducing recovery time). Furthermore, there is a strong interest in minimizing the lifecycle environmental impact of structures.

Resilience-based design principles have evolved in recent years. Studies (e.g., Cook et al. 2022) have developed resilience-based design approaches aimed at minimizing post-earthquake downtimes. Other studies (e.g., Opabola and Elwood 2024) have proposed a seismic design for damage control framework aimed at minimizing damage to structural and non-structural components through component-level deformation design limits corresponding to the damage-control limit state. The damage-control limit state is defined as the post-earthquake state beyond which the strength and deformation capacity of a structural component is compromised, and its performance in future events cannot be guaranteed without safety-critical repair. Opabola and Elwood (2024) demonstrated that buildings designed for damage control may be able to achieve functional recovery within a short period of time.

Several studies have looked at lifecycle environmental impact assessment of structures in earthquake-prone regions. Given the scope of this paper, the discussion will focus on studies related to code-conforming RC buildings. Studies (e.g., Angeles et al. 2021; Feese et al. 2015; Padgett and Li 2016) have looked at lifecycle assessment of code-conforming RC structures. Other studies (e.g., Gencturk et al. 2016; Welsh-Huggins and Liel 2017) have quantified environmental impacts associated with increased design strength and/or ductility capacity (i.e., beyond code requirements) of RC structures. The results from these studies highlight the potential trade-off in upfront embodied carbon relative to lifecycle seismic performance and future post-earthquake losses.

Despite the growing literature on resilience and sustainability considerations in seismic design, there are a few gaps that need to be explored. Similar to studies aimed at developing design limits for resilience, it is important to explore how to develop sustainability metrics for seismic design purposes. Also, while studies have explored environmental impacts associated with enhanced seismic design from the perspective of increased strength (i.e., reduced response modification factors), it is important to quantify environmental impacts associated with deformation limits derived from the seismic design for damage control approach.

This paper introduces a multiobjective design methodology that integrates resilience and sustainability goals within the performance-based seismic design framework. The approach incorporates resilience by adopting the seismic design for damage control philosophy. To address sustainability, the framework sets specific limits on two primary metrics: upfront embodied carbon and the embodied carbon linked to post-earthquake intervention. An approach for defining thresholds for these sustainability metrics are proposed within the study. The framework is applied to a case study of an RC building in California.

Methodology

Overview. The proposed approach for integrating resilience and sustainability objectives into performance-based seismic design is described in this section. As shown in Figure 1, the first steps in the methodology entail designing the structural system using the seismic design for damage control approach (Opabola and Elwood 2024). More background on the design approach is presented subsequently. Once the structure is designed, the upfront embodied carbon (EC_{up}) is estimated. EC_{up} is then compared with a defined upfront embodied carbon limit ($EC_{up,lim}$). In a case where EC_{up} exceeds $EC_{up,lim}$, material selection and structural optimization (i.e., select structural materials with lower embodied carbon without compromising the structural resilience) is necessary, and the entire design process is restarted. Further discussion on how $EC_{up,lim}$ can be defined is presented subsequently.

In a case where EC_{up} does not exceed $EC_{up,lim}$, following the performance-based earthquake engineering (PBEE) framework (Moehle and Deierlein 2004), nonlinear response history analysis outputs are combined with component-level fragility models to define the damage states of structural and non-structural components. Subsequently, an environmental impact assessment of post-earthquake intervention to restore damaged components is carried out to estimate the embodied carbon associated with post-earthquake intervention (EC_{dam}). EC_{dam} is then compared with a defined limit for embodied carbon associated with post-earthquake intervention ($EC_{dam,lim}$). In a case where EC_{dam} exceeds $EC_{dam,lim}$, further design iteration is required. Otherwise, the engineer can check if monetary losses, downtime, and casualties due to seismic events govern the decision-making process. Further discussion on $EC_{dam,lim}$ is presented subsequently.

Seismic design for damage control. For brevity's sake, detailed information on the seismic design for damage control approach is not provided here. Interested readers are referred to Opabola and Elwood (2024). Opabola et al. (2023) examined datasets of ductile RC beam, column, and wall specimens subjected to various loading protocols to understand (a) the deformation demand beyond which the residual capacity of an RC component is compromised; and (b) the influence of prior loading history on future performance. The authors concluded that, aside from a reduction in its initial stiffness, as long as the deformation at the initiation of lateral strength loss (LSL) of an RC component is not exceeded in any prior loading history (and low-cycle fatigue has not been triggered), the component's residual capacity (both in terms of strength

and deformation capacity) is unlikely to be compromised. More information on approaches to low-cycle fatigue assessment of reinforcement are provided in Elwood et al. (2021). Opabola and Elwood (2024) proposed a framework for defining the damage control limit state for structural components based on the deformation at the initiation of LSL.

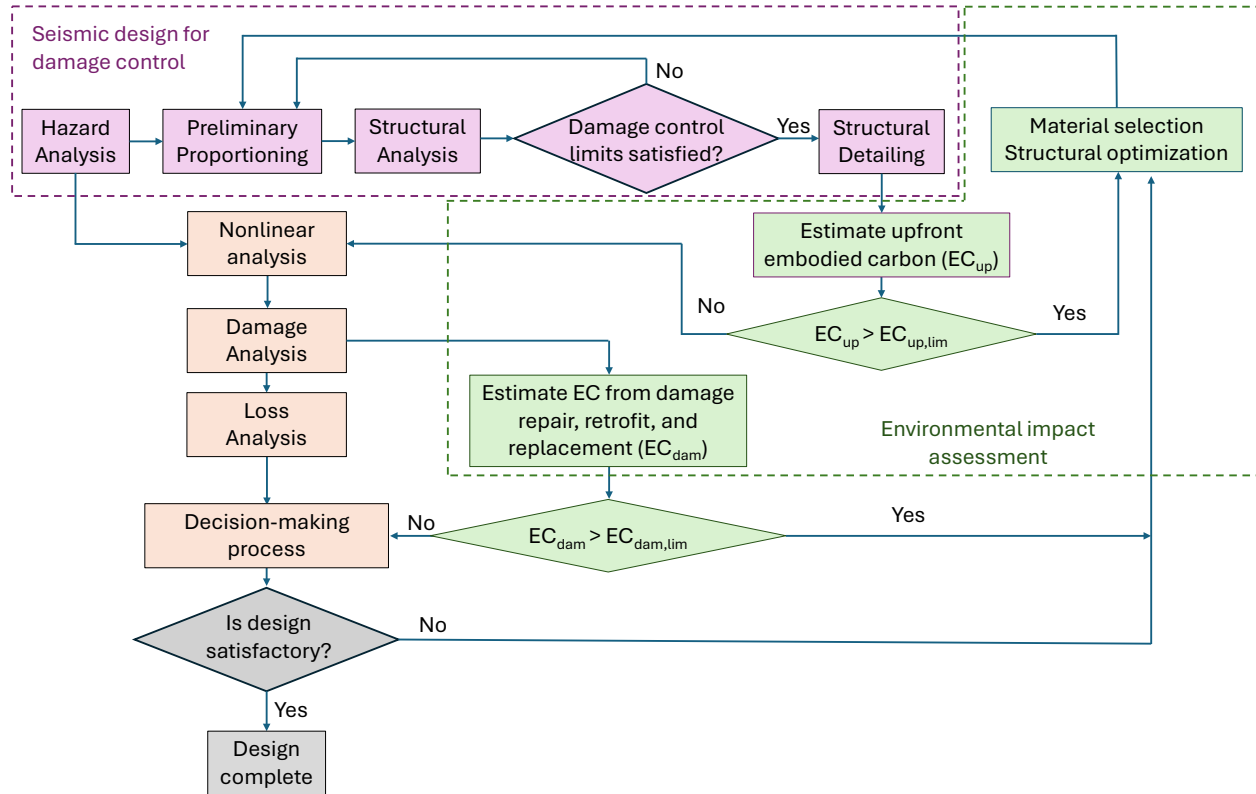


Figure 1. Proposed framework for integrating resilience-based and sustainability-based design approaches

From a probabilistic analysis point of view, to control earthquake-induced damage, there has to be a low probability of the deformation demands from a given hazard level exceeding the deformation at the onset of LSL. It is also important to account for all sources of epistemic and aleatory uncertainties in such a probabilistic analysis. Considered uncertainties in the Opabola and Elwood (2024) study: (a) uncertainty in predicting the drift at LSL (β_c) taken as 0.3 (a lognormal distribution is assumed based on test data), (b) uncertainty in the influence of loading history on the drift capacity at LSL (β_{LP}) taken as 0.15, (c) design requirements uncertainty (β_{DR}) taken as 0.2 (FEMA 2009) (d) construction quality assurance (β_{CQ}) taken as 0.25, (e) modeling uncertainty (β_M) taken as 0.25 (FEMA 2012), and (f) record-to-record uncertainty (β_{RTR}) taken as 0.40 (FEMA 2012). The total component-level uncertainty (β_{TOT}) can be estimated as the square root of the sum of squares of relevant uncertainties. Acceptable values of damage control margin ratio (defined as the ratio of median deformation at LSL to median component deformation demand) can be estimated based on the assumption that the distribution of the component demand is also lognormal.

A β_{TOT} of about 0.65 is computed. A β_{TOT} of 0.65 corresponds to damage control margin ratios of 0.43 and 0.58 for 10% and 20% probabilities of exceeding the initiation of LSL, respectively. By multiplying the median estimate of the deformation at the initiation of LSL with the damage control margin ratios, Table 1 presents the damage control limits for RC components, assuming that the probabilities of exceeding the initiation of LSL are 10% and 20%. Although not presented here, deformation limits have been developed

for coupling beams and slab-column connections. A similar procedure for deriving damage-control limits can also be developed for acceleration-sensitive and drift-sensitive non-structural components. However, this is outside the scope of this paper.

Table 1. Damage control deformation limits (hinge rotations) for RC components for 10% and 20% probability of exceeding the initiation of lateral strength loss (LSL)

Component	Design limits based on the probability of exceeding deformation at initiation of LSL (P(LSL))	
	10%	20%
Beams	0.015	0.02
Columns	$0.02 - 0.02 N/A_g f'_c$	$0.03 - 0.03 N/A_g f'_c$
Walls	$0.3\% \leq \theta_{w,lim} = 0.013 - 0.00013 l_w c/b^2 \leq 1.2\%$	$0.5\% \leq \theta_{w,lim} = 0.018 - 0.00017 l_w c/b^2 \leq 1.6\%$

N – Axial load, A_g – gross concrete area, f'_c – concrete compressive strength, l_w – wall length, c – neutral axis depth, b – wall thickness, $\theta_{w,lim}$ – design deformation for wall.

Using the defined design limits, engineers can adopt the design limits in traditional seismic design procedures. Through a series of recovery analyses (using the ATC-138 (FEMA 2021) methodology) on RC archetypal structures, Opabola and Elwood (2024) demonstrated that the defined limits can help structures recovery-based performance objectives.

Defining sustainability metrics. As shown in Figure 1, the design procedure requires $EC_{up,lim}$ and $EC_{dam,lim}$. Current design code provisions provide little to no information on thresholds for upfront embodied carbon. Hence, this study explored a simple approach to define thresholds for upfront embodied carbon based on typical material use, construction practices, and project scale.

Available data (CLF 2017) show that up to 65% of the upfront embodied carbon of a structure is attributed to the structural system – of which the superstructure contributes about 70%. This study focuses on defining limits on upfront embodied carbon for the superstructure. Figure 2a presents data on upfront embodied carbon for over 1000 buildings – of which 637 buildings are in North America (CLF 2017). Using the statistical distribution of the commercial and residential buildings dataset, sustainability classes were derived. This study adopted key percentiles as reference points. This approach would allow for further refinement of the class bounds as more data becomes available. Table 2 presents the defined sustainability classes. The 10th, 25th percentile, median (50th percentile), and 75th percentile provided a natural basis for categorizing buildings into sustainability classes, with higher percentiles indicating higher embodied carbon values. Class I and Class II effectively capture buildings with higher embodied carbon values (top 50%), which aligns with lower sustainability. Class III captures buildings with moderate carbon values, around the median and 25th percentile, making them moderately sustainable. Classes IV and V reflect buildings with low embodied carbon, particularly in the bottom 25% of the distribution, aligning with high sustainability standards. Decision-makers can choose a desired sustainability class and select a target $EC_{up,lim}$ that will be used in the design process.

The FEMA P-58 methodology provides a framework for estimating the embodied carbon associated with repairing, rehabilitating, or replacing damaged components following an earthquake. Given the larger uncertainty in predicting EC_{dam} (in comparison with EC_{up}), it is recommended that a probabilistic approach be adopted in defining the design limit for EC_{dam} . This can be in the form of a defined probability of exceeding a threshold $EC_{dam,lim}$ (i.e., $P(EC_{dam} > EC_{dam,lim})$). This study recommends defining $EC_{dam,lim}$ as a fraction (i.e., 10-15%) of the structure's total upfront embodied carbon ($EC_{up,tot}$) – including construction, non-structural and structural systems). Future studies will look at defining acceptable values of $EC_{dam,lim}$ and $P(EC_{dam} > EC_{dam,lim})$.

Material selection and structural optimization. Material selection and structural optimization are very important when the sustainability objectives are unsatisfied. Sustainable material choices (e.g., low-carbon concrete, recycled aggregates, and alternative cementitious materials) may necessary to design the superstructure for higher sustainability classes. However, the majority of studies on low-carbon concrete

mainly focus on the concrete compressive strength, without considering mechanical properties (e.g., elastic modulus) that are relevant for seismic performance. Figure 2b showcases experimental data on the impact of fly ash content on the elastic modulus of concrete. The figure shows that minimizing embodied carbon through alternative cementitious materials may significantly impact the elastic modulus. Hence, care must be taken during material selection. Structural optimization involves topology optimization to determine optimal load paths, shape optimization to refine structural configurations, and sizing optimization for precise element dimensions. Structural optimization could help engineers decide whether to adopt low-damage technologies (e.g., base isolation devices) in cases where it becomes difficult to satisfy both resilience and sustainability metrics.

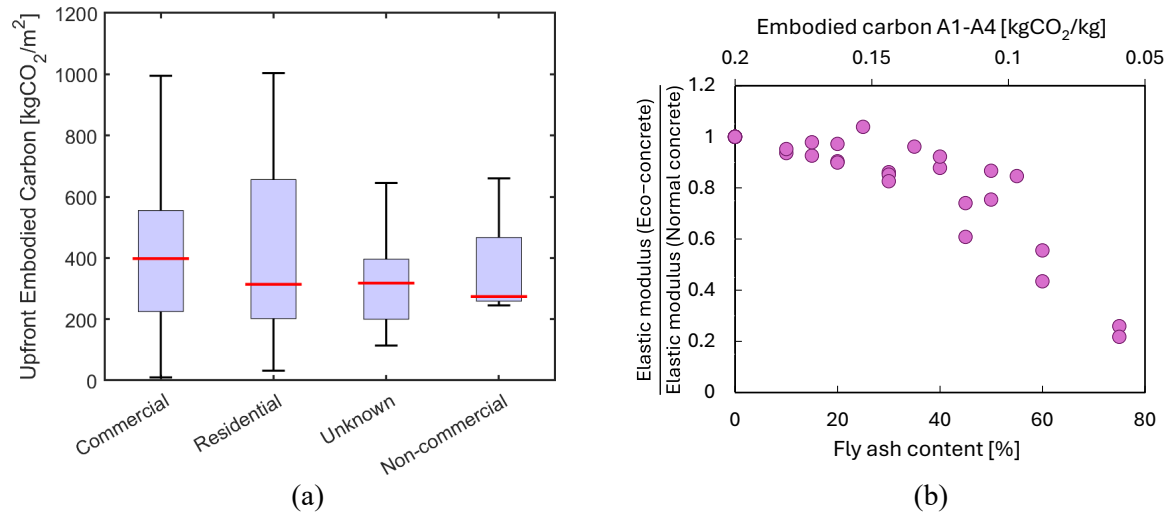


Figure 2. (a) Data on upfront embodied carbon for the superstructure of over 1000 buildings (CLF 2017) (b) Experimental data on the impact of fly-ash content on the elastic modulus of concrete

Table 2. Sustainability classes for structural systems based on upfront embodied carbon for the superstructure

Sustainability class	Range (kgCO ₂ /m ²)	Definition
Class I	≥ 550	Based on the 75 th percentile for commercial and residential buildings
Class II	350 – 550	Based on the 50 th percentile for commercial and residential buildings
Class III	200 – 350	Based on the 25 th percentile for commercial and residential buildings
Class IV	100 – 200	Based on the 10 th percentile for commercial and residential buildings
Class V	< 100	

Case study

The proposed methodology is adopted to design an eight-story RC wall building on a downtown Los Angeles site, representative of the high seismicity of the coastal California region. With soil class C, the site has a short-period spectral acceleration (S_a) of $S_{DS} = 1.0g$ and one-second period S_a of $S_{D1} = 0.6g$. The gravity and seismic loads are derived following ASCE/SEI 7-22. The typical dead and live loads are 125psf and 50psf, respectively. The roof dead and live loads are 125psf and 20psf, respectively. Figure 3 presents the wall layout of the building.

The following design objectives are set for the structure: (a) the probability of requiring safety-critical repair at design level (DE) ($P(SCR|DE)$) must be less than 20%, (b) an upper limit of 250kgCO₂/m² is set as the upfront embodied carbon for the superstructure (Class III), and (c) $P(EC_{dam} > 60\text{kgCO}_2/\text{m}^2|DE) < 10\%$. The

selected $EC_{dam,lim}$ is defined from the selected upper limit of EC_{up} as $0.1EC_{up,tot}$ and the superstructure contributes only 40% of $EC_{up,tot}$.

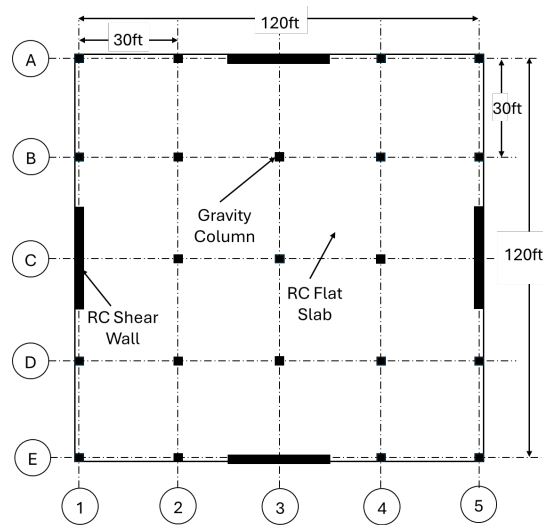


Figure 3. (a) Wall layout for the code-minimum structure

For each design iteration, linear analysis is carried out using the modal response spectrum method in ASCE/SEI 7-22, followed by structural detailing using ACI 318-19 provisions. Subsequently, a nonlinear model of the structure is developed in Opensees. The walls are modeled as two-dimensional cantilever MVLEM models with a fixed base, and a P-Delta column is used to capture secondary global nonlinearities. 2% Rayleigh damping was adopted for the analyses. Multiple stripe analyses were conducted with 20 ground motions selected and scaled to intensity levels ranging from 0.25 times DE to 3 times DE. The damage and loss analyses and the embodied carbon from post-earthquake intervention are based on the FEMA P-58 performance assessment methodology.

For the purpose of this study, three design variants are highlighted – (a) code-minimum; (b) damage-control design with deformation limits corresponding to $P(LSL) = 20\%$; (c) damage-control design with deformation limits corresponding to $P(LSL) = 10\%$. The design variables for the three archetypes are presented in Table 3. All three structures were designed using normal concrete.

Table 3. Design variables for the three considered archetypes

Archetype	Wall dimension (mm x mm)	Fundamental period T_1 (sec)	Design maximum interstory drift [%]	Design roof drift [%]
Code minimum	6706 x 508	1.6	1.7	1.2
Damage control ($P(LSL) = 20\%$)	7315 x 508	1.3	1.4	1.0
Damage control ($P(LSL) = 10\%$)	7925 x 660	1.0	1.0	0.8

Figure 4 presents the analysis output for the design archetypes. All three structures satisfied the EC_{up} metric. As shown in the figure, while the code-minimum structure did not satisfy the defined resilience metric by a small margin, a more significant unsatisfactory margin is observed for the EC_{dam} metric. It is noted that EC_{dam} accounts for both structural and nonstructural systems. On the other hand, the archetypes designed for damage control satisfied both the resilience and sustainability metrics. The results indicate that structures designed for damage control may inherently satisfy EC_{dam} design requirements. Further analyses on a wider range of archetypes are needed to draw a broader conclusion.

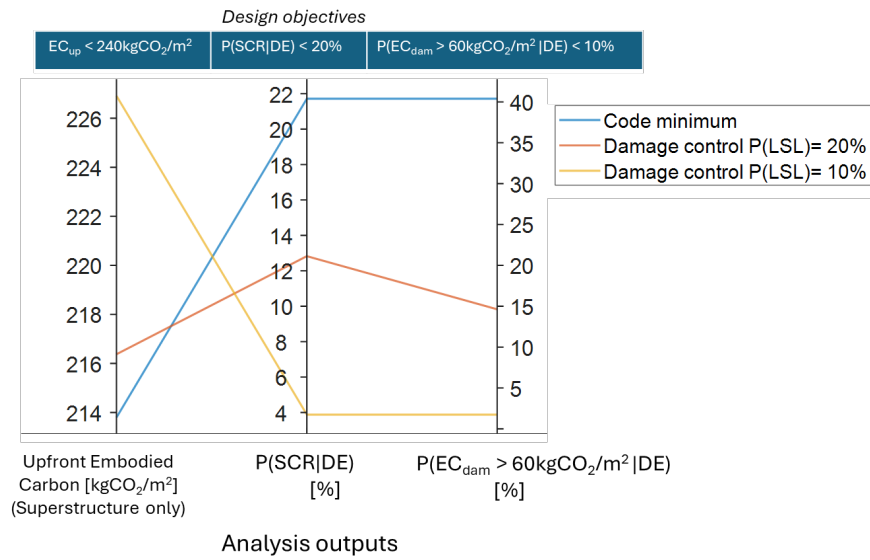


Figure 4. Analysis output for the three archetypes – Comparison of the upfront embodied carbon of the superstructure, probability of requiring safety-critical repair at DE [$P(DC|DE)$], and probability of the embodied carbon from seismic damage repair following a DE event exceeding $60 \text{ kgCO}_2/\text{m}^2$

Conclusion

This study presents a multiobjective design framework that integrates resilience and sustainability into seismic design for reinforced concrete structures, explicitly addressing seismic damage control and carbon impact mitigation challenges. The results from a case study on an RC building in California reveal that current code-compliant structures may fail to meet both resilience and sustainability goals simultaneously. By adopting a resilience-based design philosophy focused on damage control, it is possible to reduce the embodied carbon footprint associated with post-earthquake intervention. However, achieving low upfront embodied carbon in structures designed for such resilience objectives may be challenging, thereby necessitating careful material selection and structural optimization.

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DRIVING SUSTAINABLE DESIGN IN THE STRUCTURAL PROFESSION THROUGH ACADEMIC AND PRACTITIONER COLLABORATION

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Abstract

As New Zealand starts to tackle the challenge of transitioning to a low carbon economy, it's crucial for our building and construction industry to be responsive and knowledgeable on their role and impact. At present, this sector is globally responsible for about 40% of carbon emissions, with roughly half of that linked to embodied carbon, which can be influenced directly through structural design.

One of the largest current barriers to implementing low carbon design in practice is a lack of guidance and information - particularly a lack of location specific information - targeting the structural design space. Whilst practical guidance could be argued to best be developed by practicing engineers, the collaboration between academics and industry in the Structural Engineering Society of New Zealand (SESOC) Sustainable Design Task Force has resulted in progressive and soundly based technical guidance written specifically for structural engineers in New Zealand to use in their day-to-day work.

The Task Force has recently released the “Top Tips for Low Carbon Design”, which lays out 21 tips for a lower carbon outcome at the hand of structural engineers in Aotearoa New Zealand. The guide follows the design process, covering four main stages - integrating carbon management, influencing the system, influencing the detail and influencing the construction. The strength of this guide is that it looks at carbon alongside all the parallel considerations structural engineers have to balance, including but not exclusive to; seismic resilience, cost, constructability, durability, acoustic and fire performance.

Industry Context

As in many other countries, our construction sector has a growing understanding of the need to reduce annual greenhouse gas (GHG) emissions. In Aotearoa New Zealand, we are responsible for ~15% of annual emissions, increasing by 40% when including imported construction products (MfE, 2022). The Ministry of Building, Innovation and Employment’s (MBIE) Building for Climate Change Programme (BfCC) proposed new regulatory requirements, published in 2020 (MBIE, 2020). Under these requirements, the embodied carbon of a building would need to be calculated, reported and ultimately meet a cap for a building consent to be granted.

The term embodied carbon is used to describe the total GHG emissions associated with the lifetime of products or materials. It covers the whole life cycle, including raw material processing, manufacturing, any transportation to and from different locations, maintenance of the product and disposal of the product at the end of its life. These are described as “life cycle modules”. By monitoring embodied carbon throughout the design process, it can be used as a factor (alongside cost) to determine a project's success. In early project phases, the structural engineer is well placed to both calculate the embodied carbon and to advise the project team on structurally feasible lower-carbon alternatives. Figure 1 illustrates the gap between embodied carbon profiles for a ‘business as usual’ design process compared to one where carbon is accounted and considered throughout the process looking for reduction opportunities.

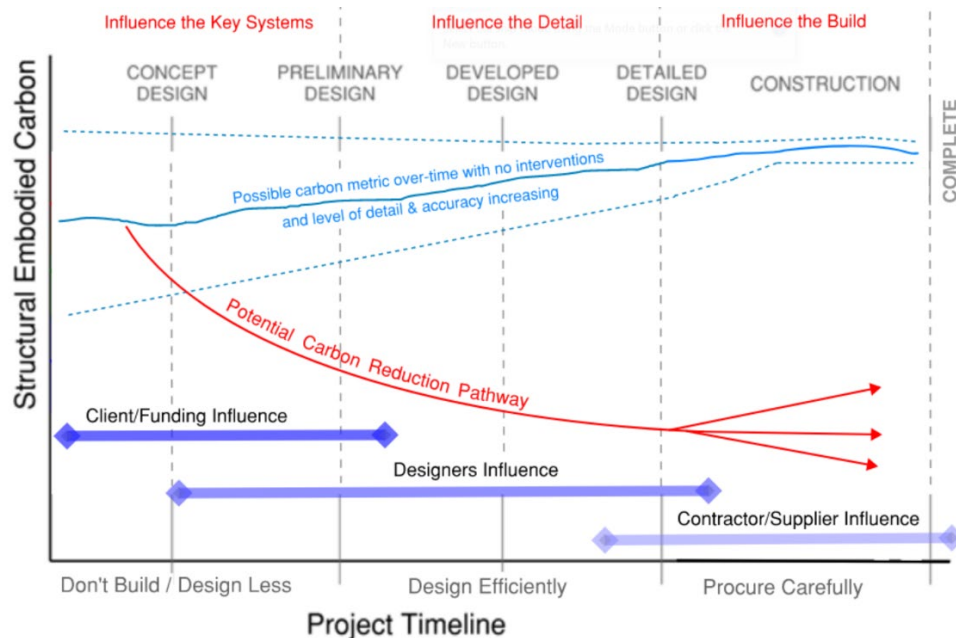


Figure 1: Figure of Carbon influence against design stage (courtesy Nick Carman)

Although there has been a lift in the understanding of the role of our actions in mitigating climate change and the need for drastic action to be taken within general society, behavioural change within the structural engineering industry, and the wider building industry has been limited. There has been debate on whether this is part of the core role of structural engineers, and a pushback on how to integrate carbon thinking into the design process when it is not asked for, or valued by the client and wider design team.

SESOC, the Structural Engineering Society of New Zealand, initiated the Sustainable Design Task Force in 2021 with focus areas of: providing feedback on consultation documents linking sustainability and structural engineering, supporting the upskilling of the wider SESOC membership in the space of sustainable structural design, and to continue to build relationships with collaborative organisations. Initially being a small group of 6 volunteers, the Task Force has grown to a team of 20 including consulting engineers, academics, material specialists, contractors, carbon experts, and regulatory officials. One of our early achievements was to publish the SESOC Position Statement on Sustainable Design, and to amend the technical societies purposes to include ‘Promote the design of safe economical structures, built to recognised standards and sustainable design principles, which are of benefit to all New Zealanders’.

With the range of skills and experience across the Task Force, this group has been uniquely placed to drive the development and dissemination of structural focused sustainability resources, to sit alongside more general built environment focused resources. In Aotearoa New Zealand, BRANZ (Building Research Association NZ) and the New Zealand Green Building Council (NZGBC) have developed an impressive repository of freely available embodied carbon information, including research reports, databases and tools.

Drive and uptake within Industry

One of the largest current barriers to implementing low carbon design in practice is a lack of guidance and information - particularly a lack of location specific information - targeting the structural design space. To ensure the guidance and resources that the Task Force developed hit at the right level, the first segment of work undertaken comprised an online survey of the SESOC membership, and a series of focus groups, to

establish the current baseline for understanding of sustainability amongst structural engineers in Aotearoa New Zealand. The survey was developed in collaboration with the Institute of Structural Engineers (IStructE) to align with their March 2023 survey entitled “Sustainability Skills Survey”. The purpose of this alignment was to enable the research team to compare and contrast responses between the global average and the Aotearoa New Zealand context. At the time of writing, the IStructE have yet to release the findings from this survey, therefore comparison has not been possible.

The demographics across the survey respondents indicated that 85% worked on privately funded projects, with the majority classifying their projects as small/medium in scale. The experience of the respondents was split 20% identifying as early career, and 40% mid-career, with the remainder 40% in senior leadership.

The key findings from the survey are summarised below:

- Respondents generally felt they had good knowledge and understanding of all the sustainability knowledge areas, however few felt they had experience in incorporating that knowledge into their work.
- ~20% of respondents felt they had no knowledge or understanding of any of the sustainability knowledge areas, with 5% stating that they did not think sustainability areas of knowledge were relevant.
- Structural engineering organisations may be doing more ‘talking’ than ‘doing’ at the moment – with the majority of respondents seeing their organisations putting sustainability into their strategy, but much fewer taking specific actions towards those goals.
- Carbon reduction and minimising use of materials were considered important for respondents, their organisations and their clients.
- Webinars and guidance books were considered the most preferred ways of learning. This may be reflected in the fact that fewer than a third of respondents had accessed either the SESOC or IStructE’s online resource library.

A few interesting findings from the survey are presented below. Participants were asked ‘How would you rate the importance of sustainability for your organisation’, results shown in Figure 2 show that a small percentage state that sustainability is not rated as important at all in their organisation, where 20% stated that it is only considered if specially requested, and just over 30% indicating that it is a main factor alongside cost and design efficiency. The participants were then asked to identify their organisations main action areas within sustainability, where measuring embodied carbon was evidently the dominant action, followed by minimising materials, questioning the brief – and nothing at all, which all rated similarly but were significantly lower.

Although participants generally rated their organisations valuing of sustainability highly, Figure 3 shows that in terms of actions, the key action item was ‘putting sustainability at the core of their strategy’ – which could be construed as more ‘talking’ than ‘doing’ – with very few respondents noting action items such as bench marking projects, setting public commitments, or investing in training.

Q: How would you rate the importance of sustainability for your organisation?

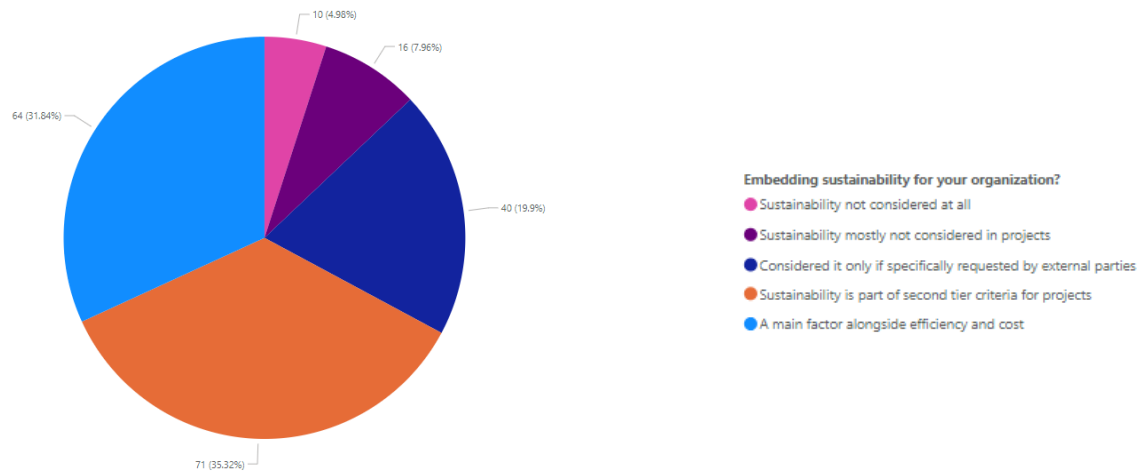


Figure 2: Findings from the Sustainability Skills Survey 2023

Q: Which of the following are your organisation doing to enable effective sustainability outcomes?

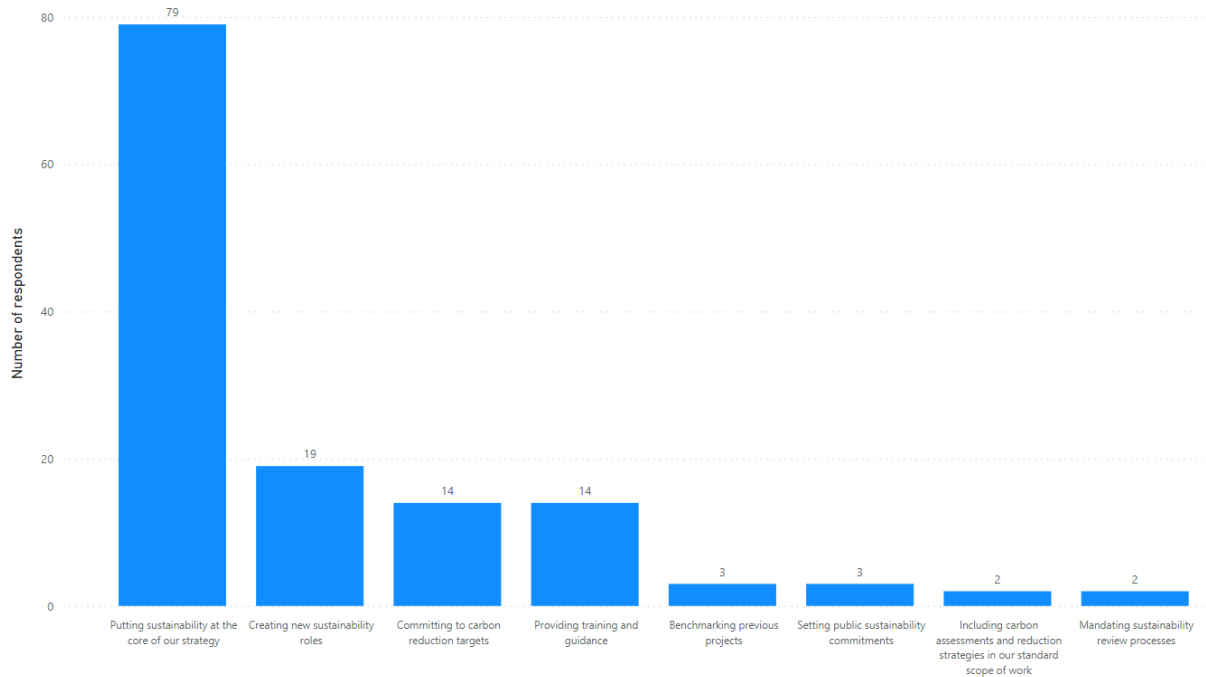


Figure 3: Findings from the SESOC Sustainability Skills Survey 2023

Opportunities and Barriers

The barriers and opportunities to implementing low carbon design, which were discussed in the focus groups, and throughout the survey are summarised in Table 1. Participants were asked how they would rate the importance of embedding sustainability for their clients, with 34% indicating that they felt sustainability was not valued at all by their clients on their projects, with only 10% stating it was a primary factor alongside efficiency and cost. Also of note, ‘meeting publicly declared targets’ was identified as the perceived main driver for clients in adopting sustainable design practices. This indicates that structural engineers believe that their clients have a lack of mandate to pursue lower carbon solutions.

Table 1: Barriers and Opportunities to Low Carbon Implementation

Opportunities	Barriers
“Build back better” after disasters	Differentiating responsibility in design team
Masterspec inclusions	Reliable and trustworthy material data
Junior engineers hosting knowledge	Lack of residential guidance
Incorporate into ethics requirements	Publicly available benchmarks

Guidance development – ‘Top Tips for Low Carbon Design’

In recognition that Guidance Books were rated as the most preferred method of receiving information, The “Top Tips for Low Carbon Design”, ready-to-implement as is, guidance was developed. The document is written to be a cohesive single guide, with connections throughout, however each “Top Tip” is also able to be read in isolation from the others. This guide has been produced to help inform and assist structural engineers in their efforts to decarbonise their designs, specifically for the Aotearoa New Zealand context. Each section of the guide will share a 'Top Tip' either from a design or management perspective. If applied successfully, the tips in this guide should result in ready to implement knowledge to reduce embodied carbon emissions from structures within the design process. Whilst some tips are more focused on buildings, most principles are applicable to all structural projects.

Top Tips - Integrate carbon management

This section of the guide focuses on those areas of practice which enable lower-carbon emissions to be sought through the design and construction process. These “Top Tips” were grouped together to indicate actions or systems which should be considered at all stages of the project, from inception through to practical completion. Carbon management covers points including calculate, communicate the impacts, the role of coordination across the design team, and engagement with the contractor.

Top Tips - Influence the system

This section of the guide covers decision-making which is likely to occur at early stages of design, from feasibility through to concept and preliminary design. Generally, these “Top Tips” are written to enable holistic low-carbon outcomes, not just for structural elements but for the building as a whole.

Top Tip #6 ‘Interrogate user requirements and refine assumptions’, directly ties into the “Challenging the brief” knowledge area of sustainability which the survey covered, this tip dives into more detail onto the types of questions that structural engineers can ask to influence the direction of design towards a lower carbon outcome. Top Tip #7 ‘Define optimal geometries’ discusses the impact of adopting structurally efficient grid layouts. Top Tip #8 ‘Sensibly maximise the life of the structure’ addresses some of the debate around how the life of the structure plays into the conversation around whole-of-life carbon. Top Tip #9 ‘Build up not down’ looks at the carbon impact of building below ground level.

Top Tips - Influence the details

Focussing on latter design stages, these “Top Tips” provide commentary and suggestions for lowering carbon emissions in specific structural elements. This section is further broken down into ‘focus areas’ – foundations, framing, floors, secondary & residential. Each of these areas includes specific actions and suggestions for structural engineers to implement into their day-to-day designs, deliberately taking a “small steps” approach to each area. Practical design considerations are discussed, along with any compromises which need to be thought about and resolved alongside a low carbon solution.

Within the Foundation section, the guide focuses on choosing the lowest volume foundation, and looking at how we specify our concrete strengths to give the lowest carbon solution.

For Primary framing, the tips look at the carbon benefits of optimising member design over rationalising for simplicity, noting that rationalisation often results in underutilisation of members in structures. In the design of primary framing, the relationship between seismic resilience and material efficiency - particularly for steel and reinforced concrete, the more carbon intensive materials, is discussed. The relationship between drift and damage is highlighted, encouraging designers to think about repairability and move away from the highly ductile ‘single-event’ design thinking.

Recognising that the embodied carbon of a structure can be heavily weighted towards the floor elements, the Top Tips include comparing the embodied carbon of multiple floor systems, and knowing how to make low carbon flooring systems work well – acknowledging the potential further design considerations of adopting timber floor systems including diaphragms design, fire, acoustics and vibration effects. Finally in the details section the guide includes guidance on low carbon design strategies for residential and partition wall framing, and ensuring the architectural elements are included when comparing options for walls, ceiling and floors.

Top Tips - Influence the build

This section of the guide provides key details for structural engineers to be able to consider and manage the carbon intensity of materials at the tender and procurement stage. Each primary structural material has its own tip which focuses on how to specify that material well, rather than comparing between materials. This was intended to provide a material agnostic approach to specification, and also to encourage each material industry to do better in owning and reducing their emissions.

Links between seismic resilience and carbon

Improved seismic resilience, or damage reduction is one of the current challenges for structural engineers, and this now sits parallel with the very real and pressing need to actively reduce the embodied carbon in the structures being designed. Seismic resilience has links with sustainable design when looked at from both the upfront carbon of the structure, and also from the potential damage and repair or likelihood of demolition and rebuild across the structure’s life as a result of performance during a seismic event. Standard Life Cycle Assessment (LCA) practices do not have a mechanism to incorporate the likely damage and repair processes necessary following natural hazard events throughout the design life. For this reason – there is also no mechanism to value the potential benefits of designing more seismically resilient structures within the LCA whole-of-life framework.

Applying probabilistic modelling, ongoing research by Toma and Stephens at the University of Auckland has shown that the carbon emissions from damage induced by earthquakes is dominated by the risk that the structure is not repairable after an event. In this situation, the entire building would be demolished, potentially before the end of its useful life, resulting in emissions from the demolition and also from the construction of a replacement building in future. Minimising the potential for a structure to be considered unrepairable can therefore reduce the building’s life cycle seismic carbon risk. There are many structural and non-structural factors that can trigger a non-repairable scenario. For example, (Kim et al., 2017)

identified major factors such as building features (e.g., extent of damage, residual capacity, etc.), and contextual factors (e.g., cost of repair, insurance, etc.) for post-earthquake decisions after the 2010/2011 Canterbury Earthquake sequence. Seismic loss estimation is highly sensitive to the definition of the non-repairable scenario (Safiey et al., 2018). Currently the FEMA P-58 framework uses a fragility function that is based on residual drift and this has been implemented in several studies, however it is worth noting that NIST (2017) mentions that most analytical approaches have not been refined to capture residual drifts accurately. The concept of using residual drift as a sole predictor of reparability has been challenged by recent studies (Safiey et al., 2019; Safiey & Pang, 2021).

The work by Gonzalez, Toma and Stephens (2023) used inter-storey drift coupled with component-based acceptance criteria to define the non-repairable scenario. Figure 4 gives an indication of the sensitivity of the probabilistic seismic carbon risk to the threshold delineating reparability. Defining a suitable threshold is complex, as the damage to the primary lateral system does not often drive the decision making process around demolition. Using metrics tied to the lateral system performance are not necessarily appropriate however these are default outputs from numerical analysis.

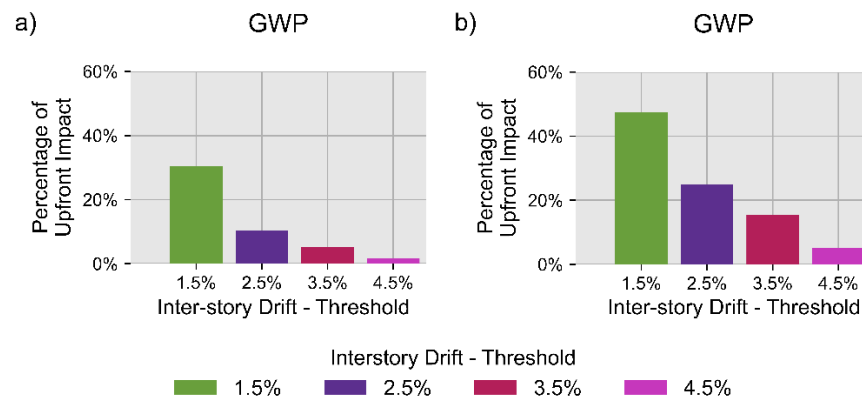


Figure 4: Total seismic carbon risk (GWP) over building lifespan for (a) steel frame building, and (b) a Hybrid concrete shearwall and moment frame structure (Gonzalez Espinel, 2024)

Second to the probability of an unreparable outcome, damage to the non-structural elements, including facades, MEP services, fitout and stairs and elevators has been shown to contribute significantly to the lifecycle seismic carbon risk. The damage profiles of these elements can be drift or acceleration controlled. Detailing which allows for this movement, limiting potential damage to the elements and connections will result in a lower carbon profile associated with damage and replacement of non-structural elements.

On the face of the argument, designing stiffer, seismically resilient structures to minimise damage to the building as a whole would be a logical solution. However, the relationship between seismically resilient and low carbon structures becomes more complex when looked at through the sustainability lens. The IPCC climate scientists have calculated a carbon 'budget' of allowable emissions beyond which the impacts on the globe become exponentially more complex, interactive and severe. Many of these impacts are expected to be irreversible, meaning that once we exceed the budget, no amount of future carbon-capturing technology will get the earth back to how it used to be. It's a scary scenario, but really hammers home their core message "Every increment of global warming will intensify multiple and concurrent hazards". In practice, this means that we need to prioritize actions which result in significantly lower emissions in the short-term. When thinking about seismic resilience – this would then mean prioritising lower up-front carbon, over any potential reductions in future seismically linked carbon emissions.

Further to this argument, any emissions from potential seismic induced damage are subject to multiple layers of uncertainty. Firstly from the probability of a moderate to severe earthquake occurring during the buildings occupied life time, secondly from the uncertainty around the modelling of carbon emissions associated with materials and construction in the future, as well as emissions from the end of life phasing, and thirdly those emissions associated with future construction are likely to be lower based on the drive to net zero by 2050 (Symons & Moses, 2024).

In striving for improved resilience, we should be careful to not create excessively carbon intensive buildings, but rather look to balance improved resilience and therefore lower potential seismic carbon risk, against lower upfront carbon. Equally, in looking to reduce upfront embodied carbon, we should not be creating buildings prone to unusually high damage, noting that convention systems can be designed to achieve low-damage objectives. These systems are not typically characterised by significant increases in upfront embodied carbon, which may be seen in more complex low-damage systems requiring extensive carbon intensive substructures.

More research to understand the relationship between design characteristics and seismic carbon risk across a range of building forms, structural systems, and hazard zones is still in development.

Conclusions

It is critical that structural engineering practice takes accountability of their scope three emissions. The reductions needed to meet our carbon budgets set to limit global warming require more than the reductions which can be achieved solely through low-carbon material selection. Our design processes and thinking need to incorporate carbon analysis. Guidance documents developed by leaders in the structural design and carbon space are best placed to help upskill the structural engineering profession.

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MODELING POST-EARTHQUAKE FUNCTIONAL RECOVERY OF BRIDGES: FRAMEWORK DEVELOPMENT AND APPLICATION

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Abstract

Understanding the post-earthquake functional recovery of bridges is critical for assessing regional seismic resilience. The recovery process of damaged bridges is stochastic by nature, as it is affected by several factors such as post-earthquake closure decisions, impeding factors that inhibit repair activities, and times spent in repairing damaged components or replacing the entire bridge. This study proposes a framework to probabilistically model bridge post-earthquake functional recovery. The framework is comprised of three modules that probabilistically evaluate the functionality state, impeding factor time delays, and repair or replacement durations. Furthermore, a duration database used as the model input is compiled, based on interviews with Caltrans officials and California bridge practitioners. Various construction and logistic constraints are explicitly considered, including: (1) logical sequencings of time delays and repair activities, (2) required number of workers per work crew, (3) work crew allocation per damaged component, and (4) extended work hours. The developed framework and database are applied to a two-span box-girder bridge in Los Angeles, California, subjected to a Caltrans-specified hazard level. The effects of strength of damage correlations on recovery duration are examined by varying hypothetical damage scenarios.

Introduction

Rapid restoration of transportation networks following earthquake disruptions is essential owing to its considerable impacts on people's daily commutes, freight transport, and emergency medical responses. Modeling and understanding post-earthquake bridge functional recovery is an integral part of seismic resilience assessment of transportation networks that extend beyond the traditional force- or displacement-based seismic design and assessment.

The performance-based earthquake engineering (PBEE) methodology (Moehle and Deierlein, 2004) advanced the traditional seismic design which focusses on life safety to incorporate additional performance metrics such as economic loss. Mackie and others (Mackie et al., 2010) integrated the PBEE methodology originally developed for buildings into bridge probabilistic repair cost and duration assessments. The California Department of Transportation (Caltrans) Seismic Design Criteria (SDC) (Caltrans, 2018) specifies that the bridge seismic design in California should incorporate structural

analysis under two hazard levels - the Functional Evaluation Earthquake (FEE) and the Safety Evaluation Earthquake (SEE).

While the PBEE methodology provides valuable insights into estimating seismic decision variables, it overlooks the connection between system damage and the intended functionality, as well as the activities needed to fully or partially restore functionality within a certain period. There have been a few frameworks developed to model seismic functional recovery for bridges. For example, Padgett and DesRoches (2007), Misra et al. (2020), and Kameshwar et al. (2020) proposed probabilistic recovery models based on expert opinions to assess time-dependent functionality change as a function of the bridge system damage state. Karamlou and Bocchini (2017) developed a model to simulate recovery based on bridge component damage, and resource and logistic constraints. However, these frameworks did not comprehensively incorporate specific impeding factors and their associated durations and sequencing which may significantly delay the recovery process. Also, they did not consider the possibility of bridge partial reopening with additional restrictions, a scenario that is likely to occur in the post-earthquake environment.

This article describes a framework developed to simulate bridge post-earthquake functional recovery. A bridge repair and recovery database are embedded into the framework based on interviews with California bridge practitioners, contractors, and officials. These data include probable post-earthquake bridge closure decisions, impeding factor delays, and component repair and system replacement durations.

Framework Development

The workflow for assessing bridge post-earthquake functional recovery is shown in Figure 1. It is initiated by bridge component damage evaluation based on the results of seismic hazard analysis. The component damage distribution is related to: (1) the bridge system-level functionality state (e.g., fully functional and partial/complete closure), and (2) time spent repairing damaged bridge components or replacing the entire bridge. The system-level functionality state further influences impeding time delays that inhibit the repair efforts. After impeding time delays and repair or replacement durations are estimated, the bridge recovery time is calculated as the sum of the two.

Based on the authors' interviews with Caltrans officials, there are two phases during a bridge recovery process: the initial response phase, and the reopening phase. These two phases reflect the evolving bridge opening decisions by Caltrans based on the progress of repair activities. Schematic recovery trajectories indicating the timestamps of these two phases are shown in Figure 2. The initial response phase happens when Caltrans dispatches its inspection crews to make a first-response decision. The reopening phase happens when, as repair activities progress, a bridge is deemed structurally safe, yet there are secondary components (i.e., only affect serviceability but not safety) still under repair and their damage influences the bridge's load-carrying capacity. Note that the reopening phase may not always be triggered.

The complete dataset for individual impeding factors, duration estimates for performing repair activities, as well as their logical sequencings are documented in Wu et al. (2024+).

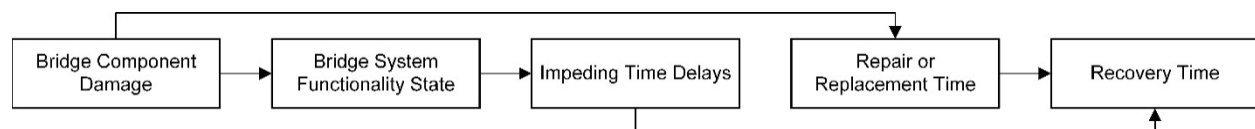


Figure 1. Workflow for assessing bridge post-earthquake functional recovery.

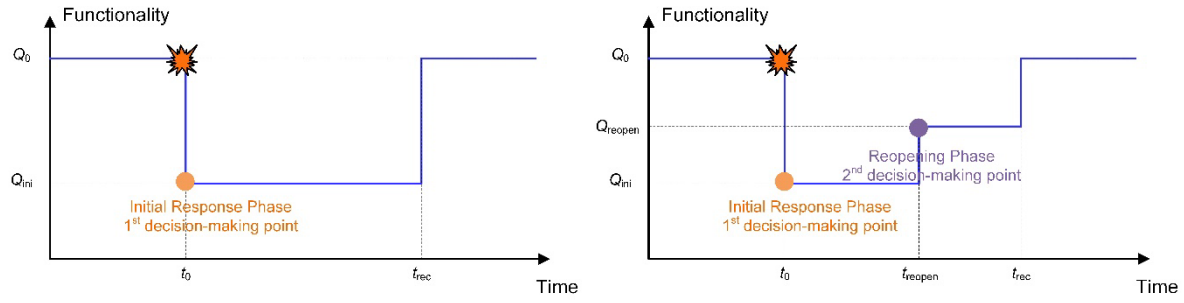


Figure 2. Schematic representation of bridge recovery trajectory: (left) with and (right) without a reopening phase.

Bridge Component Damage

A component-based strategy is used to simulate damage to the bridge. There are ten standard components that are considered in a regular bridge: column, abutment seat, girder-deck, column foundation, abutment foundation, bearing, shear key, abutment backwall, approach slab, and joint slab. For irregular bridges, the analyst can either approximate the irregular components as regular, or augment the component database developed in this study if there are fragility functions available.

The analyst specifies the quantity of each bridge component, and probabilistically samples their individual damage states under the considered evaluated seismic hazard level using fragility functions. These fragility functions can either be retrieved from prior studies evaluating similar archetype bridges, or developed by the analyst using the PBEE methodology.

Most prior studies adopted the independence assumptions when sampling bridge component damage. We adopt the functional form of the component capacity correlation model specified in Baker et al. (2024) with modifications:

$$\ln C_{ij} = \ln \theta_i + \beta_i (\sqrt{\alpha_{\text{inter}}} \varepsilon_{\text{inter}} + \sqrt{\alpha_{\text{intra},j}} \varepsilon_{\text{intra},j} + \sqrt{\alpha_i} \varepsilon_i), \quad (1)$$

where C_{ij} denotes the capacity for the i^{th} component belonging to the j^{th} group; θ_i and β_i are the median and dispersion of the component's fragility function; $\varepsilon_{\text{inter}}$, $\varepsilon_{\text{intra},j}$, and ε_i are three independent standard normal numbers. Weights are applied to these three random numbers, satisfying $\alpha_{\text{inter}} + \alpha_{\text{intra},j} + \alpha_i = 1$.

Bridge Functionality State

The bridge post-earthquake functionality state is determined by the severity of the damage distribution across the various components. Based on a review of historical bridge closure decisions and interviews with Caltrans officials, the bridge functionality states are divided as follows:

- FS1: Fully Functional.
- FS2: Moderate Lane Closure.
- FS3: Extensive Lane Closure.
- FS4: Partially Reopen with Weight Restriction.
- FS5: Partially Reopen with Minor Lane Closure.
- FS6: Partially Reopen with Weigh Restriction and Minor Lane Closure.

➤ FS7: Complete Lane Closure.

Note that FS 1-3 and 7 are applicable to both the initial response and reopening phases, and FS 4-6 are exclusive to the reopening phase.

Impeding Time Delays

Impeding factors are time delays that hinder the initiation of repair or replacement activities. Six impeding factors are defined: initial inspection, in-depth inspection, financing, design, permitting, and contractor acquisition.

The sequencing of the impeding factors was informed by the Caltrans interviews. Immediately after an earthquake, an initial inspection is performed to visually determine the extent of bridge damage. If the damage is difficult to identify visually, an in-depth inspection is requested. The sequencing of the remaining four impeding factors depends on whether an emergency protocol is initiated. If the emergency protocol is triggered, the financing acquisition procedure is waived, and the design and contractor acquisition occur simultaneously. If needed, the permitting process is initiated after the design is completed. If the emergency response is not triggered, the financing approval process is conducted at the same time with three series impeding factors, namely design, contractor acquisition, and permitting (if needed).

Repair or Replacement Time

If a bridge is considered to be irreparable (e.g., column collapse or excessive residual drift), it will be demolished and (most likely) replaced. In this case, the replacement time is estimated. If a bridge is deemed repairable, worker crews are assigned to each damaged component type to conduct the necessary repair activities. In this case, the total repair time is calculated.

In order to reflect the realistic post-earthquake repair efforts, the following construction and logistical constraints are incorporated:

- Logic sequencing of repair activities. Repair activities are not conducted completely in series or in parallel. Instead, the sequence is some combination of these two extremes. In general, the substructure-superstructure system and the abutment system, if both damaged, can be repaired at the same time (i.e., in parallel). In the substructure-superstructure system, the substructure components are repaired first followed by the superstructure. The repair sequence in the abutment is based on the relative locations of the constituent components.
- Required number of workers per crew. The minimum and maximum number of workers per crew are assigned for each component type to reflect the realistic construction practices and space constraints, respectively.
- Crew number assigned to each damaged component. Multiple worker crews can be assigned to each component type to repair multiple damaged instances of a given component type. The assigned crew number per component type should not exceed the number of damaged instances within each component type.
- Extended work hours for expedited repair or reconstruction. Under normal conditions, a standard eight-hour workday is applied to conduct repair and reconstruction activities. If emergency situations are triggered, multiple work shifts can be applied to accelerate the repair or replacement progress.

Example Application

To demonstrate the implementation of the proposed framework, we perform post-earthquake functional recovery assessments for a hypothetical two-span, multi-column, seat-type, box-girder concrete bridge. This bridge is located in the City of Los Angeles (Latitude 34.05°, Longitude 118.25°). The results from a probabilistic seismic hazard analysis determined that the spectral acceleration at a one-second period (SA_{1s}) corresponding to the Caltrans specified design earthquake level (i.e., Safety Evaluation Earthquake, SEE) equals 0.45g. Seismic damage to each bridge component under this hazard level is sampled from fragility models obtained from the existing literature (Figure 3). Note that secondary components (i.e., all components with exception of columns and seats) are only defined with slight and moderate damage, with the assumption that damage to these secondary components would not trigger severe restrictions (e.g., complete bridge closure).

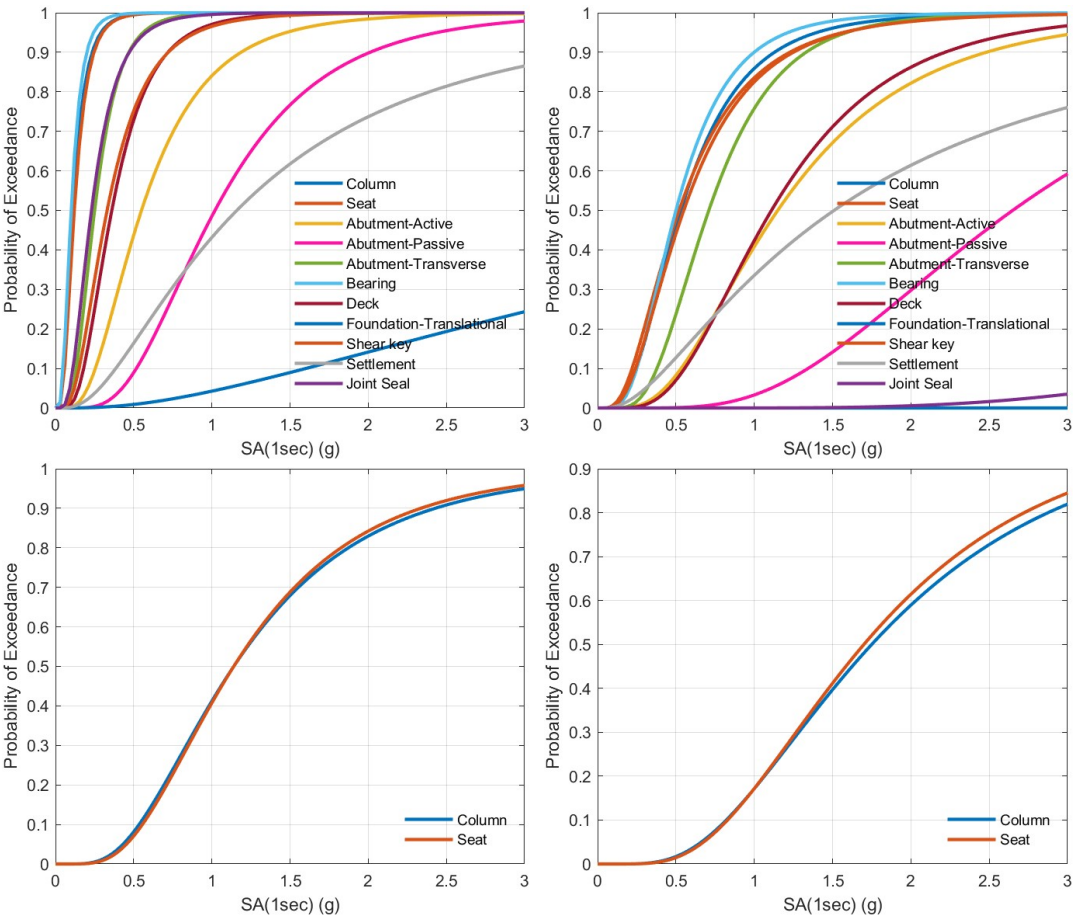


Figure 3. Fragility curves for bridge components under (top left) slight, (top right) moderate, (bottom left) extensive, and (bottom right) complete damage states (adopted from Mangalathu (2017), Shao et al. (2022), and Zheng et al. (2021)).

To investigate the effect of damage correlation on the functional recovery duration estimates, three hypothetical damage scenarios are included in Table 1 by varying the relative weight vector in Equation 1:

Table 1. Hypothetical weights assigned for different damage correlation scenarios.

<i>Correlation scenario</i>	<i>Weight vector</i>	<i>Remark</i>
Fully independent	$[\alpha_{\text{inter}}=0, \alpha_{\text{intra},j}=0, \alpha_i=1]$	Each component's capacity is simulated independently ⁵
Partially independent	$[\alpha_{\text{inter}}=0.2, \alpha_{\text{intra},j}=0.6, \alpha_i=0.2]$	Falls between the two extreme cases.
Fully dependent	$[\alpha_{\text{inter}}=1, \alpha_{\text{intra},j}=0, \alpha_i=0]$	All components from the same component type have the same damage state.

Figure 4 shows a scatter plot comparing the damage state counts for two distinct columns under different correlation scenarios. In the “Fully Dependent” scenario, the two columns share the same damage state under each realization. In the “Partially Dependent” case, the two components do not always share the same damage state due to the decreased correlation strength. In the “Independent” case, the probability that they fall into different damage states is the highest.

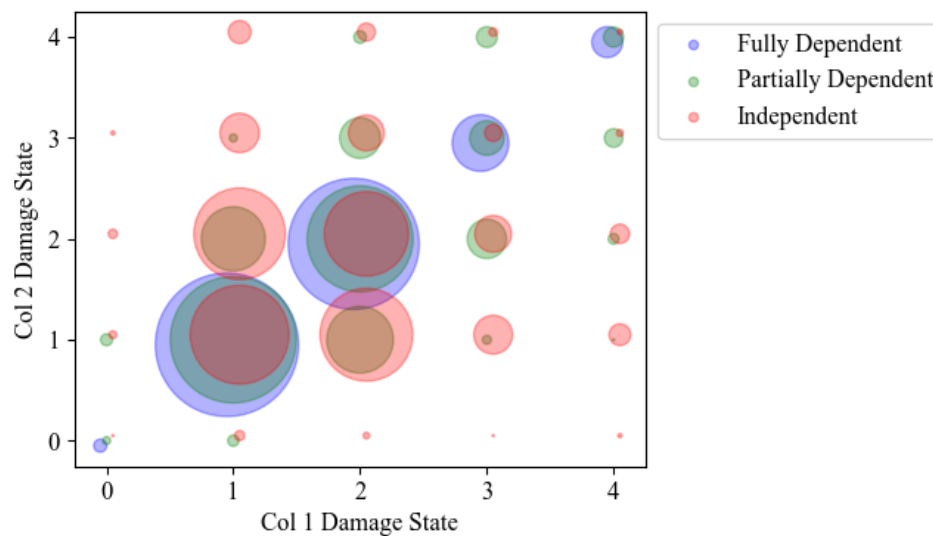


Figure 4. Component damage counts for two distinct columns. (The size of the scatter points is proportional to the counts of occurrences they represent. 0=No damage, 1=Slight, 2=Moderate, 3=Extensive, 4=Complete).

Figure 5 shows the complementary cumulative density functions (CCDFs) for the total time delays and total repair or replacement durations, under the three damage correlation scenarios. For the total time delays due to the impeding factors, the CCDF under the “fully independent” scenario is the lowest. In this case, the probability of a primary component having severe damage (i.e., extensive or complete) is the highest, making it more likely to trigger an expedited process to reduce time delays. The CCDF from the “partially dependent” case is in between the two extremes. This trend is less obvious in the total repair or

replacement duration, as the “partially dependent” scenario lies in the middle before roughly the 100-day duration and becomes the largest afterward.

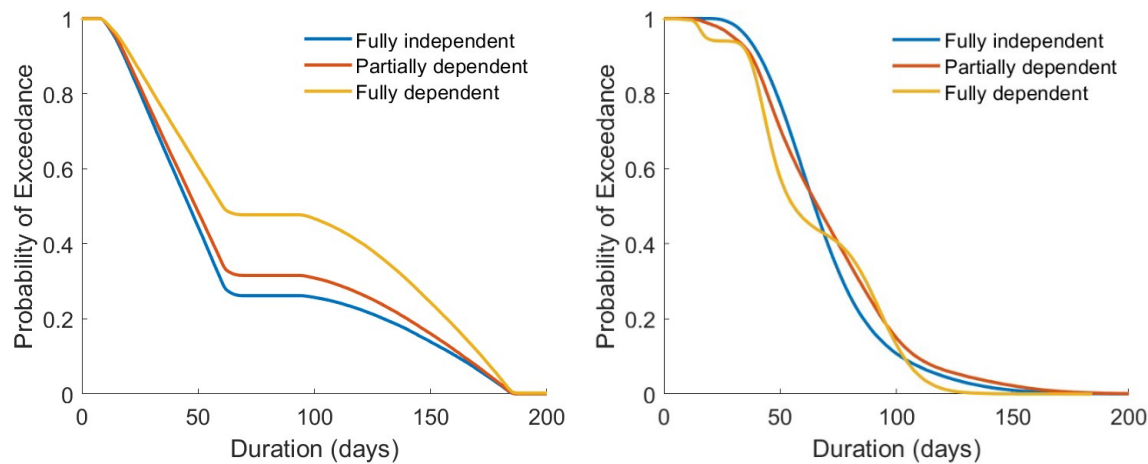


Figure 5. Empirical complementary cumulative distribution functions for the (let) total impeding time delays and (right) total repair or replacement durations.

Conclusion Remarks

A framework for probabilistically modeling bridge post-earthquake recovery is developed. A repair and recovery database is compiled as part of this framework based on interviews with California bridge engineers, contractors, and government officials. An illustrative example is presented, which estimates the probability distributions of bridge component damage states and recovery durations. The effects of varying strengths of damage correlation on the duration estimates are discussed.

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QUANTITATIVE RISK ANALYSIS FOR UTILITIES

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Abstract

Historically, electric utilities have used qualitative analysis to identify risks and prioritize short-term operational decisions and long-term mitigation planning. Recently, however, in response to regulatory pressure and to align with the state of practice in other fields of engineering, there has been a shift towards rigorous quantitative and probabilistic risk analysis to evaluate asset performance against a variety of hazards, in particular: wildfires and earthquakes. This paper will discuss several new utility risk models, including earthquake-induced landslide damage to wood poles and steel towers, wind-induced failure of overhead assets, wire-slapping, aeolian vibration of conductors, and contamination-induced insulator flashover. The paper will also discuss how use of probabilistic risk analysis has improved outcomes compared to other techniques. The challenges of data collection and analyzing vast systems with millions of assets will be highlighted.

Introduction

Risk analysis combines the probability of failures and hazards occurring with the severity of consequences of those hazards and failures to determine overall risk. Historically, qualitative analysis was used to determine probability and consequence. While this was a reasonable starting point for identifying the threats that posed the largest risk to a facility or portfolio of assets, advances in science and engineering now allow for quantitative and probabilistic analysis, in which hard numbers can be put to probabilities and consequences. This allows for more granular comparison of risk across assets and across mechanisms, which better informs both short term decision making and long-term decisions.

Risk analysis is especially useful and important in the utilities industry as society relies upon electricity, water, and gas to function properly. Utilities can own millions of assets of varying types, which are subjected to a variety of hazards. Due to the large number of assets, there has been impetus to develop risk analysis models for assets and hazards that pose a threat to service disruption and potentially loss of life, for example, if an asset failure ignites a wildfire. This paper discusses numerous engineering first-principles-based electric utility risk models that have been used to identify specific assets that are vulnerable to both natural hazards and everyday wear and tear, to inform repair, replacement, and inspection frequency decisions. Development of many of these models has been inspired by real utility asset failures that have led to significant negative consequences.

Wind on Poles and Towers

One of the most in-demand quantitative and probabilistic risk models for electric utilities is for wood poles and steel transmission structures subjected to wind loading. This model is useful for understanding utility risk because failure of these assets can lead to ignitions and subsequent wildfires, which are more likely to spread in the same high winds that are likely to cause failure of these overhead structures in the first place. Much research has been conducted to develop fragility curves for wood poles and steel structures subjected to wind loading (e.g., Darestani and Shafieezadeh, 2019). However, there is

significant uncertainty regarding the exact components that are installed, the connecting conductors and underbuild, and the condition of the assets. In the absence of such information, for utility scale asset-specific assessment, generic fragility curves can be calculated based on the failure rate across the system and assumptions about the wind hazard and dispersion. Asset-specific adjustments to fragility curve parameters are then made based on observational data, which for wood poles usually consists of the effective circumference or remaining strength of the pole. Bayesian updating can be applied to further refine the fragility curves based on the experiences of a specific utility. McDonald et al., 2025 provides more details on system-wide analysis of poles and towers subjected to wind loading.

Aeolian Vibration

Overhead conductors are typically damaged by extreme winds causing overload, buffeting leading to wear (turbulence, discussed below), or steady (laminar) wind leading to aeolian vibrations. Steady winds up to around 10 m/s are shed alternately, causing the conductor to vibrate up and down. This does not induce significant stresses, however, there are a significant number of cycles as conductors can be self-excited in steady winds. With enough cycles, fretting fatigue occurs, the wires rub together and general flaws. Eventually, a large gust will fail the conductor. Aeolian vibration induced failures can be reduced by limiting the tension in the line or by deploying torsional tuned mass dampers. The probabilistic risk of aeolian vibration-induced failure can be calculated by (1) calculating vortex shedding frequency and wavelength; (2) calculating the deflection amplitude corresponding to each wind speed by energy balance; (3) Calculating the stress and counting the number of cycles at stress corresponding to each deflection amplitude; (4) Estimating the number of cycles to failure of the conductor for a given stress amplitude; (5) calculating the percentage of the conductor life that has been used via Miner's rule. This type of procedure is useful for estimating the design life of a conductor and determining conductors most vulnerable to aeolian vibration-induced failures, to prioritize for mitigation.

Vegetation Risk

Vegetation can pose a significant risk to overhead electric utilities as tree falls and vegetation debris can contact conductors, leading to outages and in some cases arcing. Vegetation provides a dry fuel source which, when arcing occurs, can lead to ignition and wildfires. Utilities spend significant effort on vegetation management programs to clear vegetation around transmission lines, removing both the dry fuel source and reducing the likelihood of contacts. Distribution systems pose a greater challenge, however, because of the sheer quantity of distribution lines and the close proximity to population. Where height, species, and location of trees relative to overhead lines are mapped, the probability of trees falling on the lines can be calculated using tree fragility curves (FEMA 2021) and wind hazard curves informed by wind roses indicating the frequency that wind blows in each direction (as not all tree falls near a conductor will result in a strike on the conductor or its supporting structure). Another phenomenon that can lead to ignitions is debris bridging between conductor phases, leading to arcing, melting conductor, and potentially ignition if there is a dry fuel source below the line. The authors have developed and implemented a probabilistic aerodynamic model to estimate the risk of debris bridging, with the following procedure: (1) estimate the location and boundaries of debris fields around a conductor span; (2) estimate the quantity, size, and weight distribution of debris particles in each debris field; (3) for each debris particle, determine the likelihood that it will become airborne for a given wind speed; (4) for airborne debris particles, determine the probabilistic trajectory of the debris; (5) determine the likelihood that the debris will land on a nearby conductor; (6) integrate the uplift and trajectory probabilistic distributions with the wind hazard curves for each direction to determine rates of debris bridging. This procedure makes use of the total probability theorem and the principles behind the PEER framework for performance-based engineering. These types of analysis of vegetation risk are useful for both evaluating present risk and for evaluating the effectiveness of treatment options, such as implementing covered (polymer-coated) conductors to reduce the likelihood of arcing.

Conductor Clashing

In high winds, adjacent conductors supported on the same structures may come unacceptably close or physically slap somewhere along the span, causing arcing between phases, which poses an ignition risk. The probability of conductor clashing is site- and asset-specific and depends on variables including the conductor type, span length, sag-tension parameters for each phase, surrounding terrain, and wind hazard. We developed a framework to model the probability of conductor clashing using a frequency-domain analysis of adjacent conductors – modeled as single-degree-of-freedom oscillators – subject to gusting wind. Winds are synthetically generated to match an empirical frequency spectrum considering peak gust speed and turbulence due to ground roughness. Using the peaks-over-threshold method applied to the simulated wind response of the conductors, we can calculate the probability of conductor clashing in high winds.

Mechanical Wear

Abrasive wear – that is, the rubbing of faying surfaces under compressive normal force and the resulting loss of material – occurs chiefly due to large deflections when relatively light, unbraced components of overhead power lines are buffeted in turbulent wind. Turbulence results from the roughness of the ground interrupting laminar wind flow and is therefore site-specific, varying considerably for sites in wooded hills and in open areas. Wear can lead to the degradation and eventual failure of connections, which presents an ignition risk. We developed a site-specific model for the depth of wear at a connection by combining a single-degree-of-freedom structural model derived from a pendulum component with a wind model. We assume the dynamic pendulum displacements are Gaussian and define the up-crossing frequency as a Rayleigh distribution of pendulum swing amplitude. The total length of rubbing at the connection of interest is calculated by counting the number of swings above a given amplitude over the lifetime (age) of the component and then scaling the total distance traveled by the pendulum to get the distance rubbed at the top of the pendulum. The volume of wear (i.e., the volume of material lost) can be determined using Archard's Law, and the depth of wear obtained by dividing the volume of wear by the area of the faying surface. This procedure is repeated for every cell of a wind rose to obtain the wear at the connection considering wind of every speed and bearing, which can then be summed.

Arc Faulting

Utility rights-of-way may include poles or towers supporting overhead conductors as well as underground pipelines or cables. The potential for arcs to occur between the base of the electric structures and the buried utility lines is well established. In 1994, the Canadian Electrical Association (CEA) noted that the combination of reduced availability of land for utility corridors and increasing transmission line voltages had “increased the potential for deleterious interactions to occur between electrical and pipeline facilities.”

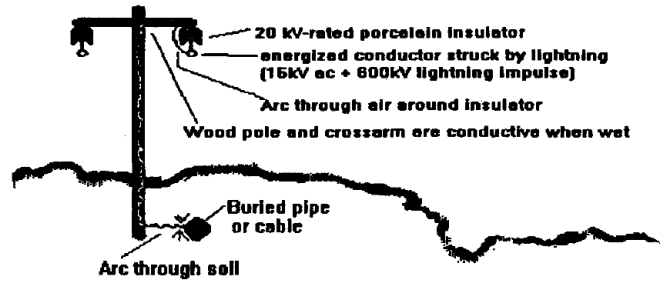


Figure 1. Overview figure of arcing between the base of a transmission or distribution structure and a nearby buried pipeline due to lightning striking an overhead conductor. From Kinsler and Hmurcik (1999).

Subsoil arcs may be initiated by lightning strikes to or by faults on an overhead conductor. A lightning strike to an overhead conductor can cause flashover of the line insulator and create an ionized path across the insulator, down the support structure, and into the soil to the nearest buried ground, which may be a nearby buried pipeline (Kinsler and Hmurcik, 1999). If current from the overhead conductor follows the ionized path established by the lightning strike, it may terminate on the buried pipeline in the form of an arc, causing damage to the pipe wall and coating (Kinsler and Hmurcik, 1999). Damage to the pipe wall may be immediately severe, e.g., puncture, or compromise the long-term integrity of the pipe. Figure 1 provides a diagrammatic overview of lightning-initiated arcs. Faults on the power system itself can also result in subsoil arcs. Faults may result from the breakdown or failure of line insulators and the breaking of overhead conductors, among other causes (CEA, 1994).

We developed a probabilistic, site-specific risk model of subsoil arcing that comprises two stages: the first stage captures the rate at which sustained arcs occur, and the second stage captures the rates of possible consequences of sustained arcs, namely damage to the pipeline coating and pipe leaks. As shown in Equation (4), rates of possible consequences are calculated separately: for lightning-initiated arcs ($\lambda_{consequence-lightning}$) using Equation (5) and for fault-initiated arcs ($\lambda_{consequence-fault}$) using Equation (6). The rate of lightning-initiated arcs is a function of the probability distribution of the lightning current given a strike, $p(I_{lightning}|lightning\ strike)$; the probability that the lightning-critical distance exceeds the actual separation distance between the base of the overhead structure and the nearby buried pipeline given a lightning current, $p(D_{lightning-critical} > D_{actual}|I_{lightning})$; and the probability of the consequence of interest given that a sustained arc occurs, $p(consequence|arc)$. Data required to compute the rate of arcs and their consequences at a site include lightning strike properties (such as first-stroke lightning current and the rate of lightning strikes), fault properties (such as fault current and clearing time), soil properties (such as its resistivity), and pipeline properties (such as its wall thickness and the dielectric strength of its coating material).

$$\lambda_{consequence} = \lambda_{consequence-lightning} + \lambda_{consequence-fault} \quad (1)$$

$$\lambda_{consequence-lightning} = \lambda_{lightning\ strike} \int_0^{200kA} p(consequence|arc)p(D_{lightning-critical} > D_{actual}|I_{lightning})p(I_{lightning}|lightning\ strike)dI_{lightning} \quad (2)$$

$$\lambda_{consequence-fault} = \lambda_{fault} \int_0^{\infty} p(consequence|arc)p(D_{critical} > D_{actual}|I_{fault})p(I_{fault})dI_{fault} \quad (3)$$

Avian Electrocution

Collisions with power lines and electrocutions on overhead power equipment are estimated to result in millions of avian fatalities annually (APLIC, 2006). In addition to the threat posed to bird populations, these avian fatalities carry with them the risk of ignitions that could lead to wildfires (Barnes et al., 2022). Only a small percentage of avian electrocutions are thought to result in ignitions in general – however, the risk of ignition varies widely by region (Barnes et al., 2022; Eccleston et al., 2023). Because dry feathers provide insulation, most avian electrocutions occur when the conductive fleshy parts of a bird simultaneously contact two energized conductors or a conductor and a ground (APLIC, 2006). We identified five geometric cases for avian electrocution, some of which include multiple sub-cases: (1) bridging adjacent conductors; (2) bridging between an overhead suspension insulator and a metal-grounded crossarm (**Figure 2**); (3) bridging conductors while perched on a conductor; (4) bridging between pole-top equipment and a conductor; (5) in-flight collision with and electrocution by bridging two phases of a circuit. For each of these five cases, we developed a model to calculate the probability of electrocution, p_f , based on a bird's dimensions (based on species anatomical data) and those of the transmission system. The model accounts for uncertainty in the size of a bird and its position on the overhead equipment. We applied this model for more than twenty species known to be at risk in a particular utility's service territory and quantified how different mitigation measures – namely, changes to the horizontal and vertical phase spacing of conductors – affected each species' probability of electrocution for each geometric case. The results were used to assess the effectiveness of the mitigation measures under consideration. Future improvements to this model could incorporate other factors known to influence a bird's risk of electrocution, including its age, use of poles (e.g., for hunting, resting, roosting, and nesting), habitat, availability of prey, seasonal changes in behavior, and courtship and territorial behavior (APLIC, 2006).

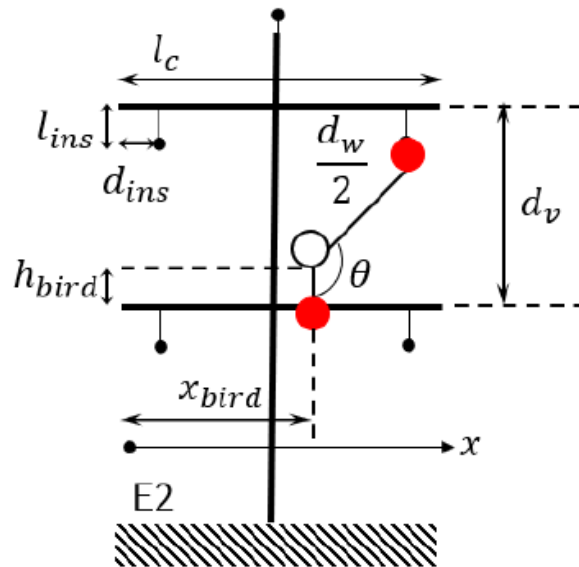


Figure 2. Schematic of one mechanism of electrocution for a bird perched on a metal-grounded crossarm. Red circles indicate where the bird makes contact with the crossarm and with the overhead suspension insulator.

Conductor Creep

Conductor creep – the non-recoverable (also called plastic or permanent) deformation that occurs in a material under load – results in elongation, decreased tension, and increased sag. Strand settlement and deformation that occur during initial mechanical loading, lifetime mechanical loading from self-weight as well as intense short-term loading (e.g., from ice accretion or high wind loads), and elevated temperatures

can all cause permanent elongation, often lumped into the single term *creep*. Creep caused by elevated temperatures is of particular interest because of climate change – and associated higher ambient air temperatures, which will result in higher conductor temperatures – and Dynamic Line Ratings, which will lead to periods of greater electrical loading and associated increases in conductor temperature. If not accounted for in sag-tension calculations, elevated-temperature creep could result in unanticipated sag that increases the risk of clearance violations. Creep predictor equations at normal and elevated temperatures were developed by Harvey and Larson (1972) for aluminum conductors and depend on the conductor type, stress, time spent at that stress, and the conductor temperature. We developed a probabilistic framework based on CIGRE (1981) for estimating the creep and sag in a conductor given historical data on electrical and environmental (i.e., wind and snow) loading. The results of the model can be used to identify locations with the greatest predicted creep and sag; to identify locations with the greatest unanticipated sag.

Contamination-Induced Insulator Flashover

Over the course of its decades-long service life, an insulator will be exposed to airborne pollution, some of which will settle on its surface. When a heavy fog or light rain occurs, it may put the salts in that layer of accumulated deposition on the insulator’s surface into solution, forming a conductive pathway and potentially leading to the formation of dry bands, dry band arcing, and eventually flashover. Insulators in coastal regions may be more prone to flashover because of the higher salt content in the accumulated deposition, as well as the potential for salt-fog. Flashed-over insulators require replacement, and sparks from insulator flashover are known to cause ignitions of poles and proximate vegetation. Heavier precipitation provides a degree of natural protection against flashover by washing away accumulated deposition. Manual insulator washing programs also protect against flashover but are typically limited in scope.

To help inform asset management decisions for utilities with large service territories and insulator portfolios, we developed a probabilistic, site-specific model for predicting the rate of contamination-induced insulator flashover, λ_f , which is calculated by integrating the probability of flashover given a deposition amount, $p(f|D)$, with a distribution of possible distribution amounts, $p(D)$, and the rate of critical moisture events, λ_m , as shown in Equation (4) (Bhattacharjee et al., 2024). **Figure 3** shows an examples of fragility curves – that is, examples of $p(f|D)$ – for polymer and ceramic or glass insulators at a voltage stress of 53.1 kV/m.

$$\lambda_f = \lambda_m \int_0^{\infty} p(f|D) \cdot p(D) dD \quad (4)$$

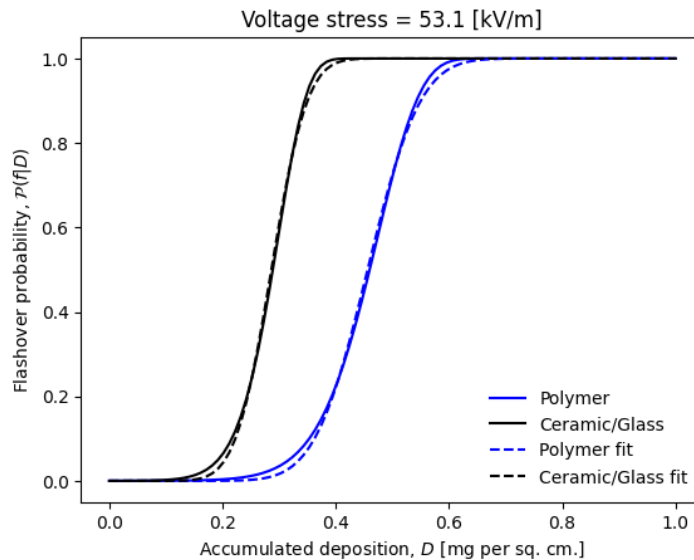


Figure 3. A fragility curve that maps the amount of accumulated deposition on an insulator's surface to its probability of flashover at a voltage stress of 53.1 kV/m.

Earthquake-Induced Landslide. Overhead utility structures are not typically vulnerable to inertial forces caused by earthquakes because they are lightweight relative to their strength. However, they can be vulnerable to ground movement. Probabilistic landslide analysis can be useful for evaluating the hazard posed to individual steel transmission towers or wood poles. For steel transmission towers, finite element models have been used to determine fragility curves for differential tower leg displacements that results in different damage states (e.g., secondary member buckling, primary member buckling, collapse). For wood poles, post-earthquake lean is of concern because of standards that require replacement of wood poles if the lean exceeds certain thresholds. Jampole and Kenawy proposed a model for calculating the probability of lean exceeding a threshold based on the pole and conductor properties and configuration, integrated with the likelihood of landslides of different displacements computed using techniques from Rathje and Saygili (2009).

Data Needs

The risk models described in this paper require site- and asset-specific data to compute asset-specific probabilities of failure. Some of this input information may be available from the asset owner, such as an asset's age, configuration, material properties, typical loading, and recent inspection data, which can inform assessments of the asset's present condition. Some environmental model inputs – such as historical temperature, air quality, rain, fog, wind, and lightning data – may be publicly available at varying spatial resolutions, and their values at the sites under consideration in the risk assessment inferred or imputed from the available data. Other environmental model inputs, such as soil properties, may be more difficult to obtain without site visits to collect the needed data. Risk models may be quite data-intensive, and depending on budget or schedule constraints, making judicious assumptions based on good engineering judgment may be necessary. The model outputs, however, will only be as reliable as the input data used – that is, garbage in, garbage out. Evaluating the predictive power of risk models typically requires that they be calibrated to historical data on outages, asset failures, and asset repairs, which are typically recorded by utilities along with causes (when known).

Summary

This paper has described several engineering first principles-based electric utility risk models that have been used to identify specific assets vulnerable to failures and to compare risk across a portfolio. The consequence of utility asset failures can be severe given unfortunate circumstance, especially related to wildfires. Understanding, quantitatively and probabilistically, the likelihood and consequence of failures is a valuable tool for utilities to select appropriate mitigations, such as implementing public safety power shutoffs, undergrounding, covered conductors, vegetation management, upgrading poles and towers, or adding system redundancy.

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AN INTEGRATED RESILIENCE-BASED DESIGN APPROACH TO ACHIEVE RESILIENCE AND SUSTAINABILITY

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Abstract

Resilience is the ability to recover quickly and restore functionality of our built environment, including buildings and infrastructure. This is critically important for the functionality of our societies and economies. There is increasing recognition that we need address multiple hazards including earthquakes and climatic events exacerbated by climate change, in an integrated manner. The increasing concern about climate change requires us to ensure that we do take into consideration the carbon emissions associated with our design, implementation and functionality, on the whole of life basis, so that resilience can be achieved both sustainably and cost-effectively.

Economic, sustainable and functional design can be better achieved by considering resilience in both dimensions of functionality and time for recovery. Often, we have focused mostly on low damage, and not enough on functionality or recovery. The achievement of resilience and sustainability requires close collaboration of professionals across multiple disciplines (including planning, architecture, structural, geotechnical and mechanical-electrical engineering), from the early stages of project development.

A resilience and sustainability-based design philosophy developed through a combination of research and practice over the past two decades, has been applied on a range of projects across multiple sectors including buildings, bridges, water and transport systems. The paper will present selected case studies, to illustrate how applying this principle has helped achieve resilience in a cost-effective manner, and often at a reduced cost.

Resilience and sustainability can be achieved cost effectively but requires focus by multiple disciplines from an early stage. Resilience does not need to cost more, financially or in terms of carbon!

Introduction

We live in challenging times, highlighted by a range of recent perils in New Zealand such as the 2010-2011 Canterbury earthquake sequence, the 2016 Kaikoura earthquake, the 2022 Nelson-Marlborough storm, 2023 Auckland Anniversary storm and the 2023 Cyclone Gabrielle, not to mention the Covid-19 pandemic. Similar perils have been faced around the world. These events have exposed the vulnerability of our society but have provided valuable insights to understand the resilience of our built environment. They have provided real life laboratories to learn from through observation and research (Brabhaharan 2006, Mason and Brabhaharan, 2021).

In the 2010-2011 Canterbury earthquakes, buildings (with the exception of a few which collapsed due to poor design and caused loss of life), were severely damaged and did not provide the functionality required for society. Many had to be demolished or undergo repairs over a long period of time. In the 2016 Kaikoura earthquake, the key arterial highway and main line railway serving the South Island were closed for a very long period of 10-15 months, leading to significant detours around lower capacity roads and severe disruption to the flow of goods and services. The severe storm events in the past few years caused damage to residential properties as well and critical infrastructure.

The detours combined with the rebuilding of buildings and infrastructure, not only led to disruption and socio-economic consequences to society, but also led to significant carbon emissions due to longer vehicle travel distances and materials and energy associated with the reconstruction. The consequence of poor resilience leads to poor sustainability and climate change.

Climate change is one of the defining issues facing humanity in the 21st century. The impact of this requires us to reconsider how we plan, design and operate our built environment, including buildings, infrastructure and other systems. There is a need to consider the multiple perils we face such as earthquakes, storms and climate change in a holistic and integrated manner.

We also face challenges in financing the construction, maintenance and reconstruction of our built environment. Therefore, we also need to be cognisant of the cost of achieving resilience and sustainability. It is often assumed that achieving resilience and sustainability will be costly. Can we achieve resilience and sustainability in the face of climate change in a cost-effective manner, and therefore be affordable to society?

What is Resilience?

Definition. Resilience is a term which has often been used loosely without understanding the real meaning and context. It is not about strength or robustness, though these can contribute to resilience. In the context of our built environment:

Resilience is the ability to continue to remain functional despite ongoing change, or rapidly return to functionality in the face of sudden adverse events.

Functionality. Note the focus on functionality in the need for resilience (Brabhakaran, 2006). Our built environment essentially serves the functional needs of society, whether it be buildings to live, study, work or operate from, or the functionality of infrastructure to provide us access (road, rail, airports etc), water for use in our lives, provide energy to operate our equipment or communications to connect us.

Therefore, it is essential for us to have a focus on functionality in engineering our built environment. So why do we focus only on damage? The expectations of post-earthquake functionality from society were reinforced by the recent Resilient Buildings Project.

Metrics of Resilience. The definition of resilience and the central need for functionality can be represented using the twin metrics of level of functionality (or service) and time for recovery. This can be best illustrated using the diagram in Figure 1.

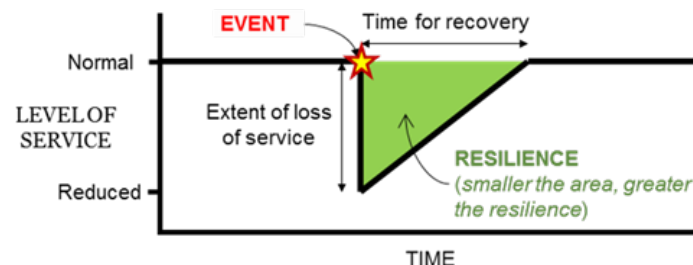


Figure 1. Resilience Concept for our Built Environment (after Brabhakaran, 2004)

Research into strategic management of natural hazard risks to roads in the early 2000s led to the development of resilience metrics for transport networks which provide access for communities – availability state (the degree of functionality) and outage state (time required for recovery to functionality) (Brabhaharan et al, 2006). These provide simple metrics that were used to assess the resilience of transport networks, and subsequently development of business cases for resilience enhancement. This was also extended to other infrastructure.

Sustainability

Definition. Sustainability from a climate change perspective for the built environment can be defined as:

Meeting the functional needs of Society, without compromising the environment, now and into the future.

It is important to consider climate change and sustainability in the planning, engineering and operation of our built environment.

Climate Change. There is substantial evidence presented by scientists that climate change is leading to increased global temperatures. Climate change predictions often focus on increased temperatures, sea level rise, and changes in rainfall and drought for different regions. What is more important for our engineering of the built environment is the extreme weather events that are being observed around the world including in New Zealand. This may be the result of climate change or long-term weather trends.

Severe Weather Events. Myre et al (2019) have modelled and compared extreme rainfall patterns in Europe over 62 years between 1951 and 2013. The amount of rainfall in extreme weather events (largest 1% or 0.03% of events) over the 30-year period 1984-2013 have been much higher compared to the previous ~30 years 1951-1984, while the average rainfall changes have been relatively modest. The observed rainfall even exceeded their own climate change predictions, see Figure 4a. Their research also shows that the frequency of these extreme weather events is also increasing, see Figure 4b. Although no systematic study has been carried out in New Zealand, such increased extreme weather events are also being observed in New Zealand. USA researchers also record the increasing costs associated with weather-related events.

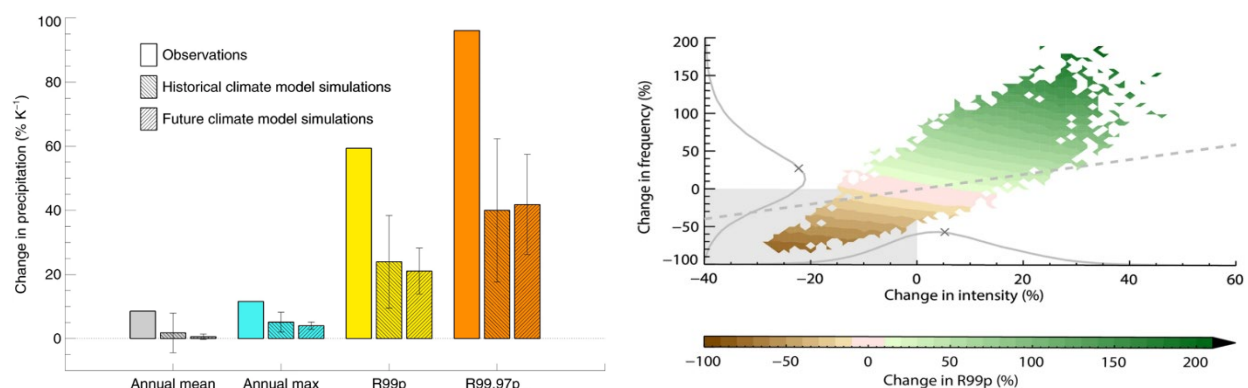


Figure 2. (a) Left: Changes in rainfall observed in Europe between 1951 and 2013 (b) Right: Changes in frequency and intensity in precipitation observed in Europe (after Myre et al 2019)

Severe Weather Impacts. The severe storm events in Canterbury (2021), Marlborough (2021-2022), Auckland (2023) and the East Coast of the North Island (2023) over the past 5 years indicate that these severe weather events are leading to flooding, landslides, high river flows and debris rafting, leading to severe impacts on our critical infrastructure, built and natural environments and society. Such impacts were reinforced by the January 2023 Auckland Anniversary Storm and February 2023 Cyclone Gabrielle.

Inter-relationship between Climate Change, Resilience and Sustainability. The intertwined nature of climate change, resilience and sustainability is illustrated in Figure 3. Climate change leads to poor resilience of our built environment, which leads to increased disruption and reconstruction and associated increased carbon emissions, which in turn exacerbates climate change and severe weather.

Other natural hazards such as earthquakes also lead to significant damage, disruption and loss of functionality of our built environment, with an associated need for reconstruction, and hence contribute to the increased carbon emissions associated with disruption and reconstruction.

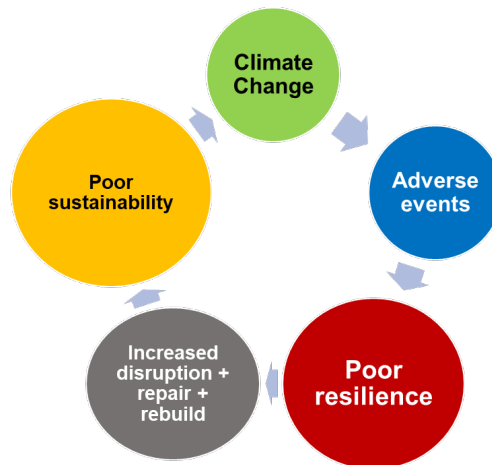


Figure 3. Intertwined Climate Change, Resilience and Sustainability, and Need for a Holistic Approach

Climate Change Actions. Our response to climate change from an engineering perspective with respect to our built environment requires a dual response:

- Resilience – we need to enhance the resilience of our built environment to mitigate and manage the severe weather impacts that is already having significant adverse effects on our built environment and society and could potentially get worse.
- Sustainability – we need to reduce carbon emissions to arrest, and hopefully reverse climate change, through reducing the embodied carbon associated with engineering of our built environment. Otherwise the climate change impacts, including on resilience, could potentially become worse.

Resilience and Sustainability in Design

Need for an Integrated Approach. Resilience and sustainability are intertwined and therefore we need to take an integrated approach in the planning, design and operation of our built environment for all the perils including earthquakes, climatic effects and climate change. Financial sustainability also requires that we need to make our designs cost effective, so that we can indeed ensure that achieving resilience and sustainability is affordable.

Design Philosophy. An appropriate design philosophy needs to be developed and adopted to achieve resilience and sustainability. Our current design approaches were developed at the time of a more stable environment in Europe, and we now do not have that stability with significant change and uncertainty as to severe weather effects from climate change, earthquake hazards and the financial environment. We need a more nuanced approach that can take into consideration these uncertainties.

Early approaches for design were based on limiting the loss of life from collapse to an acceptable level, and serviceability during normal operational conditions. The focus was on achieving a low enough probability of loss of life and limiting deformations so as not to affect use under normal live loads. This approach was essentially extended to earthquake design, with later consideration of low damage design and performance-based design. How low should low damage be? What performance is required from performance-based design? Capacity design was developed to ensure an appropriate hierarchy of failure, still focused on damage and collapse. The focus remained on limiting damage and enhancing life safety performance. While life safety is critical, we need to ensure that our structures are usable, and functionality can be restored quickly after hazard events. As noted above, the primary need from our built environment is the functionality that we derive from them. So why do we not focus on functionality and resilience?

Resilience-based Design. Resilience focuses on functionality and the ability to recover quickly, and therefore provides a useful design philosophy to adopt. Our built environment, whether it be buildings or infrastructure, are primarily there to provide society the functionality they offer. It is therefore prudent for this to be the focus in the design and operation of our built environment.

The fundamental principle in resilience-based design is to enable continued functionality or quick return to service, whether it is for infrastructure or buildings. The twin metrics of resilience – level of functionality and time required to recover functionality after an adverse event - can be used in adopting a resilience-based design approach. Such a design approach is illustrated conceptually using the resilience diagram shown in Figure 4(a). Focusing on functionality and developing the design to reduce the green triangle to the blue triangle would provide enhanced resilience, and society would enjoy the restored functionality sooner. Resilience can be enhanced by limiting the reduction in functionality in hazard events and/or enabling quick restoration of the functionality.



Figure 4. (a) Left: Enhancing Resilience through reduced reduction in functionality and rapid return to service (as indicated by blue triangle) (b) Right: Staged recovery of functionality over time

Immediate return to full functionality may not be needed in some situations, and a phased approach might be appropriate, while the rest of the socio-economy return to normality, as illustrated in Figure 4(b).

Key factors in considering a resilience-based design approach would be:

- In frequent, low impact events such as moderate storm events, a rapid return to functionality would be important, as the broader socio-economy will be functional and loss of functionality would be disruptive.

- In low frequency, high impact events, such as a large earthquake (eg. 2011 Christchurch earthquake), the ability to provide for response and survival would be important, and therefore maintenance of partial functionality and rapid return to partial operational functionality would be critical. Return to full functionality could occur over time, as the socio-economy may not require this.
- Understanding the resilience context of the asset within the overall built environment and infrastructure network and interdependencies is important, as this needs to form the basis for achieving an appropriate level of resilience.
- Consideration of the hierarchy of importance of different parts of the built environment, with:
 - critical parts of the system, and difficult-to-restore components need a greater level of resilience;
 - less critical and quickly-restorable components needing a lesser level of resilience; and
 - accepting slower return to functionality for parts that are not important for the function of society.

Sustainability in Design. Sustainability in design requires focus on reducing embodied carbon associated with construction and carbon emissions during operation, and thus limit greenhouse gases that lead to climate change.

It is also important to take into consideration the sustainability benefits from resilience-based designs. Resilience-based designs lead to improved functionality and therefore less carbon emissions associated with disruption, such as detours due to road closures, transport of water for emergency needs, and provision of alternate accommodation. As an example, the 2016 Kaikoura earthquake led to closure of the main highway and railway for 10-15 months and disruption for a further few years, leading to significant detours and associated carbon emissions, as well as emissions and embodied carbon related to a huge reconstruction effort. A more resilient built environment would need less repair or reconstruction, and consequently there would be less embodied carbon and energy associated with reconstruction. Therefore, a resilient built environment would also lead to greater sustainability, and resilience and sustainability can be achieved together.

Economics of Resilience and Sustainability. Often an assumption is made that resilience and sustainability will lead to a higher financial outlay, and that resilience will lead to higher embodied carbon. This then leads to the adoption of a low-resilience design. Does the incorporation of resilience and sustainability really lead to higher financial costs? Does resilience in design lead to higher carbon emissions and increased climate change? It would be prudent to consider these through case studies.

Case Studies of Design for Resilience

The resilience-based-design approach has been developed through practical application of this concept in design practice across several sectors. Selected case studies are presented to illustrate application of the concept in real life practice.

Case Study – Ferrymead Bridge in Christchurch. The 2010-2011 Canterbury earthquakes in Christchurch, New Zealand led to severe damage to the Ferrymead bridge predominantly due to severe liquefaction and lateral spreading resulting in the piers on the riverbanks being pushed towards the river. The Christchurch City Council decided that a resilient bridge needed to be built to provide a secure connection to the suburbs as well as provide a showcase to illustrate and provide confidence as to how Christchurch can be rebuilt to be more resilient to future earthquakes.

The initial design concept was to replace the bridge with a more robust but similar three span bridge with ground improvement to strengthen the riverbanks and also use larger piles at the riverbanks. A focus on resilience allowed the design team to explore how the twin metrics of resilience – functionality and time for recovery, can be considered to achieve a more resilient bridge, taking into consideration the highly liquefiable soils and potential for lateral spreading. This focus led to selection of a two-span bridge, locating the pier in the middle of the river away from the riverbanks, slender piers at the abutments that attract less lateral spread load, and the use of short land spans, see Figure 5. If the land spans are damaged or displaced due to lateral spreading, functionality (access) can be quickly restored using steel planks over these short spans and can then be readily repaired by re-aligning the land span structures. This led to the provision of a resilient connection to provide access for the community.

It should be noted how the early focus on resilience, and the approach to reduce loss of functionality, and enable quicker recovery of that functionality – the focus on the twin metrics of resilience, refer Figure 1 and Figure 4, led to the resilient design solution. In addition, the solution adopted led to a lower financial cost (because of less piers, slender pier piles and no ground improvement). Though the focus at the time wasn't on embodied carbon, the reduced concrete and avoidance of ground improvement resulted in substantially lower carbon emissions and therefore enhanced sustainability. It should be noted that the focus wasn't on strength, robustness or low damage. Fundamental to the outcome was the early integrated focus on resilience, between the structural and geotechnical engineers and the Council.

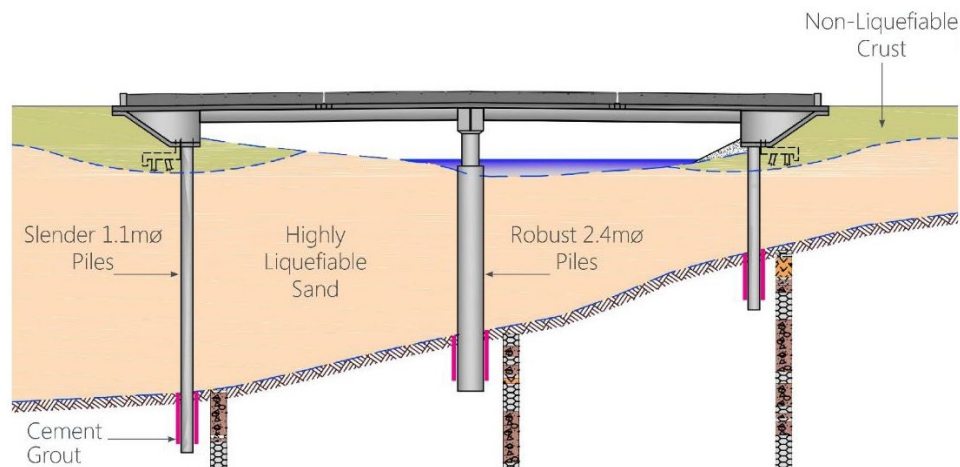


Figure 5. Ferrymead Bridge Christchurch – Long Section showing Design Concept and Geology

Case Study – Transmission Gully Expressway. The 27 km Transmission Gully Expressway project north of Wellington city, was conceived to provide additional capacity and enhance traffic safety given that the pre-existing coastal highway passed through residential settlements and had no space to expand. Given the poor resilience of access into Wellington, this also would provide an alternate route into the region, resilience was adopted, for the first time, as a key project objective.

The original design concept involved the expressway crossing the major active Ohariu Fault a few times on bridges and viaducts and crossing existing hillsides with large landslides. The focus on resilience, with consideration of the twin metrics of functionality and time for restoration of access, enabled the alignment and design form to be changed by:

- Locating the alignment on the opposite side of the valley to only cross the active fault once.

- Adopting an embankment at the active fault crossing to accommodate displacement of the fault rupture and enable restoration of functionality quickly by earthmoving to form ramps (both embankments and bridges will be damaged, but functionality can be restored quickly with embankments whereas replacement of the viaducts will take years to reconstruct).
- Avoiding half bridges adjacent to steep slopes where they can be damaged by landslides, and instead locating the expressway on reinforced soil embankments that can be restored quickly.
- Adopting natural benched 45-degree rock cut slope profiles that are stable in large earthquakes but accepting small wedge failures that can be captured on benches or quickly cleared to restore access.

The focus on resilience – functionality and time for recovery – led to a significantly more resilient design, where access into Wellington can be restored quickly. The design also led to substantially lower construction costs (by 30% on the \$ 1 billion scheme). Though decarbonisation wasn't the focus at the time, the resilience-based design led to substantially lower embodied carbon (by adopting nature-based solutions - cut slopes and embankments instead of bridges, viaducts, anchored and shotcreted slopes).

Conclusions

Our built environment is there for society to benefit from the functionality they provide. It is prudent that resilience requires focus on functionality and quick recovery of use. Sustainability is important for the ongoing functionality of society, the built environment and of future generations. Climate change is already leading to more frequent and severe extreme climatic events, and climate change consideration requires the twin focus on decarbonisation to arrest future changes as well as resilience to climatic events. Sustainability and resilience are intertwined and interdependent and require an integrated focus on both.

We live in an era of instability and uncertainty, and this requires us to review our traditional design approaches. Early and integrated focus on functionality through a resilience-based design approach with consideration of sustainability in design are important for society to achieve resilience and sustainability. This requires focus on the twin metrics of functionality and recovery rather than damage. Nature based solutions can be considered to provide both resilience and sustainability.

Resilience and sustainability do not necessarily cost more and may in fact cost less as demonstrated through case studies. However, their achievement requires an integrated focus by planners, designers and owners from an early stage of development, where the location, building form and design concepts can be influenced to infuse resilience and sustainability, rather than as an after-thought.

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SEISMIC DESIGN OF LOW-RISE BUILDINGS IN NEW ZEALAND

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Abstract

The largest proportion of new building stock in New Zealand is low rise construction. The current New Zealand Standards do not require capacity design for most of these buildings and do not prescribe acceptable structural systems or set limits around regularity and redundancy. The existing design framework tends to cater for larger, more complex structures and the requirements around low rise systems are prone to misinterpretation at times, leading to poor outcomes. A summary of the design status quo for low rise buildings is included, with a comparison of NZ seismic requirements with American Standards. The paper then outlines the provisions being developed by the SRWG with an aim to make robust seismic design simple and achievable for common low-rise structures.

Introduction

The majority of buildings in New Zealand are low rise structures. Low rise refers to a single or two story building, which may include partial roof space or basement. At present in New Zealand (NZ), low-rise buildings are generally not required to be designed using capacity design principles. The Seismic Risk Working Group (SRWG) is reviewing the seismic design provisions for low rise buildings.



Figure 1. Typical commercial and hospital building in New Zealand.

Existing Design Framework and Outcomes

Current NZS1170.5 and Draft TS1170.5. NZS1170.5 is the current Loading Standard cited in the NZ Verification Methods and used in the design of buildings. In 2022, an updated National Seismic Hazard Model (NSHM) was released, and a project was undertaken to incorporate the updated hazard into NZS1170.5. This led to the release of a draft Technical Specification, TS1170.5. TS1170.5 generally replicates the design philosophy and framework of NZS 1170.5. For the purposes of this paper and comparisons, seismic Loading Standard refers to NZS1170.5.

Current New Zealand Design Framework. The New Zealand design process requires a structural engineer to ‘adopt a ductility (μ)’. The adoption of a ductility level sets both the seismic base shear demand and the required detailing in the relevant material Standard. The ductility levels and descriptions in the Loading Standard are shown in Table 1. While the implied system behavior for each level is not clearly articulated, an ‘elastic’ ductility level equates to the level of loading to design a new building if it is not expected to be able to rely on non-linear behavior, ie ductility (μ) =1 loads. Fully ductile (μ =3 to 6) equates to the level of loading to design a new building expected to have non-linear behavior into strain hardening regions.

Table 1. Seismic Design framework Definitions in NZ seismic Loading Standard

<i>Ductility</i>	<i>Description</i>
Brittle	A brittle structure is defined as a structure with structural components that are not capable of inelastic deformation without undergoing sudden and significant loss of strength. The structural ductility factor, μ , for brittle structures shall be taken as 1.
Elastic	Not defined (Example $C_d(T) \approx S_a \times 1 \times W_t$)
Nominally ductile ($\mu=1.25$)	A structure designed and detailed in accordance with this NZS and the appropriate material standard, so that a structural ductility factor that is greater than 1 and less than or equal to 1.25 can be used to assess the ULS seismic actions. ($C_d(T) \approx S_a \times 0.8 \times W_t$)
Limited Ductile ($\mu=1.25$ to 3)	Appropriate material standard detailing provisions that will result for potential inelastic zones within members when post-elastic behavior is expected, and other zones or members where the development of inelastic behavior is suppressed. Potential inelastic zones shall be capable of sustaining the level of inelastic demand placed on them when they are subjected to ULS shaking. Zones where inelastic behavior is suppressed are to be designed, to ensure suppression of that behavior. These zones are to be detailed in such a manner that the ULS horizontal deformations can be sustained without significant (eg >20%) loss of vertical load carrying capacity. ($C_d(T) \approx S_a \times 0.4 \times W_t$)
Fully Ductile ($\mu=3$ to 6)	A structure designed and detailed in accordance with this NZS and the appropriate material standard, so that it is appropriate to use a structural ductility factor greater than 1.25 and less than or equal to 6 to assess the ULS seismic actions. ($C_d(T) \approx S_a \times 0.27 \times W_t$)

* $C_d(T)$ = horizontal design action coefficient (seismic base shear) in NZS1170.5. Short period ($T=0.4s$) is adopted for this example. W_t = seismic weight of the structure. S_a =spectral acceleration.

Hazard and Life Safety. The New Zealand Building Code (NZBC) has a performance objective for life safety. The Loadings Standard refers to Annual Individual Fatality Risk (AIFR) as the risk metric related to this life safety performance objective, and Ultimate Limit State (ULS) as the design point. It is considered that application of the ULS requirements (in the Loading Standard and associated material Standards) in combination with an Annual Probability of Exceedance (APoE) of 1/500 earthquake shaking will lead to a risk of collapse and AIFR that will be tolerable. The tolerable range for AIFR is 10^{-5} to 10^{-6} . Furthermore, to ensure a tolerable risk of collapse, there should be a reasonable margin between ULS actions and actions that are associated with structural failure through a loss of strength or stability. (TS1170.5 Commentary). The event associated with structural failure through a loss of strength or stability can be referred to as the Maximum Credible Earthquake (MCE).

Ductility Level for Low Rise Structures. For low rise buildings in New Zealand, practitioner engineers predominantly adopt ‘nominally ductile, $\mu=1.25$ ’. This is chosen because it provides the simplest path through seismic design requirements for practitioners. As seen in Table 1 above, the derivation of the horizontal design action for ‘nominally ductile’ implies that some nonlinear deformation capacity is expected, and a reduction in seismic base shear demand is permitted. For low rise systems some of the material Standards do not explicitly require hierarchical design necessary to ensure ductile behavior is achieved.

Limitations for Nominally Ductile structures. It is useful to compare some common nominally ductile lateral force resisting systems with international equivalents. A comparison has been completed between the NZ limitations on use and detailing requirements for some common systems, with their equivalent American counterparts. As can be seen, NZ requirements and limitations are considerably less onerous.

Table 2. Comparison of Requirements for seismic systems used in low rise – NZ vs USA

	NZ* (nominally ductile)		USA	
Systems	Limitations	Design requirements	ASCE limitations	Design req'ts
Wall systems				
Reinforced concrete shear wall	No limitation prescribed	Structures shall be proportioned to ensure that mechanisms can only form in acceptable locations, or if outside acceptable locations must be detailed such that strain limits are not exceeded.	Ordinary RC walls are not permitted in high seismic zones. Special RC wall system is allowed up to height of 160/100 feet (48/30m)	Capacity design required.
Precast concrete shear wall	No limitation prescribed	Consider the relative stiffness, strength and ductility of connections using different materials. Other requirements as for wall system.	Intermediate precast shear wall systems are allowed up to 40ft (12m), or up to 45ft (13.7m) for single story warehouse buildings.	In connections between wall panels, or between wall panels & the foundation, yielding shall be restricted to steel elements or reinforcement'
Plywood shear wall	No limitation prescribed	No capacity design requirements in cited Standard. New Standard (not cited) requires capacity design.	Walls sheathed with wood structural panels are allowed up to 65ft (19m)	Capacity design required. SDPWS provides prescriptive requirements for wall systems.
Frames				
Steel concentrically braced frame (X – tension only)	Limited to 8 stories for nominally ductile (category 3)	Category 4 CBF no capacity design required. Connections potentially weakest point, loads capped at elastic.	Tension only must be Ordinary CBF. Permitted in high seismic zones, height limit 10.7m (35ft), or 18.3m (60ft) with roof weight under 1kPa.	Ordinary CBF requires capacity design. Connections to be designed for brace probable capacity.
Steel eccentrically braced frame	No limitation prescribed.	Capacity design required.	Special EBF permitted to height of 160/100 feet (48/30m)	Capacity design required
Moment Resisting Frames				
Steel moment (portal) frame	Maximum of 4 stories.	No capacity design requirements.	Ordinary frames permitted in high seismic zones to height of 20m if weight under 1kPa, or 10m if weight is under 1.68kPa.	Ordinary moment frame requires connection design for precluding connection failure (i.e. capacity protected design)
Cantilever Systems				
Cantilever out-of-plane concrete wall	Single story only.	Capacity design required.	Not allowed	
Other				

Steel roof truss diaphragm	No limitation prescribed.	No capacity design requirements.	Requirements provided in AISC-341.	Design to overstrength of vertical system.
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*New Zealand seismicity is equal to areas of high seismicity in America i.e. NZ is covered by SDC D, E & F per ASCE7-22. This table assumes ‘nominally ductile’ design approach for the listed systems.

Outcomes using current provisions. The current design framework presents challenges for practitioners in NZ, with interpretations of requirements sometimes leading to structures with potential sudden failure mechanisms. While the competence of individual engineers plays a part in this outcome, it is necessary to reflect on the critical role that Standards play. Some of the challenges practitioners face when designing low rise buildings are as follows.

- The intent of the Standards around defining and protecting critical load paths of low-rise structures under lateral loading can be difficult to extract and therefore sometimes misinterpreted.
- Many aspects of NZ Standards tend to cater for more complex and multi-story structures.
- The material Standards do not provide information on acceptable mechanisms for some low-rise systems, leaving practitioners to solve this for each design. Without this type of information, practitioners default to ‘adopt a ductility of 1.25’, and no capacity design.
- There are no listed acceptable lateral force resisting systems. While in theory this allows innovation in design, in practice it leads to poorly configured and/or atypical structural systems.
- Standards in New Zealand are less prescriptive than American Standards. The prescriptive American standards provide less opportunity for misinterpretation of requirements.
- No pre-qualified connections are provided for lateral force resisting systems. Multi story buildings tend to use proprietary energy dissipating devices or specific detailing. These solutions generally have testing of the detailing or devices completed on a project-by-project basis, which is cost prohibitive for smaller low-rise projects, and are considered an Alternative Solution.

Issues with current framework

Risk outcome of using current provisions. As part of the SRWG process, a study has been completed reviewing the relative risk of buildings designed without capacity design, and which therefore may have little or no post yield capacity. To complete this review, fragility curves were developed for buildings which have potential sudden failure mechanism such as those that may be found in structures detailed to the current requirements for nominally ductile. The fragilities curves are defined by a collapse margin ratio, CMR (median collapse intensity normalized by the design intensity), and lognormal standard deviation of β , representing the record-to-record variability in collapse performance. Buildings designed without capacity design principles were represented by lower CMR value and lower β value, as is consistent with the FEMA P695 characterization of record-to-record variability. This review indicates that these buildings present an AIFR of 4.4×10^{-5} which exceeds the target risk metric of 10^{-6} to 10^{-5} and therefore does not meet the life safety objectives of the NZBC.

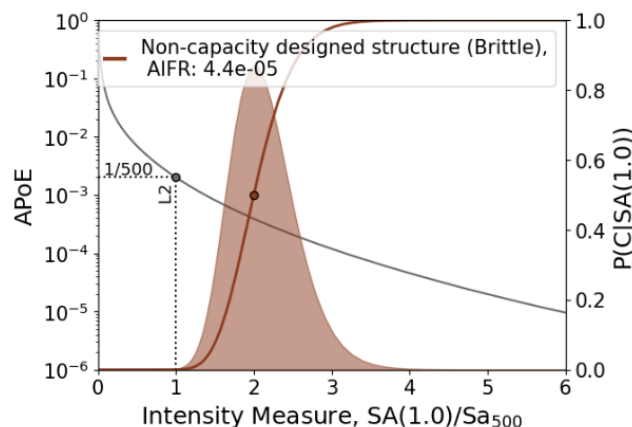


Figure 2. Fragility curves for structures not detailed for capacity design

2022 National Seismic Hazard Model. The 2022 NSHM has been incorporated in the draft Technical Specification TS1170.5. This indicates a trend for increased seismic loads across NZ. An example for Masterton (a location in the north island of NZ), is shown below, indicating a 75% increase in hazard for short period structures at this location.

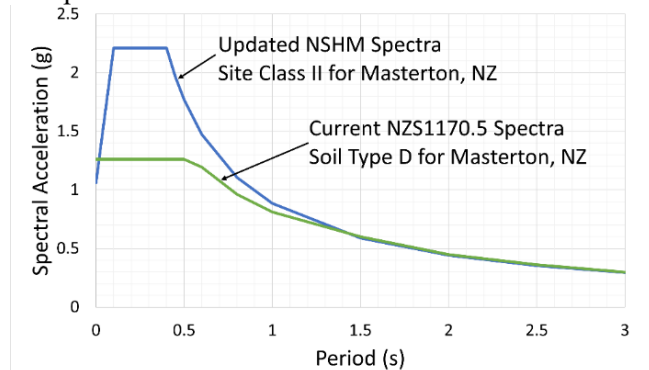


Figure 3. Masterton current spectra in NZS1170.5 and proposed spectra in TS1170.5

Rising to the Challenge

The existing design framework and philosophy, and recent update to the NSHM presents two challenges for engineers in New Zealand. One is that our current design philosophy is potentially leading to an unacceptable AIFR, and the second is that we also need to accommodate increased forces from the 2022 NSHM. There are two options we can consider to improve the performance of these buildings and meet the life safety performance objectives.

- Option a – increase the force we need to design these types of buildings for, i.e. make it stronger.
- Option b – improve the performance of structures by applying principles of robust seismic design.

Increase strength requirement – one solution? Assuming the current design framework and requirements remain unchanged, then one option is to simply increase the force that these buildings need to be designed for so that they meet the AIFR performance metric. While this would be simple for practitioners to implement, it would likely lead to a disproportionate increase in build cost and retain the structural vulnerabilities and potentially poor interpretations that the current framework suffers from. To complete this review, the same fragility curves were adopted as developed above, however a shift to increase force was introduced to gauge when the AIFR would meet the life safety targets. An increase of 1.7 has been estimated as required.

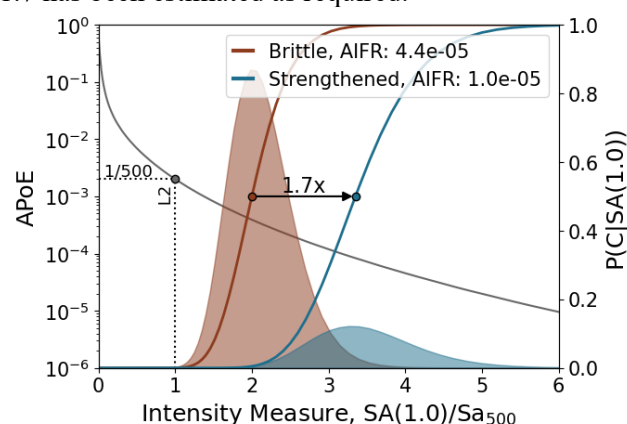


Figure 4. Fragility curves -strength factor increase for structures not detailed for capacity design

Increase robustness – better solution? Another solution is to redefine the design framework to allow for the benefit of system robustness provided by good design principles. For many low-rise structures, a blanket requirement for capacity design would be difficult to implement, as full capacity design can be impractical and would likely result in uneconomic structures and foundations. We can however design with detailing to ensure the following behavior;

- The structural system is a prescribed system known to have reliable performance
- The structure does not have weak points, with critical load paths protected
- The structural system has capacity for some load redistribution and/or post-yield capacity
- The system is well configured, meeting regularity and redundancy limits
- The system has a robust tied together foundation system with appropriate geotechnical input

These principles have some similarity to the requirements articulated for an Ordinary Steel Moment frame (OMF) in AISC-341. AISC-341 Commentary chapter E1-2 articulates the following for an OMF. ‘Despite the lack of a quantitative inelastic deformation requirement, the overall intent of OMF design is to avoid nonductile behavior in its response to the lateral load. To provide for minimal inelastic deformation capacity – that is, to avoid nonductile behavior – the general intent of the OMF design provisions is that connection failure should not be the first significant inelastic event in the response of the frame to earthquake loading.’ Designing buildings with these principles increases robustness and facilitates a shift in the fragility of the building stock. To complete this review, fragility curves were developed reflective of a structure with inelastic deformation capacity. This structural system provides increased record-to-record variability, β , and increased CMR. These structures would meet the AIFR targets.

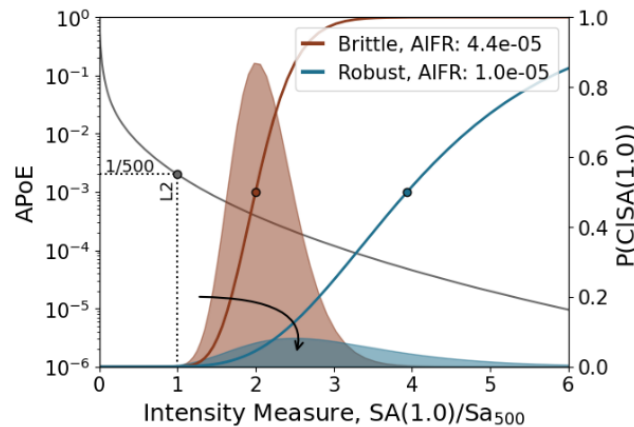


Figure 5. Fragility curves showing resulting AIFR for structural systems with robust detailing

Seismic Risk Working Group

The Seismic Risk Working Group was commissioned by the New Zealand government in 2020. They are tasked with considering the scope of changes likely needed to the building regulatory system due to the National Seismic Hazard Model and updating design and analysis approaches. The main goal of the SRWG is to develop a design approach which provides better outcomes for society from our built environment in earthquakes, recognizing cost and sustainability. (Elwood et al, 2023).

SRWG proposed systems. The SRWG has developed three main groupings of lateral force resisting systems, intended to replace the current ductility level definition.

Table 3. SRWG Proposed Seismic Design Types

Type	Description*
Type 1	A structural system in which inelastic deformation capacity is relied upon to achieve life safety performance objectives
Type 2	A structural system in which inelastic deformation capacity is assumed but need not be quantified in order to achieve the life safety performance objective.
Type 3	A structural system in which inelastic deformation is not relied upon and strength is primarily used to satisfy the life safety performance objective.

*These are working descriptions and are still being developed by the SRWG.

The inclusion of ‘Type 2 systems’ is a system that is intended to capture the guiding principles outlined above for a robust low rise structure.

Type 2 systems. Type 2 systems would encapsulate common low rise structural systems. The framework around Type 2 would lead to a system which is designed to robust principles, with capacity design, but would not require engineers to explicitly calculate and quantify nonlinear displacement capacity. Design base shear would be able to be reduced appropriately due to the use of prescribed well performing systems and limited non-linear behavior. The following outlines how Type 2 systems may be developed into design provisions.

- For these simple and common systems, verification of non-linear capacity should be completed by material Standards, which would then provide rules for design of the structures.
- Force reduction factor should be provided, allowing for non-linear capacity and the additional capacity provided by safety factors, probable material strengths, and the use of set member sizes.
- Type 2 systems would be limited to listed structural systems only, with prescriptive requirements.
- Type 2 systems should include pre-qualified joints for common systems. It may be possible to adopt the AISC-358 framework for example, with some modification for New Zealand.
- Regularity and redundancy requirements would be provided.

The proposed provisions would have the benefit of reducing design forces and making robust design simple and accessible for practitioners. It would also reduce misinterpretation of requirements.

Example for Common Structural System

The following will reflect on a typical structure designed in New Zealand. For this example, a simplified version of a very common structural system has been chosen.

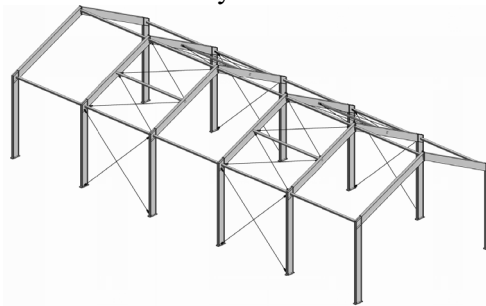


Figure 6. 3d of example building – simplified typical low rise

Current Framework. The lateral force resisting system (LFRS) in the transverse direction is a steel moment frame, with tension only concentrically braced frame (CBF) system longitudinally. The roof plane is braced via a horizontal truss formed with tension only bracing. The steel portal frame and CBF frames do not require capacity design, and connections can be a potential weak point, which seems contrary to the robust principles and the requirements for risk targets around MCE. The design of the steel CBF requires the calculation of multiple different seismic coefficients, which can introduce confusion and a potential point for errors. The derivation of loadings and detailing requirements for the roof plane truss bracing is not clear, and design outcomes vary based on practitioner interpretation of requirements. This example focused on a common steel system, however similar parallels can be drawn for masonry block, concrete and timber systems.

Proposed Framework. It is proposed that prescriptive tables be provided to outline acceptable Type 2 systems, with an example of these prescriptive tables below. It introduces the terminology of a ‘potential yielding element or action’, which identifies acceptable mechanisms, and the term ‘non-yielding element or action’, which outlines actions which need to be suppressed, or a component of the system which needs to be protected. A force reduction factor would also be used in the derivation of seismic base shear. It is anticipated that this would include reductions due to the ductility provided by inelastic behavior, and additional capacity provided by load and resistance factored design requirements, such as safety factors, probable material strengths, and the use of set member sizes.

Design provisions would need to be provided by the relevant material Standards. Limitations on use of each system should also be provided and could potentially include restrictions on use for ‘heavy’ construction or higher importance structures such as hospitals. Applying these prescriptive requirements would result in a system designed to protect connections, with some inelastic capacity.

Table 4. Example of Prescriptive Type 2 System table

<i>System</i>	<i>Potential Yielding Element or Action</i>	<i>Non-Yielding Element / Action</i>	<i>Reduction factor*</i>
<i>Wall systems</i>			
Precast concrete shear wall	In connections between wall panels or between wall panels and the foundations, restrict yielding to steel elements or reinforcement	Concrete failure	2 ($C_d(T)$) $\approx 0.5 \times S_a \times W_t$) **
<i>Frames</i>			
Steel concentrically braced frame (X – tension only)	Tension yielding of braces	Braced frame connections Collector beam elements Columns / Chords Hold down connections	3 ($C_d(T)$) $\approx 0.35 \times S_a \times W_t$)
<i>Moment Resisting Frames</i>			
Glulam portal frame	Nail slip of knee joint connection	Knee plates Apex joint Connections to foundations Rafter and column sections	2 ($C_d(T)$) $\approx 0.5 \times S_a \times W_t$)
Steel portal frame (Equivalent to OMF)	Plastic behavior of rafter beam section, or web panel shear. (Connection protected)	Frame connections Column connection to foundation Buckling of beams & columns	2 ($C_d(T)$) $\approx 0.5 \times S_a \times W_t$)
Steel portal frame (Equivalent to Intermediate Moment Frame)	Yielding of rafter adjacent to rafter / column knee joint or at a prequalified connection designed to allow a minimum story drift angle of 0.02 radians.	Frame connections Column connection to foundation Buckling of beams and columns	3.5 ($C_d(T)$) $\approx 0.3 \times S_a \times W_t$)
<i>Diaphragm</i>			
Roof plane truss diaphragm	Tension brace yielding	Brace connections, Buckling of struts, Truss chords	Design for o/s of vertical sys

*These are placeholder numbers added for this paper as an example only. Reduction factors for each system will need to be determined.

** S_a =spectral acceleration

Summary

Low rise structural systems are diverse and wide ranging and form the largest percentage of new building stock in New Zealand. However, under the requirements of current Standards, capacity design is not required for most of these buildings, and the interpretation of the existing design framework by some practitioners is leading to poor outcomes. The SRWG goal is to provide better outcomes for society from our built environment in earthquakes, recognizing cost and sustainability. It is important that this addresses the most common systems being used in New Zealand. Along with regularity and redundancy requirements, the adoption of Type 2 systems would promote the design of robust low-rise systems. Providing practitioners with prescriptive requirements for good performing reliable systems would simplify the design process and reduce misinterpretation. The aim is to lead designers to the right solution and make robust design simple and achievable for common structures.

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SALK INSTITUTE LABORATORY: RESILIENT DESIGN CASE STUDY

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Abstract — High Performance Laboratory

The new laboratory for the Salk Institute is an example of current best practices for a comprehensive resilient design. The concrete structure features vertically post-tensioned rocking walls, designed to be damage resistant, able to deform and re-center after a major earthquake. Architectural elements such as the skin and glazing system, are explicitly designed to accommodate expected deformations. Other elements, such as the full-building skylight, operable façade wall, architectural stairs, and the internal pedestrian bridge, have been designed to have low damage. FEMA P-58 economic loss analyses were performed to validate the performance and refine the anchorage requirements for the bracing ceiling elements and mechanical equipment.

Introduction

The new laboratory for the Salk Institute, the Joan and Irwin Jacobs Science and Technology Center, will be on axis with the iconic architectural masterpiece by Louis Khan. The design is complete, and construction is expected in 2025. Key design elements, such as the expressed architectural concrete form, and the dominant central axis, reference the Khan building (see Figure 1). The resilient design goals and the affinity for technical innovation, along with sustainability objectives, are aligned with the mission of the world class research laboratory. The salient feature of this case study is the comprehensive application of resilient design techniques. Resilient design concepts are applied to the structure, the architectural elements, and the mechanical bracing. For the structure, the rocking re-centering walls were designed using the ACI 550.7-19 (ACI, 2019) guidelines. These guidelines, though intended for precast concrete structures, were adapted for the cast-in-place construction. Site-specific ground motions were used as inputs for the nonlinear response-history analyses. The drift demands were inputs for the resilient design of the architectural elements. The FEMA P-58 economic loss modeling confirmed satisfying the resilience performance goals (FEMA, 2018).

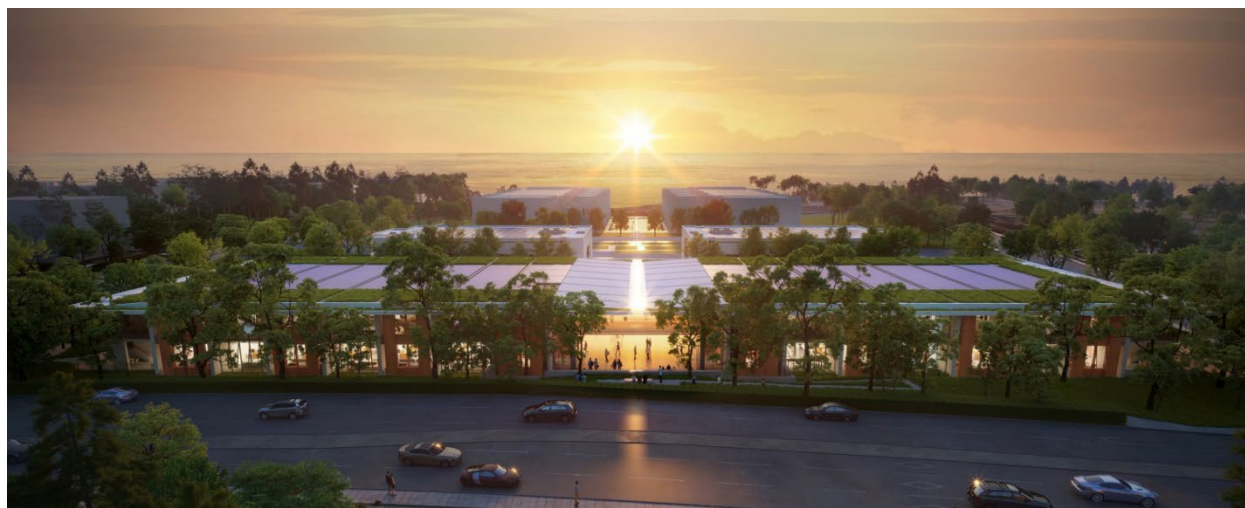


Figure 1. The Joan and Irwin Jacobs Science and Technology Center at the Salk Institute.

Structural Design and Analysis

The Structure. The building is a two-story reinforced concrete structure with a full basement that extends beyond the superstructure footprint. The above grade plan dimensions are 502 feet x 213 feet. The roof is 26 feet above grade. The roof and second floor framing consist of post-tensioned flat plate slabs. These slabs feature long spans, with a 22' x 37' column grid. To support the long spans, the slabs incorporate partial post-tensioning and void elements, such as Bubble Deck. The void forms reduce the seismic mass, stiffen the vibration response, and lower the carbon footprint of the building. The first level basement and mat foundation are mild reinforced concrete slabs. Basement walls are conventional cast-in-place reinforced concrete walls, while the shear walls are cast-in-place hybrid rocking walls, with vertical post-tensioning using threaded rods, and mild steel yielding energy dissipaters.

The above ground structure is divided into two halves, symmetric about the central axis. There is a 70-foot-long internal steel pedestrian bridge, linking the two halves, on the second floor. The two halves of the roof structure are connected by a full-building skylight. Figure 2 illustrates the two symmetric halves and the bridge.

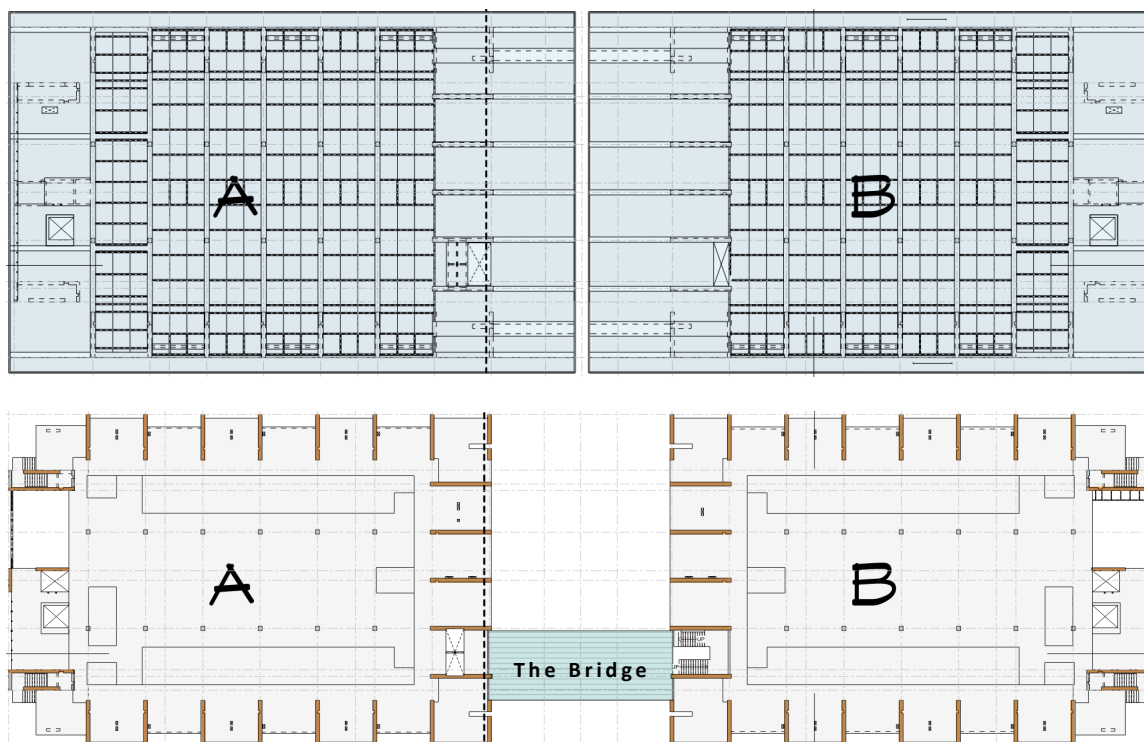


Figure 2. Structural layout of the building. The roof plan is above, and the second-floor plan is below.

The Analysis. The overall modeling utilizes a two-stage analysis, to account for the elastic response of the ground floor/basement, and the nonlinear response of the superstructure. The superstructure was modeled in OpenSees. Post-tensioned rocking walls were modeled using elastic beam-column elements seating on a bed of no tension concrete springs to account for rocking behavior. Mild steel energy dissipaters and post-tensioned threaded rods were modeled using discrete corotational trusses and were kinematically constrained to the walls. More information on modeling post-tensioned rocking walls can be found elsewhere (Basereh et al. 2024). The model for the post-tensioned rocking walls used a common

base, corresponding to the ground floor slab. A suite of 11 scaled ground motions, amplitude-scaled to the site-specific risk-based maximum considered earthquake (MCE_R) response spectrum were used to perform nonlinear response-history analysis. The wall reactions were then extracted and applied to an elastic model of the ground floor slab and basement walls using ETABS. The base model had nonlinear soil springs, and a semi-rigid ground floor diaphragm.

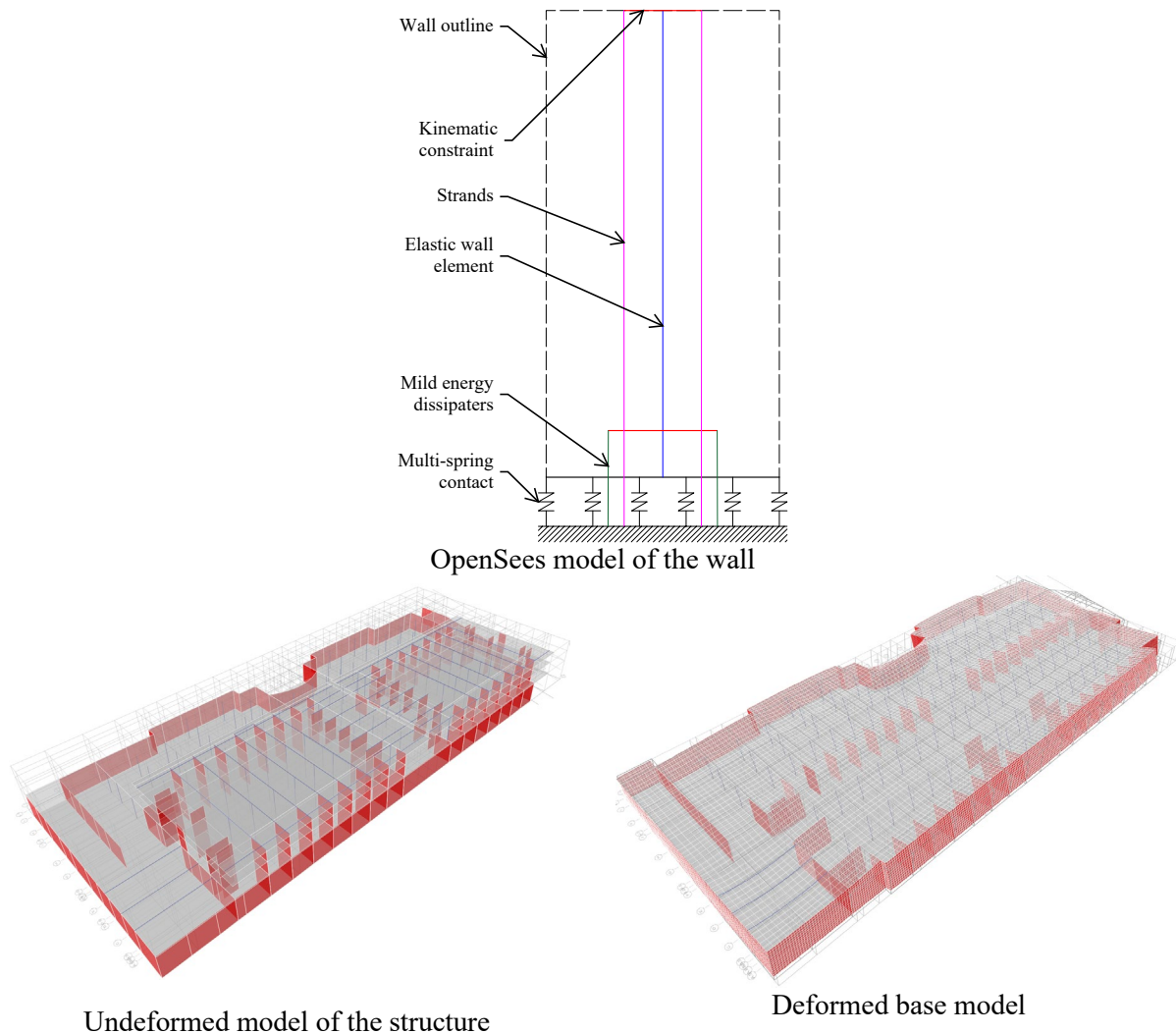


Figure 3. Two-stage analysis of the building.

The Lateral Design. The lateral force resisting elements are vertically post-tensioned rocking walls and were designed using the ACI 550.7-19 provisions developed for precast walls and adapted for cast-in-place walls. The walls form a discrete single-crack flexural hinge above the ground floor, at a prepared interface joint. Crossing the flexural joint are post-tensioned rods, mild steel yielding bars, and shear dowels. The high-strength post-tensioning rods provided around half of the flexural response, per ACI 550.7-19 requirements. The remaining flexural strength was provided by mild steel yielding bars. These bars are mechanically anchored beyond the flexural plane. The energy dissipating bars are machined to have reduced area and confine the yielding to a predetermined length (Holden and Restrepo 2003). Short greased shear dowels are provided at the boundary elements, crossing flexural-shear interface. These dowels are not necessary, per the ACI 550.7-19 guidelines, because the clamping force from the vertical

PT is sufficient to transfer shear at the rocking interface. However, we added them as a precaution against sliding. The flexural walls were designed using strut and tie models, with capacity design forces from nonlinear response history analyses (see Figure 4).

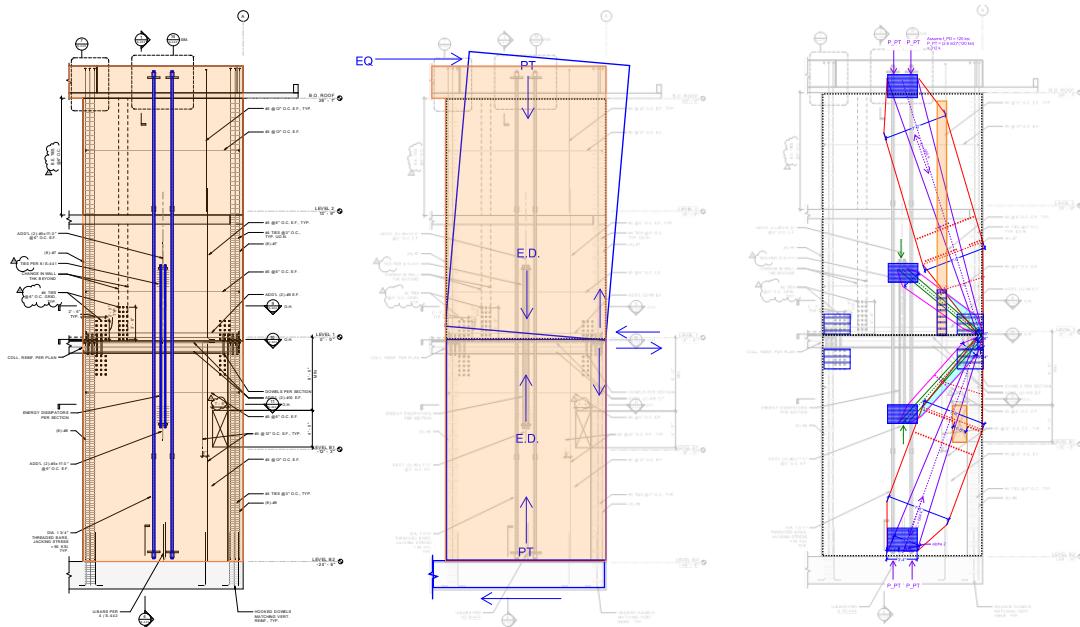


Figure 4. Strut and tie design of walls.

Real World Complications. The building's architectural expression, characterized by exposed concrete elements, dominated the design process, taking priority over structural performance. To mitigate the potential structural imbalance in the seismic response, certain concrete architectural elements were designed to remain non-load-bearing during earthquakes. This was achieved by incorporating hidden layers of compressible foam (see Figure 5).

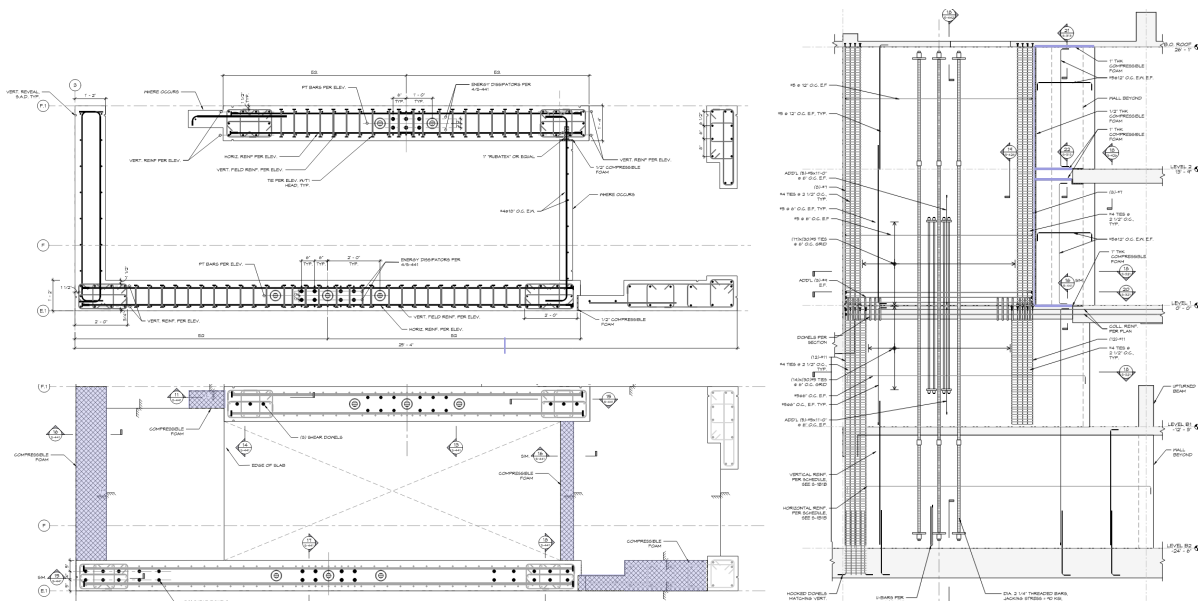


Figure 5. Structural disengagement using compressible foam.

The Pedestrian Bridge. The steel structure spans between two halves of the building at the 2nd floor. The bridge supports were designed to accommodate relative deformations between the supports, either coming together, pulling apart, or slicing motions (see Figure 6). To allow for various deformations, a single support was used to resist bridge inertial loads. The bridge is seismically anchored at the south end, and free to translate at the north end. The pin joint utilizes a spherical bearing to carry vertical gravity loads and seismic inertial loads, while allowing rotation in plan. The non-redundant pin support is capacity protected by a steel “butterfly plate” fuse (see Figure 7).

The Resilient Design of the Architectural Elements. The primary architectural elements were designed to be damage resistant, under the expected deformations when subjected to the MCE_R hazard level. This work was performed in direct collaboration with the architect and the glazing system design engineers. Examples include the large windows, the full-building roof skylight, and the architectural stairs. The low-damage glazing system and architectural stairs are shown in Figures 8 and 9, respectively.

The glazing detail notch shown in Figure 8 is a product of this collaboration. The thickness of the sealant was determined by deformation demands under the design earthquake (DBE) hazard. The windows are intended to remain damage-free under the mean DBE hazard, and to avoid breakage or dislodgement under the mean MCE_R hazard. The sealant can deform up to 100% in shear without damage. This deformation limit under the DBE hazard set the sealant thickness, impacting both the appearance and cost. The window glass was intended to rock within its frame, distorting the sealant in doing so. The potential for glass damage and potential dislodgement arises when the gap between the frame and the concrete closes, a condition set based on local deformations under the peak response associated with MCE_R hazard level.

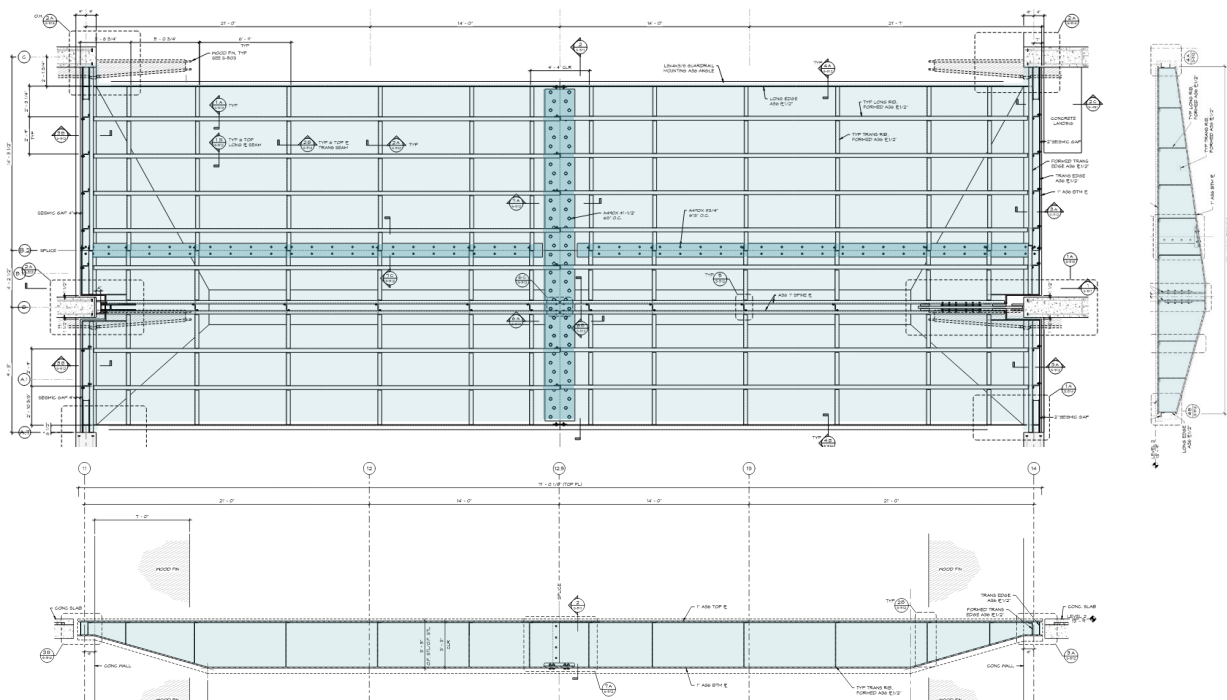


Figure 6. Steel bridge connecting two halves

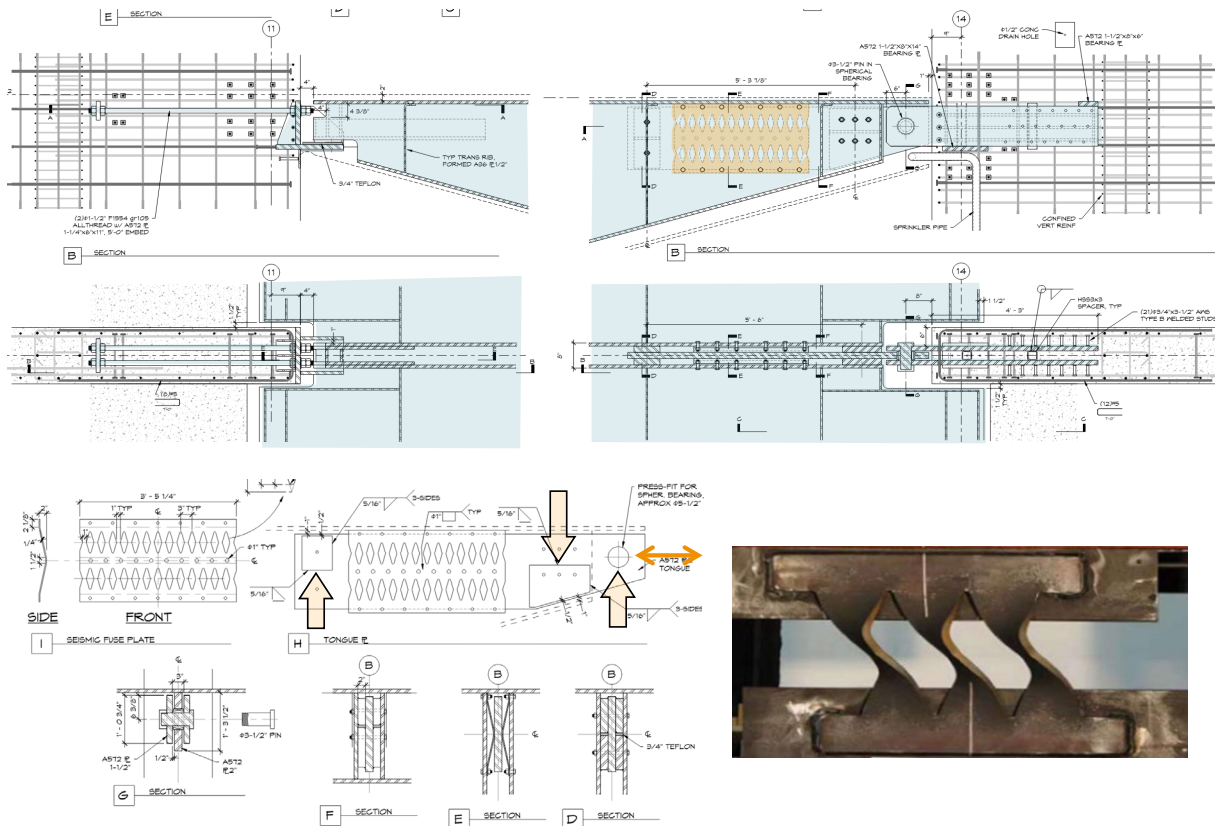


Figure 7. Fused pin support

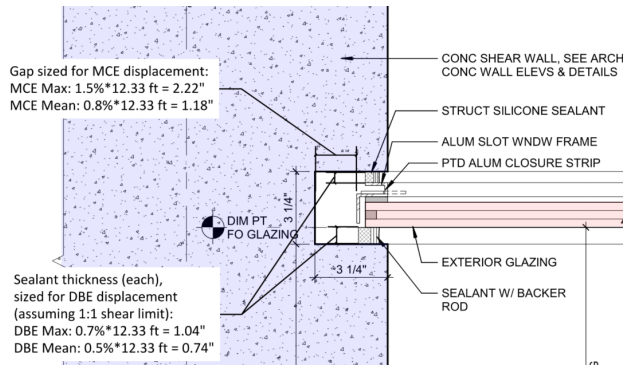


Figure 8. Low-damage glazing system

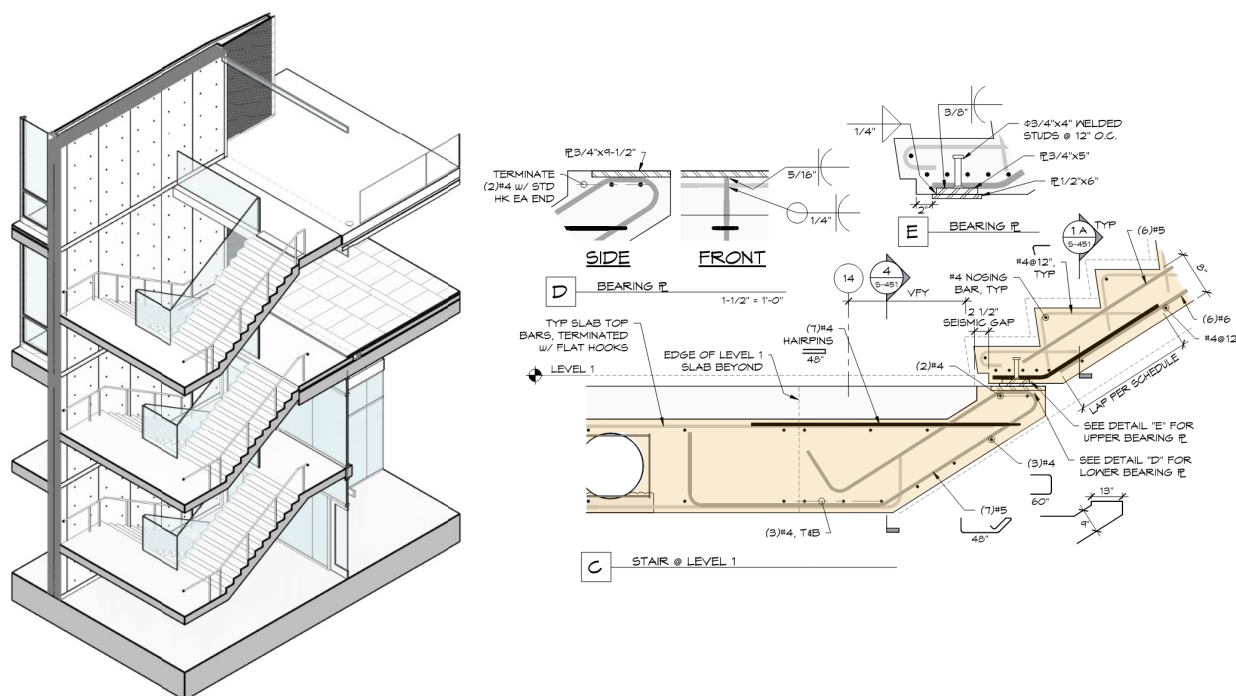


Figure 9. Low-damage architectural stairs

Economic Losses

The building's resilient performance was validated and iteratively refined using results from economic loss modeling conducted with FEMA P-58 and SP3 software. Many of the standard fragilities from FEMA P-58 required modifications to reflect particular design details of the project.

Two examples are the post-tensioned rocking recentering walls and the architectural skin system. As noted, the walls do not have a conventional plastic hinge but were designed to form a single flexural crack at the slab-wall interface. Consequently, fragility curves had to be customized to account for the potential of reduced damage, higher drift capacity, and re-centering.

For the glazing system, the fragilities were modified to reflect the increased deflection capacities provided by the design. As described above, the specific architectural details were directly related to the damage limit states.

Loss Modeling Trends and Refinements. The design achieved the following performance levels, per the modeled FEMA P-58/SP3 loss analysis:

- U. S. Resilience Council Safety Rating of 5 stars
- U. S. Resilience Council Damage Rating of 5 stars
- U. S. Resilience Council Recovery Rating of 5 stars
- Expected DBE loss of 4.7%
- Expected time for Functional Recovery of 4.5 days

These values required that the mechanical equipment, piping, and duct bracing be improved. The strengthening was required to be 1.3 to 2.5 times more than that of the basic US code.

Without the exceptionally high bracing settings, the results would be as follows:

- U. S. Resilience Council Safety Rating of 4 stars
- U. S. Resilience Council Damage Rating of 4 stars
- U. S. Resilience Council Recovery Rating of 4 stars
- Expected DBE loss of 5.9%
- Expected time for Functional Recovery of 5.0 days

As a benchmarking exercise, a code-designed surrogate was evaluated per the modeled FEMA P-58/SP3 loss analysis. The model setting have code mechanical bracing, and default fragilities for the lateral system and the architectural skin system. The results would be as follows:

- U. S. Resilience Council Safety Rating of 4 stars
- U. S. Resilience Council Damage Rating of 3 stars
- U. S. Resilience Council Recovery Rating of 4 stars
- Expected DBE loss of 11%
- Expected time for Functional Recovery of 5.3 days

Conclusion

This case study demonstrates that a high-performance resilient design is achievable with a fixed base structure in a seismically active area. It shows the relative importance of both architectural detailing for low damage, and the need for large bracing forces for MEP systems.

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NATIONAL ACTIVITY TO IMPROVE EARTHQUAKE RESILIENCE OF BUILDINGS

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Abstract

In recent years, many major earthquakes have occurred in Japan. The Building Standards Law aims to prevent the loss of human lives due to the collapse of buildings in major earthquakes, and although this goal has been achieved in recent earthquakes, it has been confirmed that some buildings are difficult to continue using after an earthquake. In order to ensure the post-earthquake functional use of buildings after such a major earthquake, it is necessary to design buildings that exceed the seismic performance stipulated by the standards. Therefore, this study is conducted to ensure the "seismic resilience performance" of reinforced concrete apartment buildings so that buildings can continue to be used even in the event of a major earthquake. This paper presents an overview of the study.

The seismic resilience that is the focus of this study is assumed to evaluate the degree of seismic performance degradation and the degree of restoration of RC buildings after a major earthquake, as shown in Figure 1. Regarding the degree of seismic performance degradation, we focus on the damage level of buildings calculated using the damage classification criteria, calculate the damage level using static nonlinear incremental analysis, and demonstrate the validity of the calculation method. On the other hand, regarding the degree of restoration, we focus on shortening the investigation time required after an earthquake, and introduce our efforts to establish evaluation criteria for early restoration by introducing structural health monitoring technology.

Introduction

The need for post-earthquake functional use has been recognized due to recent earthquake damage, and research on this issue is being conducted [1] and technical data is being collected. On the other hand, the earthquake resistance regulations in Japan are set for ensuring seismic safety performance during severe earthquakes as a minimum standard, and the aforementioned "post-earthquake functional use" is positioned as something more than the earthquake resistance performance required by the current standards. Although such performance is stipulated in the Act on Promotion of Quality Assurance of Housing (hereinafter referred to as the QAA(Quality Assurance Act)), it is stipulated as something related to seismic safety performance against severe earthquakes, and not only is it rarely applied to RC apartment buildings, but it is also insufficient for users to understand the performance [2], and this issue remains. In addition, the certification standard for long-term high-quality housing has been revised recently, and RC apartment buildings that meet the current standards will be certified under the above standard if the base shear coefficient is over than 0.3, the frame has a ductility, and the maximum response story drift angle during severe earthquake is 1/75 or less [3]. Therefore, after the validity of the above standard is clarified, remained issue should be investigated.

Given the above background, this study aims to establish a seismic performance evaluation system based on the Quality Assurance Act as a framework for ensuring post-earthquake functional use for RC apartment buildings, and will first conduct the following studies:

- 1) Study of the structural characteristics of the RC apartments to be evaluated
- 2) Study of measures to popularize earthquake response evaluation methods
- 3) Study of damage evaluation methods based on earthquake response

This paper shows the results of above study.

Outline of Analytical Model

Overview of target houses

The target houses used in this paper are four buildings with rectangular column moment resisting frame (MRF) structures and those buildings with wing-walls (see in Table1). The floor plans for each structure are common and are shown in Figure 1. The parameters are the number of floors and the with/without of wing-walls. The reason for targeting structures with wing-walls here is that previous research [4] has cited them as an effective measure for ensuring post-earthquake functional use. The target buildings here are designed to satisfy the certification criteria based on the Quality Assurance Act mentioned above.

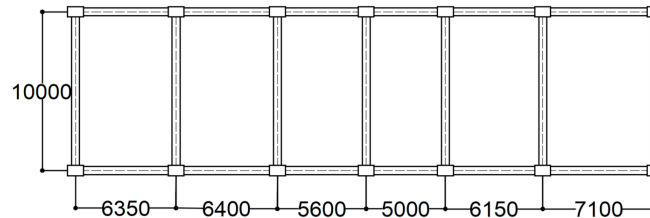


Figure 1. Floor plans

Table 1. Outline of target frames					
Model	Number of stories	Wing-walls With/Without	Ultimate shear strength/Ultimate flexural strength for beam	1st natural period (s)	
5a	5	WO	1.76	0.37	
5b		W	1.67	0.26	
8a	8	WO	1.58	0.52	
8b		W	1.39	0.38	
11a	11	WO	1.49	0.64	
11b		W	1.34	0.47	
14a	14	WO	1.58	0.75	
14b		W	1.3	0.56	

Modeling Methodology

In this paper, a load incremental analysis was performed using the analysis program SNAP ver8 from Structural Systems Co., Ltd. A node was set at the center of the column and beam members to model them. The critical cross-section position was the member face position, and the rigid zone end was set at a position $D/4$ (D : member depth) less than the face position. The constraint conditions for each node were pin support for the first-floor column base and free for the rest, and the foundation beam was also modeled. In addition, an external force distribution based on the domestic building code (top heavy distribution) was used to perform a non-linear incremental analysis in one direction of the building. The column and beam members were treated as springs on line element. Columns with wing-walls was treated as one column member as single-axis springs. The columns were treated as single-axis springs, flexural springs uses MN model considering flexural moment-axial force interaction, and shear springs. The beams were treated as flexural springs and shear springs as single-axis springs.

Static nonlinear incremental analysis results

Figure 2 shows the relationship between the base shear and top floor horizontal drift angle and the relationship between the collapse mechanism and the ratio of tangent stiffness to initial stiffness based on

the static incremental analysis. The collapse mechanism is defined as the time when plastic hinges occur at the 1st floor column base and both ends of all beams. The ratio of tangent stiffness is the tangent stiffness divided by the initial stiffness, and the initial stiffness is the secant stiffness of the first unique points calculated by trilinearizing the base shear - top deformation relationship. The collapse mechanism occurs before the deformation reaches three times the deformation at the tangent stiffness ratio of 5% in all models, confirming the validity of the method for simply evaluating the collapse mechanism shown in Reference [5]. The base shear coefficient of the frame with wing-walls is 0.4 or more, which is larger than that of a MRF, which is about 0.3. Figure 3 shows the plastic hinge distribution of a MRF at the maximum story drift angle of 1/75. At a scale of 8 stories or more, it can be seen that no plastic hinges occur in the column base or upper story beams. Table 2 shows the average ductility factor of all beams at the maximum story drift angle of 1/75. When the number of stories is the same, the average ductility factor of all beams of a MRF structure is small, and the value decreases as the number of stories increases. The structural characteristics of the RC apartment building designed in this study show that there is a wide range of damage levels at the maximum story drift angle of 1/75. Therefore, it is clear that the maximum story drift angle is not suitable as an index for uniformly evaluating the damage level of buildings.

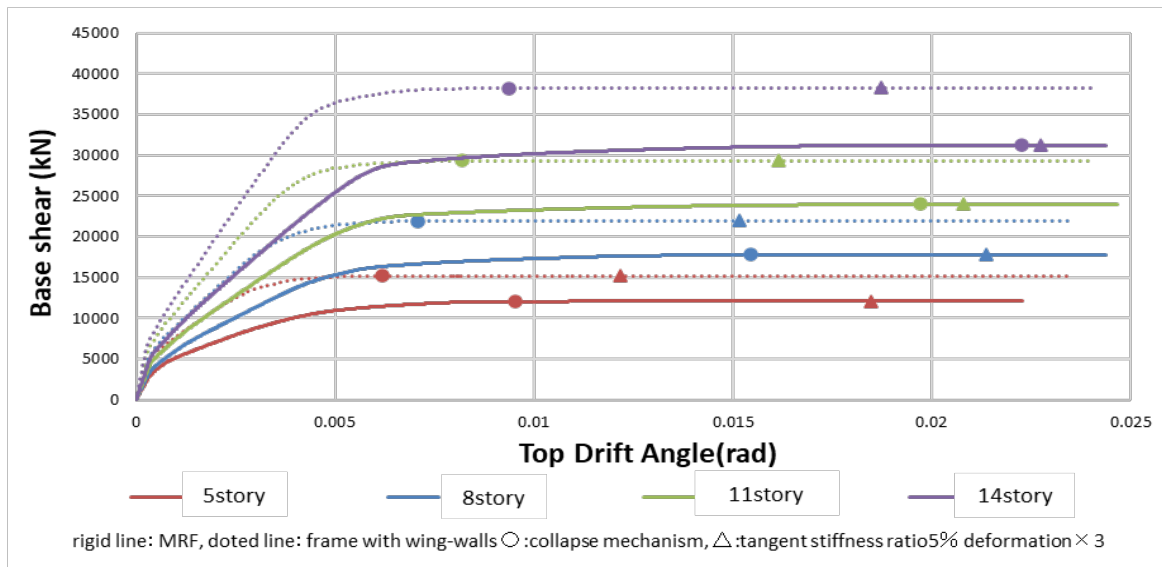


Figure 2. Base shear – Top floor horizontal drift angle

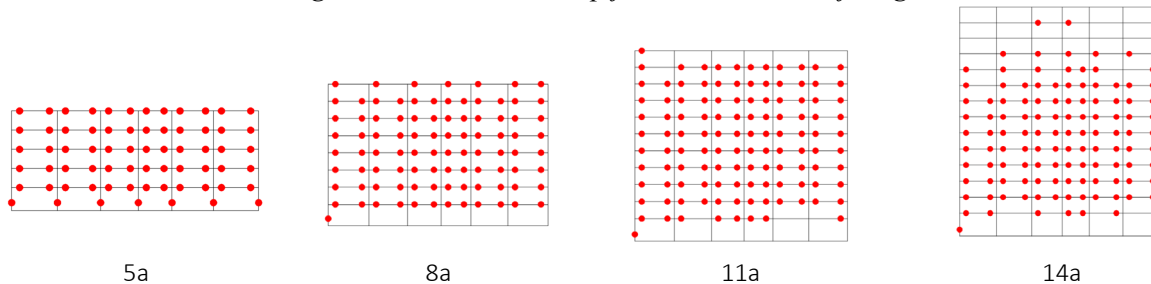


Figure 3. Plastic hinges distribution at max story drift angle of 1/75

Table 2. Average ductility factor of all beams at max story drift angle of 1/75

	5a	5b	8a	8b	11a	11b	14a	14b
Average ductility factor	5.53	6.32	2.67	4.80	2.08	3.72	1.55	2.65

Overview of a Web program for disseminating earthquake response evaluation methods

Flowchart of the response evaluation Web program

The response evaluation web program is targeted at buildings that are structurally designed according to calculation route 3 (the horizontal load carrying capacity calculation method using non-linear pushover analysis) among RC apartment buildings. Figure 4 shows the flow chart of this program. First, the user ① performs structural design using a commercially available integrated structural calculation program. After that, in the response evaluation web program, the user uploads electronic data (Data A) including the horizontal load carrying capacity calculation results output from the above commercial program. However, since Data A does not contain all the information necessary for subsequent calculations, some information is manually inputted by the user. After that, ② the file converter extracts only the necessary data from the input data, and ③ the response point step calculation is performed. When calculating the horizontal load carrying capacity with the commercial program, the non-linear pushover analysis is performed by incrementing the external force at each step. Since the results of this static incremental analysis are discrete data, when calculating the response points using the response spectrum method, it is necessary to interpolate between steps, and as a result, the response point step is calculated as a small number of steps rather than a natural number. In this study, this number is called the "response point step." Since the response point step is a small number, data A does not contain analysis data at that response point step. Therefore, the user reads the response point step displayed on the screen, changes the increment between steps in the commercial program, and performs ④ static incremental analysis again to obtain analysis data at the same point as the response point step. After that, in the response evaluation web program, Data B, which contains the data at the response point step, is uploaded and manually inputted by the user to perform ⑤ response evaluation, and the calculation results are displayed in the web program.

Earthquake Response point step calculation

In the response step calculation, the calculation is performed according to the flowchart shown in Figure 5, and the response step and graph for each direction are output. The calculation of the yield point displacement was using the following two methods (1) and (2).

(1) Calculation method A according to the notification by Ministry of Construction

The calculation is performed according to Section 2-3 of the Ministry of Construction Notification No. 1457 of 2000. This calculation method calculates the yield point displacement by replacing it with a bilinear curve as shown in Figure 6. However, the representative displacement at the safety limit (Δ_s in Figure 6) cannot be defined only by the results of the load increment analysis in the horizontal load carrying capacity calculation. Therefore, this time, the calculation was performed using the representative displacement at the final step of Data A.

(2) Calculation method B according to the performance evaluation guidelines [6]

The calculation method according to the performance evaluation guidelines [6] calculates the yield point displacement by replacing it with a trilinear curve as shown in Figure 7. However, because the definition of the "significant specific point" is unclear when determining the tentative yield point ($\delta y'$ in Figure 7) shown in the Performance Evaluation Guidelines [6], the tentative yield point $\delta y'$ was calculated as the point when the representative point drift angle (= representative displacement / representative point height) is $1/150$ (rad). By setting it in this way, in this calculation, it is possible to replace the total energy with an equivalent trilinear when the representative point drift angle is $1/75 = 1.33\%$ (rad).

Earthquake response evaluation

In the response evaluation, the maximum story drift angle in each direction and for each story is calculated by input Data B. Data B stores the story drift angles at the time of the response step

corresponding to the locations of the vertical members, and the maximum values of these are calculated for each direction and for each story.

Earthquake response evaluation results using web program

Table 3 shows the results of a response evaluation conducted using the developed web program (calculation methods A and B) for the RC apartment building models shown in Table 1. For all models, the maximum story drift angle was smaller with calculation method A than with B. This is because when calculating the yield point displacement, Δ_s was used as the representative displacement at the final step, which reduced the yield displacement and therefore the response. As a topic for future study, it is necessary to properly evaluate the representative displacement at the safety limit.

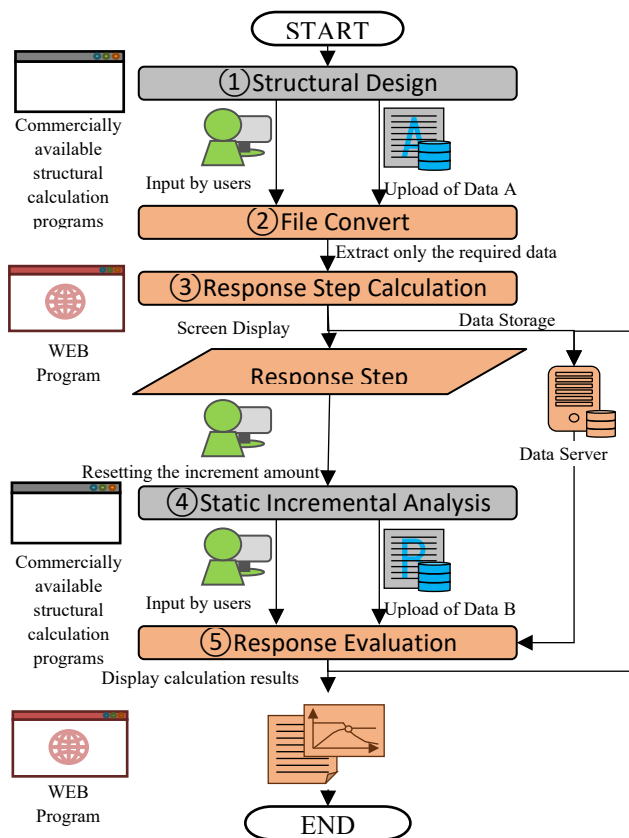


Figure 4. Flowchart of response evaluation web program

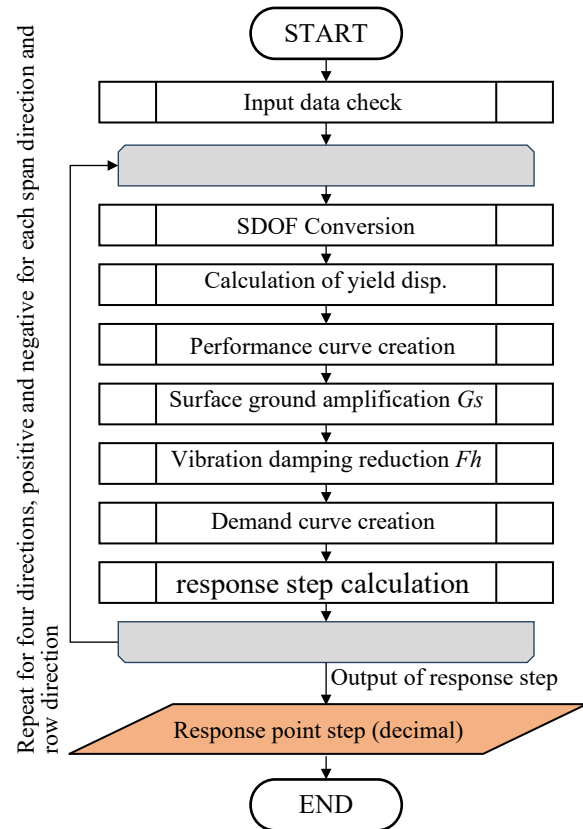


Figure 5. Flowchart of response step calculation

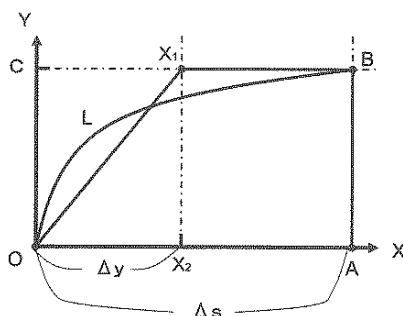


Figure 6. Calculation method of Notification No. 1457

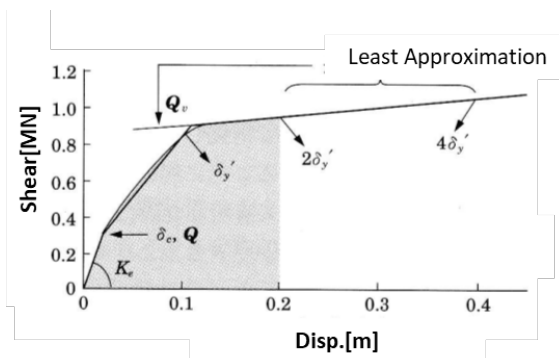


Figure 7. Calculation method of guidelines [6]

Table 3. List of maximum story drift angles

Model	A	B	Model	A	B
5a	1.86%	2.01%	5b	1.23%	1.32%
8a	1.68%	1.94%	8b	1.05%	1.16%
11a	1.52%	1.79%	11b	1.00%	1.14%
14a	1.51%	1.66%	14b	1.06%	1.10%

Damage Evaluation Method Using Global Damage Degree at Earthquake Response Points

In this study, the global damage degree of whole building is calculated using the seismic performance residual ratio (hereinafter R value) following the previous study [7]. In the study, the local damage level of members was calculated by the member drift angle, but in this paper, the shear deformation ratio of each column and beam member is calculated for the overall deformation obtained by performing a nonlinear incremental analysis. The relationship between the damage level of each member and the ductility factor is determined according to the deformation ratio, the damage level of each member is calculated, and these are added up to calculate the R value. The damage degree is defined as slight when the R value is 95% or more, minor damage when it is 80% to 95%, medium damage when it is 60% to 80%, and severe damage when it is less than 60%.

The safety limit point in this paper is defined as the point at which the carrying shear force of the member begins to decrease. This point is the point at which the ultimate shear strength decreases and reaches the ultimate flexural strength, according to past study [8]. The R value at maximum response point is calculated by the response calculation method described above.

Calculation results of residual seismic performance

In the calculation of the R value, the flexural deformation of both columns and beams is dominant in all models, so in this paper, the ductility factor obtained from the flexural deformation at the member end shown in Table 4 is used. Damage level I is determined by the ductility factor at the crack strength of each member, and damage level II and above are determined by the ductility factor in previous experimental study [9]. The R value and the top drift angle of each frame are shown in Figure 8, and the building representative ductility factor relationship is shown in Figure 9. Here, the building representative ductility factor is the value obtained by dividing each top drift angle by the top drift angle at the second specific point calculated by trilinearizing the base shear-top drift angle relationship. The R value is strongly correlated with the building representative ductility factor regardless of the building scale or structure. In addition, the larger the building scale is, the higher the R value for both the top drift angle and building representative ductility factor of entire building. The reason is that, since this frame is a total collapse type, the R value is calculated by totaling the damage level of all members, and the larger the building scale, the greater the ratio of the number of column members with less damage on the second or higher floors to the total numbers.

Table 5 shows the R value at the safety limit and maximum response point defined in this study. The R value at the building safety limit is generally around 60%, which is the classification for severe damage. On the other hand, the R value at the maximum response is judged to be severe damage for the 5story model, but other models are judged to be minor to moderate damage.

Figure 10 shows the relationship between the R value at maximum response point and the top drift angle, Figure 11 shows the building representative ductility factor, and Figure 12 shows the average ductility factor of all beams at response time. Figure 10 shows that there is a correlation between the R value at maximum response point and the top drift angle for the MRF and the MRF with wing-walls. On the other hand, Figures 11 and 12 show a strong correlation between the R value at maximum response point and the representative building ductility factor and the average ductility factor of all beams, regardless of the type of frame. By keeping the representative building ductility factor below 1.7 and the average ductility

factor of all beams below 2.0, it is possible to limit the extent of damage to minor damage, while by keeping the representative building ductility factor below 3.0 and the average ductility factor of all beams below 5.0, it is possible to limit damage to moderate damage. Table 6 shows the R value at the building safety limit and maximum response point. The ratio of maximum response relative to the safety limit state was generally less than 0.7 times, with frames with wing-walls tending to have smaller values than MRF.

Table 4. Ductility factor of flexural rotation angle at member end - damage degree relationship							
	Damage level	I	II	III	IV	V	
	Column	crack	0.87	1.81	3.7	10	
	Cloum with wing-walls		0.9	2.3	4.37	10	
	Beam		0.87	1.57	2.79	10	

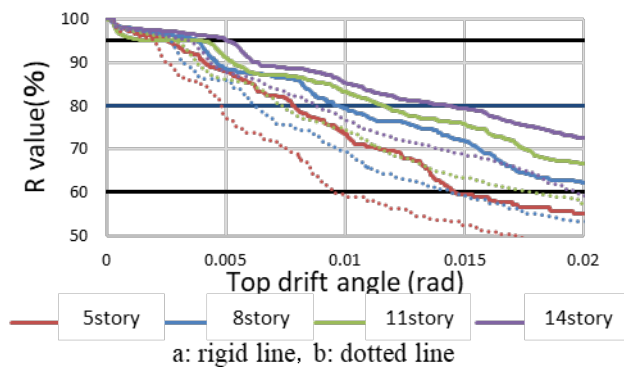


Figure 8. R value and top drift angle

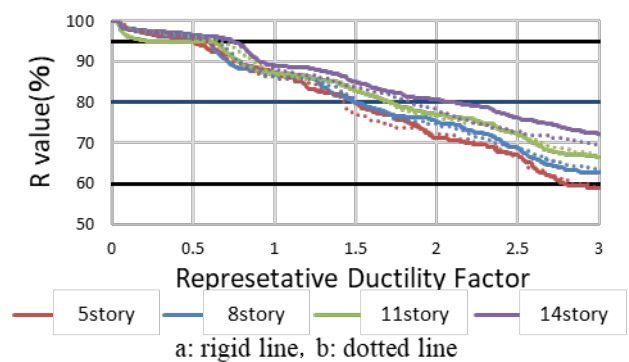


Figure 9. R value and representative ductility factor

Table 5. R value at each point

	5a	5b	8a	8b	11a	11b	14a	14b
Safety limit point	42.1	42.2	56.2	51.8	65.4	59.0	63.2	56.1
Max. response point	56.7	54.6	70.8	66.8	77.0	75.0	83.6	82.0

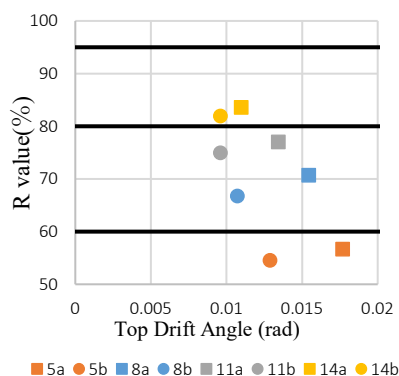


Figure 10. R value at maximum response point and the top drift angle

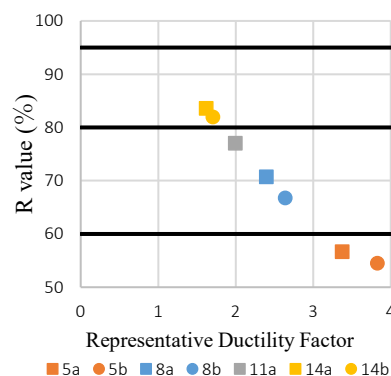


Figure 11. R value at maximum response point and building representative ductility factor

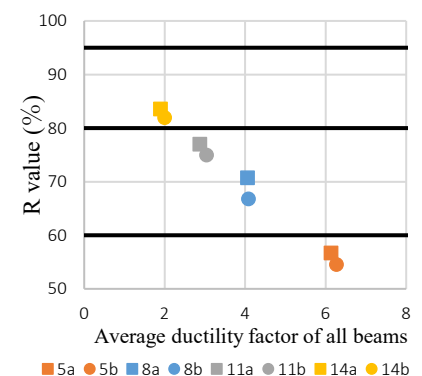


Figure 12. R value at maximum response point and average ductility factor of all beams

Table 6. Representative ductility factor at response point, safety limit state and the ratio

	5a	5b	8a	8b	11a	11b	14a	14b
Representative ductility factor at response point	3.37	3.83	2.39	2.64	1.99	2.21	1.62	1.70
Representative ductility factor at safety limit state	6.93	8.09	4.22	5.37	3.18	4.36	4.07	5.06
The ratio of each ductility	0.49	0.47	0.57	0.49	0.63	0.51	0.40	0.34

Conclusion

In this paper, after presenting the structural characteristics of 8 RC apartment buildings designed using standard Japanese structural calculation methods, we have presented an overview of the response evaluation web program and a calculation results. Moreover, using nonlinear incremental analysis, we showed a method to calculate the global damage degree of a building at maximum response point, and demonstrated that there is a correlation between the damage degree and the ductility factor of the building. In addition, the ratio of the ductility factor of the response point to the seismic safety limit was below 1.0, and it was shown that the model building examined had a high seismic safety performance. In the future, we will make a plan to modify the web program based on the issues we have identified and make it available to the public. In addition to the evaluation method of seismic safety performance shown here, we will develop a calculation method that focuses on the repair time after an earthquake, and evaluate the margin for the repair limit state, and finally incorporate it into Japan's long-term high-quality housing certification system.

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FUNCTIONAL RECOVERY REQUIREMENTS FOR NEW BUILDINGS IN THE NEHRP PROVISIONS AND ASCE 7

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Abstract

As part of an ongoing project under FEMA's National Earthquake Hazards Reduction Program (NEHRP) Architect-Engineer Contract, the NEHRP *Recommended Seismic Provisions for New Buildings and Other Structures* has formed a Provisions Update Committee (PUC) under the management of the Building Seismic Safety Council. The PUC is developing potential code language for functional recovery for the 2026 NEHRP Provisions. If approved, the potential new appendix will be considered by the American Society of Civil Engineers (ASCE) in its Structural Engineering Institute's consensus process for ASCE 7-28 *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* and then into the International Building Code. The proposed provisions expand the NEHRP Provisions and ASCE 7 which have historically focused almost exclusively on ensuring life safety at the design earthquake and limiting the probability of collapse at the maximum considered earthquake. Functional recovery categories are established relative to the importance of a function within a structure to community-level recovery following a risk-targeted functional recovery earthquake. Quantitative criteria are set for each category consisting of a target functional recovery time and associated probability of exceedance. These explicit criteria are used in supporting studies which address (a) prescriptive procedures for functional recovery, and (b) explicit functional recovery time methods employed on a project-specific basis. The proposed provisions and commentary in the NEHRP Provisions represent a substantial step toward a more recoverable new building portfolio in the United States. Final results, conclusions, and recommendations are still pending.

Overview of Requirements

The functional recovery appendix proposed for the 2026 NEHRP Provisions and ASCE 7-28 is organized into eighteen sections as described below.

Section A.1 General. This section establishes the scope of the functional recovery appendix and the applicability of its sections depending on Functional Recovery Category and building versus nonbuilding structure type. It is reiterated in this section that the requirements of the functional recovery appendix are in addition to, not in lieu of, the requirements in the remainder of the NEHRP Provisions and ASCE 7. In short, while obviously somewhat interrelated with the primarily life-safety and collapse objectives of seismic design in the NEHRP Provisions and ASCE 7, functional recovery is an entirely new set of criteria which must be satisfied independently. However, certain exemptions and deemed-to-comply cases are provided. Finally, the target risk, reliability, and functional recovery times by Functional Recovery Category are established. See Table 1 and 2 for proposed section applicability for building and nonbuilding structures, respectively, and Table 3 for proposed functional recovery time targets by Functional Recovery Category.

Section A.2 Definitions. This section establishes the new definitions including, but not limited to, recovery objective, recovery state, functional recovery, impeding factor, and external factor.

Section A.3 Symbols. This section establishes the new symbols.

Section A.4 Basic Intended Function. This section establishes the responsibility for identifying the basic intended function of the structure, lists the systems which need to be considered for functional recovery design by the responsible parties, and specifies allowable exemptions of such systems.

Section A.5 Functional Recovery Category. This section establishes the Functional Recovery Categories ranging from A to E in decreasing order of performance similar to grades. Functional Recovery Categories are to functional recovery what Risk Categories are to life safety and collapse for seismic design. See Table 4 for proposed Functional Recovery Category descriptions.

Section A.6 Seismic Ground Motion Values. This section establishes the Risk-Targeted Functional Recovery Earthquake (FRE_R), its response spectrum, and its response spectral acceleration parameters which are used throughout the functional recovery appendix. The FRE_R is the risk-based equivalent of the Risk-Targeted Maximum Considered Earthquake (MCE_R) but for functional recovery objectives instead of collapse. It is defined by 10% risk in 50 years of exceeding the target recovery time for a functional recovery fragility that has (1) a 25% reliability, and (2) a logarithmic standard deviation of 0.6.

Section A.7 Geologic Hazards. This section establishes the functional recovery criteria for geologic hazards such as surface fault rupture, slope instability, liquefaction, lateral spreading, etc. Functional recovery to tsunami hazards is not included in the functional recovery appendix since, in the development of the provisions, it was decided that tsunami was a related but ultimately separate hazard which warrants its own study.

Table 1. Proposed Section Applicability - Buildings

Description	Section																	
	A.1	A.2	A.3	A.4	A.5	A.6	A.7	A.8	A.9	A.10	A.11	A.12	A.13	A.14	A.15	A.16	A.17	A.18
Buildings																		
Buildings other than seismically isolated buildings or buildings with damping systems assigned to Functional Recovery Category A meeting certain, specific limitations	X	X	X	X	X	X	X	X	-	X	X	X	X	X	X	X	-	X
Buildings other than seismically isolated buildings or buildings with damping systems assigned to Functional Recovery Category B	X	X	X	X	X	X	X	X	-	X	X	X	X	X	X	-	-	X
Buildings other than seismically isolated buildings or buildings with damping systems assigned to Functional Recovery Category C or D	X	X	X	X	X	X	X	X	-	X	X	X	X	X	-	-	-	X
Seismically isolated buildings assigned to Functional Recovery Category A	X	X	X	X	X	X	X	-	-	X	X	X	X	X	X	X	-	X
Seismically isolated buildings assigned to Functional Recovery Category B	X	X	X	X	X	X	X	-	-	X	X	X	X	X	X	-	-	X
Seismically isolated buildings assigned to Functional Recovery Category C	X	X	X	X	X	X	X	-	-	X	X	X	X	X	-	-	-	X
Any building assigned to Functional Recovery Category A	X	X	X	X	X	X	X	-	-	-	X	X	X	X	X	X	X	X
Any building assigned to Functional Recovery Category B	X	X	X	X	X	X	X	-	-	-	X	X	X	X	X	-	X	X
Any building assigned to Functional Recovery Category C or D	X	X	X	X	X	X	X	-	-	-	X	X	X	X	-	-	X	X

X indicates section is required; - indicates section is not required

Table 2. Proposed Section Applicability – Nonbuilding structures

<i>Description</i>	<i>Section</i>																	
	<i>A.1</i>	<i>A.2</i>	<i>A.3</i>	<i>A.4</i>	<i>A.5</i>	<i>A.6</i>	<i>A.7</i>	<i>A.8</i>	<i>A.9</i>	<i>A.10</i>	<i>A.11</i>	<i>A.12</i>	<i>A.13</i>	<i>A.14</i>	<i>A.15</i>	<i>A.16</i>	<i>A.17</i>	<i>A.18</i>
Nonbuilding Structures																		
Nonbuilding structures other than seismically isolated structures or structures with damping systems assigned to Functional Recovery Category B	X	X	X	X	X	X	X	-	X	X	X	X	X	X	X	-	-	X
Seismically isolated nonbuilding structures assigned to Functional Recovery Category B	X	X	X	X	X	X	X	-	-	X	X	X	X	X	X	-	-	X

X indicates section is required; - indicates section is not required

Table 3. Proposed Target Functional Recovery Times

<i>Functional Recovery Category</i>	<i>Target Functional Recovery Time</i>	
	<i>General Expectation</i>	<i>Acceptance Criteria</i>
A	Immediate	0 hours
B	Within a few days	3 days
C	Within a couple of months	2 months
D	Within half a year	6 months
E	No target time	No target time

Table 4. Proposed Functional Recovery Categories

<i>Functional Recovery Category</i>	<i>Community Functions Supported by Structures</i>
A	Community functions that support essential and immediate safety and survival needs.
B	Community functions that support urgent and basic human survival needs and that prevent the escalation of adverse disaster consequences.
C	Community functions that support routine self- and group-preservation, and that sustain short- and long-term economic, educational, and governance activities and services.
D	Community functions that enhance a community's general well-being and expedite the return to normalcy.
E	All other community functions not addressed in Categories A through D. These include functions expected to support people's quality of life that are not essential to the community's recovery.

Section A.8 Analysis and Design of Structural Systems for Buildings. This section establishes criteria for the structural system in buildings and is based on performing an additional ASCE 7 Chapter 12 check but modified for functional recovery. Some of the modifications include:

- Replacement of the ratio of the response modification coefficient to seismic importance factor, R/I_e , with a new functional recovery response modification coefficient, R_{fr} .
- Replacement of the allowable story drift ratio, Δ_a , with a new functional recovery allowable story drift limit, Δ_{afr} .
- Replacement of the deflection amplification factor, C_d , with a new functional recovery deflection amplification factor, C_{dfr} , the latter taken equal to R_{fr} .
- Amplification of forces for force-controlled actions in foundations to an overstrength/capacity-limited level.

The functional recovery response modification coefficient, R_{fr} , and functional recovery allowable story drift limit, Δ_{afr} , are dependent on both the structural system and Functional Recovery Category and are established based on functional recovery analyses of structural-only archetype models (Blowes et al., in review). See Table 5 for proposed R_{fr} and Δ_{afr} by structural system.

Section A.9 Analysis and Design of Structural Systems for Nonbuilding Structures. This section establishes criteria for the structural system in nonbuilding structures and is based on performing an additional ASCE 7 Chapter 15 check but modified for functional recovery. Modifications are similar in framework to those in Section A.8 for buildings except that the R_{fr} values for nonbuilding structures are based on expert judgment as opposed to analytical study.

**Table 5. Proposed Functional Recovery Response Modification Coefficient
and Allowable Story Drift**

<i>Seismic Force-Resisting System</i>	<i>R_{fr} and Δ_{afr} by Functional Recovery Category</i>	
	<i>A, B and C</i>	<i>D</i>
BEARING WALL SYSTEMS		
Special reinforced concrete shear walls	3.5 0.01h _{sx}	4 0.0125h _{sx}
Reinforced concrete ductile coupled walls	4 0.01h _{sx}	5 0.0125h _{sx}
Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	3 0.01h _{sx}	3.5 0.0125h _{sx}
BUILDING FRAME SYSTEMS		
Special reinforced concrete shear walls	4 0.01h _{sx}	5 0.0125h _{sx}
Reinforced concrete ductile coupled walls	4 0.01h _{sx}	5 0.0125h _{sx}
Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	3 0.01h _{sx}	3.5 0.0125h _{sx}
Steel buckling-restrained braced frames, non-moment-resisting beam-column connections	5 0.01h _{sx}	5 0.01h _{sx}
Steel buckling-restrained braced frames, moment-resisting beam-column connections	5.5 0.01h _{sx}	6.5 0.0125h _{sx}
MOMENT-RESISTING FRAME SYSTEMS		
Steel special moment frames	5.5 0.01h _{sx}	6.5 0.015h _{sx}
Special reinforced concrete moment frames	4 0.0075h _{sx}	5 0.0075h _{sx}

Section A.10 Analysis and Design Criteria of Nonstructural Systems. This section establishes criteria for the nonstructural systems of buildings or nonbuilding structures and is based on performing an additional ASCE 7 Chapter 13 check but modified for functional recovery. Some of the modifications include:

- Replacement of the component importance factor, I_p , with a new seismic functional recovery component importance factor, I_{pfr} , which varies by Functional Recovery Category.
- Definition of a seismic floor acceleration (i.e., an estimate of the peak floor acceleration) determined using a modified version of the component design force, F_p , equation in ASCE 7.
- Requirement to check nonstructural elements, components, equipment and distribution systems spanning between points of attachment (e.g., the ability of a pipe or duct to span between brace points) unless demonstrated as deemed to comply for the seismic floor acceleration.
- Additional prescriptive design requirements for architectural, mechanical, electrical, plumbing and fire protection components.

The seismic functional recovery component importance factor, I_{pfr} , is established based on functional recovery analyses of full building archetype models (Blowes et al., in review). Deemed-to-comply criteria for nonstructural elements, components, equipment and distribution systems spanning between points of attachment are based on target probabilities of component-level damage that could impede recovery back-calculated from the full building archetype models (Molina-Hutt and Zimmerman, 2024). See Table 6 for proposed seismic functional recovery importance factors by Functional Recovery Category.

Table 6. Proposed Functional Recovery Nonstructural Component Importance Factors by Functional Recovery Category for Earthquake Loads

<i>Functional Recovery Category</i>	<i>Seismic Functional Recovery Component Importance Factor, I_{pfr}</i>
A and B	1.75
C	1.5
D	1.25

Section A.11 Seismic Certification Requirements for Components and Equipment. This section establishes seismic certification requirements for components and equipment in addition to providing exceptions and deemed-to-comply. Seismic certification for functional recovery is an extension of the current practice of seismic certification of critical equipment for Risk Category IV structures (e.g., an air handling unit serving an operating room in a critical care hospital) but now to all components and equipment necessary to support the structure's basic intended function. Deemed-to-comply criteria are based on target probabilities of component-level damage that could impede recovery similar to those in Section A.10 (Molina-Hutt and Zimmerman, 2024).

Section A.12 Documentation and Review. This section establishes the aspects of functional recovery which are required to be shown on the construction documents, stipulates requirements for delegated design and design changes during construction, and establishes the role and responsibilities of the new role of the functional recovery coordinator.

Section A.13 Observation. This section establishes the responsibility for and reporting on observation and sets requirements for pre-construction functional recovery conference(s) and post-construction functional recovery walk-through(s). It also defines the act of observation as similar to structural observation as defined in modern building codes and material standards (i.e., visual observation for general conformance) except not exclusive to structural components.

Section A.14 Quality Assurance. This section establishes the requirements for a quality assurance plan and a contractor responsibility statement as well as detailing the special inspection and testing required for functional recovery.

Section A.15 Post-Earthquake Functional Recovery Plan. This section establishes the responsibility for and scope of the functional recovery plan. The functional recovery plan is the adaptation/extension of Building Occupancy Resumption (BORP) or Back-to-Business (B2B) plans within the format of the NEHRP Provisions and ASCE 7.

Section A.16 Emergency Systems. This section establishes the requirement for emergency/backup electrical power, water supply, and sewage storage for structures assigned to Functional Recovery Category A, primarily as a means to buffer against external factors such as utility disruption.

Section A.17 Explicit Functional Recovery Assessment Procedure. This section establishes an alternate means of compliance to the prescriptive procedures of Sections A.8 (structural) and A.10 (nonstructural). It is intentionally referred to as an explicit assessment procedure as opposed to a performance-based procedure in order to avoid the mis-association that all aspects of performance-based design (e.g., nonlinear response history analysis) are required.

Section A.18 Consensus Standards. This section occurs as the last section of all seismic chapters to serve as a pointer to the consensus standards.

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BENCHMARKING A PRESCRIPTIVE DESIGN METHOD FOR LOW-DAMAGE SEISMIC DESIGN IN NEW ZEALAND

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Abstract

New Zealand is in the process of developing guidance for low-damage seismic design (LDSD) of buildings. The guidelines include three volumes and are intended to set out an overall seismic design approach that delivers improved building performance in future earthquakes. The first volume introduces the benefits of LDSD and what this means for developers, owners, and tenants. The second volume defines a LDSD framework for engineers and architects, setting out performance objectives for LDSD and addressing the inter-relationship between the overall outcome objectives and corresponding performance goals and physical states. The third volume will provide detailed technical guidance on how to achieve a low-damage design, from project establishment and preliminary design through to engineering design criteria to be used in detailed design. The performance goals for LDSD include targets in terms of expected annual losses and the probability of exceeding a level of repair cost (relative to building replacement cost). This paper reviews the simplified approach that designers can use to satisfy these objectives and the results of loss-assessment that have been used to inform the LDSD loss targets.

Introduction

The 2010-2011 Canterbury earthquake sequence caused significant damage to many modern buildings in Christchurch (Kam et al., 2011). In addition, the M_w 7.8 Kaikōura earthquake, which occurred on November 14th, 2016, caused significant damage to numerous buildings in Wellington, despite the epicentre being some 200 km away to the southwest (Cubrinovski et al. 2020). In the immediate aftermath of the earthquake about 11% (1,800,000 sq. ft.) of the city's office space was closed for assessment and many office buildings had to be demolished (Elwood et al., 2020). In light of such events, the industry recognized that a life safety focus is insufficient to meet community expectations and communication about the likely performance of buildings in earthquakes should be improved (Kam et al. 2011). Recognizing this, Engineering NZ and Structural Engineering Society New Zealand (SESOC), in partnership with the New Zealand Ministry of Business, Innovation and Employment (MBIE), established a Low Damage Seismic Design (LDSD) project in 2019. This project built on ideas and concepts for low-damage design that were circulating in New Zealand at the time (Hare et al. 2012).

A key objective of the LDSD project is to deliver buildings that are less likely to be damaged and thereby limit the disruption and losses future earthquakes cause. The project also aims to develop clear guidance material to enable the construction of low-damage buildings. The project is now nearing completion. This paper provides an overview of the three volumes that are due to be released in the coming year, and will describe benchmarking activities and findings that have been undertaken to support the guidelines.

Overview of the low-damage seismic design (LDSD) guidelines

The reference to buildings as being “low-damage” appears to have emerged in New Zealand around the time of the 2010-2011 Canterbury earthquakes. The name appears to recognize that earthquake damage may not be completely preventable, given the uncertainties in earthquake shaking and building performance, but a well-designed “low-damage” building should be less likely to experience damage than

a standard building. Following the Canterbury earthquakes, there also appeared to be a range of views about what a “low-damage” building was, with some engineers claiming a building was low-damage considering only the structural system and others recognizing that a whole-of-building approach should be advocated. To address such variability in the industry, which risked creating more communication issues, it was decided that LDSD guidelines should be developed to provide more consistency.

The outputs of the LDSD project will primarily consist of the three following documents:

- Volume 1: Low Damage Seismic Design: Benefits, Options and Getting Started
- Volume 2: Low Damage Seismic Design: Performance Framework
- Volume 3: Low Damage Seismic Design: Technical Guidance

A summary of the proposed content for the three volumes is summarized in the following sections. It is emphasized the content is still under development, has yet to be peer reviewed, and maybe subject to change. However, recognizing the on-going international efforts to develop similar guidance, it is hoped that sharing this draft content will be beneficial.

Volume 1 LDSD Client Guidance. Volume 1 is primarily being developed for building owners, tenants and users. The document will be written in plain English. It will provide a non-technical overview of LDSD and why it is important in terms of minimizing disruption, the sustainability benefits LDSD will provide and related improved returns for building owners. Volume 1 will draw upon some content from FEMA P58-7 (FEMA, 2018). Volume 1 will also include a briefing guide building owners, tenants or users can use to determine what building performance they need and a client briefing template to assist communicating this information to other parties i.e. design teams.

Volume 2 LDSD Performance Framework. Volume 2 is primarily developed for the design team, project managers, contractors and informed clients. The document intends to encourage non-technical audiences to better understand the LDSD performance framework. It includes an overview of the New Zealand regulatory environment as it relates to new buildings and sets out a framework to support LDSD.

Volume 2 sets out three proposed “outcome objectives” for buildings subject to LDSD as: (i) Life Safety; a low probability of loss of life or significant injuries (addressed in the same way as standard buildings), (ii) Damage Control; a low probability of damage leading to significant economic loss, and (iii) Improved functionality; by reducing the probability of extensive, the time to return to an operational state is likely to be significantly lower than a standard, code compliant building. Post-earthquake regulatory compliance can be impacted by damage to secondary and non-structural elements. While post-earthquake compliance can be seen as an additional outcome objective, it is not a specific consideration of the LDSD performance framework. However, it is considered that low damage solutions should be more easily returned to a compliant state.

Damage Control Performance Goal

Three building categories are provided for in the LDSD guidelines: Level 1, Level 2 and Level 3. LDSD Category Level 2 and 3 will require consideration of a new Damage Control Limit State (DCLS). In contrast, it is proposed LDSD category Level 1 buildings will not require consideration of the new DCLS. While LDSD Category Level 1 buildings will not have specified damage control performance goals it is expected they will be better than current code. Specifically, Level 1 buildings might be categorized as ‘thoughtful’ code compliant buildings, which would include design recommendations such as:

- More stringent limits on vertical and horizontal irregularities.
- Capacity design requirements for the primary lateral load resisting system

- More restrictive drift and/or system ductility demand limits.
- Additional construction monitoring requirements for structural and non-structural elements.

The performance goals proposed in relation to damage control for LDS Category Level 2 and 3 buildings are summarized in Table 1 below. These goals are to limit the expected annual loss (EAL) for the building and building elements, and limit the probability of building damage exceeding 5% of the building replacement cost in an earthquake (expressed over a 10-year period). The second half of this paper will describe the benchmarking study that has been undertaken to inform the relative loss values will be described.

Table 1. Proposed damage control performance goals

<i>LDS category</i>	<i>Expected Annual Loss (EAL)</i>	<i>Probability of exceeding 5% replacement cost in an earthquake over a 10 year period</i>
Level 2	0.15%	5%
Level 3	0.05%	2%

Table 2 summarizes the proposed annual probabilities of exceedance (APoEs) to be considered for LDS Category Level 2 and 3 buildings. Table 2 also details the APoEs for two other building performance limit states defined in the NZBC:

- Serviceability Limit State (SLS1) – this limit state aims to avoid damage that would prevent a building from being used as intended without repair.
- Ultimate Limit State (ULS) – this limit state is primarily concerned about maintaining overall structural integrity and gravity load support with a focus is on life safety. The building may be damaged beyond economic to repair.

Table 2. Proposed annual probabilities of exceedance for LDS Category Level 2 and 3 buildings

<i>Limit State*</i>	<i>Annual Probability of Exceedance</i>		
	<i>Code Minimum</i>	<i>LDS Level 2</i>	<i>LDS Level 3</i>
SLS1	1/25	1/50	1/50
DCLS	N/A	1/250	1/500
ULS¹	1/500	1/500	1/500

*ULS APoEs detailed in Table 2 are for conventional Importance Level 2 buildings. Importance Level 3 and 4 buildings would be designed using ULS APoEs of 1/1000 and 1/2500 respectively.

As detailed in Table 2, it is also proposed that the APoE for SLS1 will be reduced from 1/25 to 1/50 for LDS Category Level 1, 2 and 3 buildings. Reducing the APoE for SLS1 delays the onset of damage in smaller events and increases the probability a building will be in a repairable state following a damaging event beyond SLS1 (Pettinga et. al. 2019). The reduced APoE is also consistent with that used for new building design in Japan and performance-based design in the United States (PEER, 2017).

Improved Functionality Performance Goal

Reducing the time to return a building to an operational state is enhanced by reducing the probability of damage. This indirect observation recognizes it is difficult to accurately assess the time required to return a building back to an operational state following an earthquake, as it can be affected by a large number of variables. Many of these variables are outside of the control of the project design team and/or client (e.g. cordons, utility outages or labor shortages). However, it is proposed that critical non-structural elements and systems necessary for an operational state would be identified by the project team. A strategy would

be established for each of the critical non-structural elements and systems to enable the specified operational state to be returned within the acceptable timeframe. Such a strategies could include:

1. Limiting damage to the component to extent that it could likely continue to function as necessary to enable the operational state, or
2. Identifying alternative short term remedial strategies so that these can be implemented while repairs to the damaged component are completed, or
3. Identifying and defining a repair strategy for the component so that it could likely be repaired within an acceptable timeframe.

Volume 3 LDSD Technical Guidance. Volume 3 is primarily being developed for the design team and will contain recommended technical design provisions to be adopted by design team for LDSD projects. Two verification pathways will be available to demonstrate that a building will comply with the LDSD Performance Goals:

- Prescriptive Design Method
- Direct Design Method

The Direct Design Method enables design teams to demonstrate compliance via application of recognized performance assessment procedures and guidelines, such as the FEMA P58 (ATC, 2018) approach. However, it is expected that most buildings will be designed via the Prescriptive Design Method. This procedure can be used for buildings that satisfy certain size and regularity restrictions that contain elements, systems or components where performance is predictable, and the design is covered by the NZBC through an acceptable solution or verification method. It is envisaged that most modest-sized buildings will fit into this category and the design effort required to demonstrate compliance with the LDSD Performance Goals is comparable to that associated with designing a conventional building.

In addition to ULS and SLS1 design requirements for standard buildings (but with SLS1 design checks conducted for an APoE of 1/50years), buildings designed according to the Prescriptive Design Method will need to verify a new damage-control limit state (with APoE set in Table 2) for the following criteria:

- Peak storey drift demand at DCLS $\leq 0.5\%$
- Design ductility demand at DCLS ≤ 2.0
- Acceleration-sensitive parts and components verified for DCLS intensity level (as opposed to the SLS1 intensity level for parts and components in normal buildings).

Provided the design has satisfied the criteria above (together with regularity and general design requirements and the SLS1 and ULS checks), then it can be considered the LDSD Performance Goals in Volume 2 have been met. The next section describes the benchmarking work undertaken to support this Prescriptive Design Method.

Benchmarking the performance of low-damage buildings

To benchmark the performance of LDSD buildings, a set of 4-storey case-study buildings were designed in line with the draft LDSD criteria and subsequently assessed in line with the FEMA P-58 approach. The hypothetical case study buildings were located in the three main centers in New Zealand: Auckland, Christchurch and Wellington. The seismicity of these three cities could be described as low, high and very-high respectively. All buildings have the same footprint which is 24 m by 40 m, with a storey height of 3.6m, except at the ground storey which was 4.5m high, as shown in Figure 1.

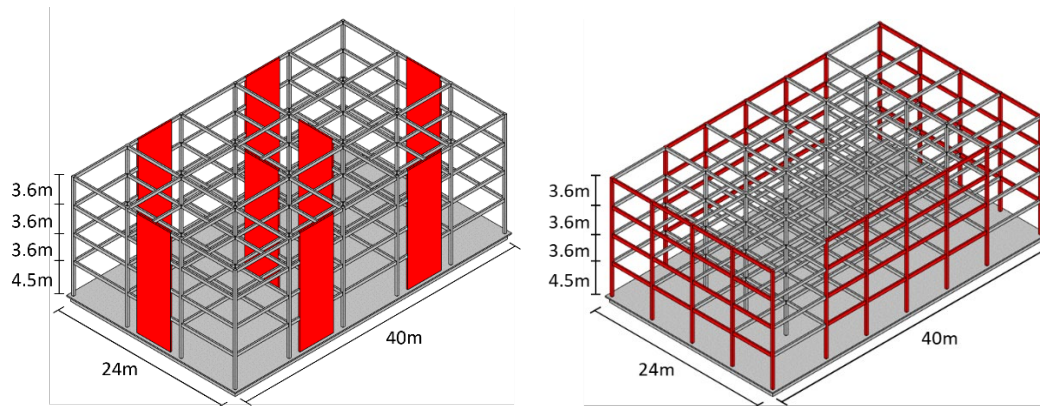


Figure 1. Cantilever RC wall structures (left) and steel MRF structures (right) were examined as case-study buildings in Wellington, Christchurch and Auckland (adapted from Dong et al. 2024).

Two lateral load resisting systems are examined here: steel moment resisting frame (MRF) systems and cantilever reinforced concrete (RC) wall systems. The number and thickness of walls and the number of MRFs was varied in order to satisfy the design criteria. The steel MRFs were designed with reduced beam sections (RBS) in line with common New Zealand practice. The RC walls were designed assuming ductile detailing (with well confined boundary elements and sufficient transverse reinforcement to protect against shear failures). The gravity structural system is not specifically designed as it is assumed to provide negligible lateral resistance. However, gravity columns are included in the models so that P-delta actions could be introduced (and the column nodes were also used to distribute the seismic mass across the floor). Foundation flexibility is accounted for in a simplified manner (linear rotational springs) but non-linear soil-structure interaction effects are not included in the design or later performance assessment.

The buildings were designed using the equivalent static method in accordance with NZ draft technical standard TS1170.5 (2024), together with the additional LDS criteria defined earlier. Building periods range from 0.3s to 1.4s. The structural material quantities required for each resulting lateral load resisting system are shown in Figure 2. For additional details of the design and results see Sullivan et al. (2025).

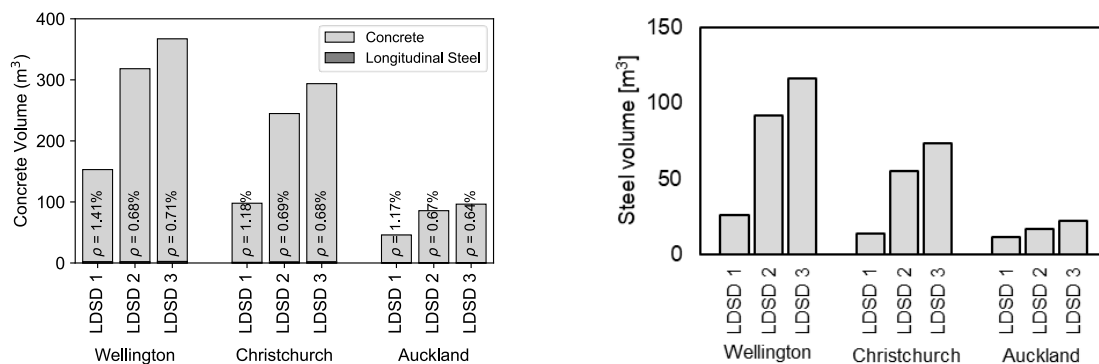


Figure 2. Material quantities (volume of concrete and steel) required for the cantilever RC wall structures (left) and steel MRF structures (right) obtained for the different LDS buildings.

To gauge the likely performance of the case-study buildings, numerical models were developed for NLTH analysis in OPENSEES. The steel frames were modelled with lumped-plasticity frame elements, characterized with modified Ibarra-Medina-Krawinkler deterioration models. The RC walls were

modelled with lumped-plasticity frame elements characterized with Takeda thin hysteretic models and a 5% strain hardening ratio. The strengths of potential plastic hinges were modelled with expected strengths. Viscous damping was modelled with a Rayleigh damping model with 5% damping placed on the 1st and 3rd modes of vibration in the NS direction. Additional details of the modelling and NLTH analysis procedure are described in Sullivan et al. (2025).

Multi-stripe analyses were conducted, using 20 pairs of ground motions selected and scaled (by Yeow et al. 2018) to be hazard consistent at nine intensity levels. Collapse was deemed to occur if the wall roof displacement exceeds the estimated mean drift capacity, assessed according to Abdullah and Wallace (2018), or the MRF's beam plastic hinge rotation exceeds a median value of 4.7%. The peak storey drift demands and peak floor acceleration demands at each intensity were used as inputs to PACT for loss assessment.

For the loss assessment, an inventory of damage components was developed from the work of Dong et al. (2024) with adjustments for LDSD Sullivan et al. (2025). The component fragility and repair cost functions were taken from New Zealand literature where possible. However, the fragility for much of the mechanical and electrical equipment was taken from the FEMA P-58 database. The median capacity for damage states related to bracing of acceleration-sensitive components was assumed to be proportional to the design intensity level. Consequently, for such elements, the median capacity was scaled in proportion to the PGA at the DCLS intensity level. A threshold loss value of 0.5 was adopted as part of the loss assessment in PACT. For additional details on the performance assessment, refer to Sullivan et al. (2025).

Figure 3 shows the EAL values, representing the average annual cost of repair (due to direct losses), obtained from the performance assessment. It is seen that the likely damage and losses are reduced, but the amount of reduction does depend on the structural system and the local seismicity. The figures also show the contribution that acceleration-sensitive non-structural elements and drift-sensitive structural and non-structural elements make to the EAL. As the Prescriptive Design Method was found to be relatively ineffective in Wellington, the EAL obtained for a base-isolated MRF building, designed and assessed by Dong et al. (2024), are also shown.

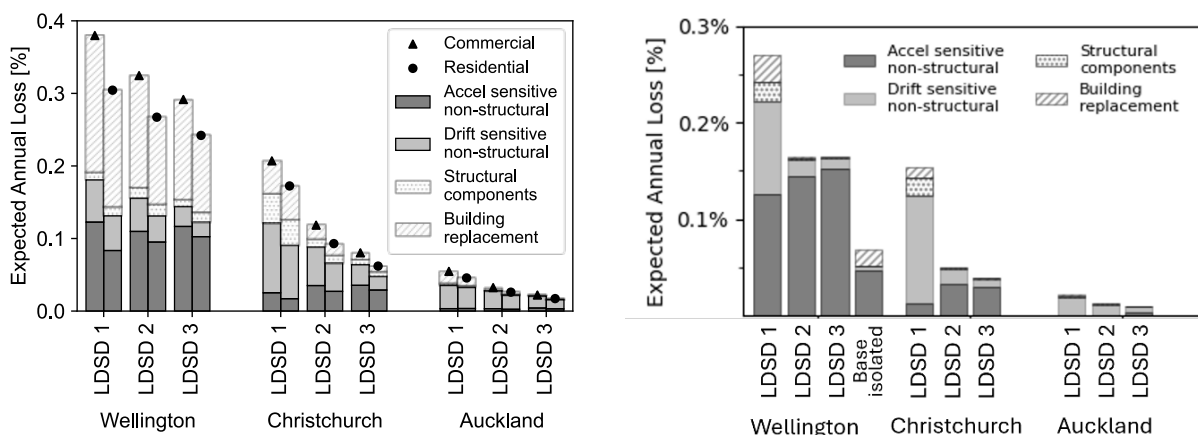


Figure 3. Expected annual loss (EAL) values obtained for the cantilever RC wall structures (left) and steel MRF structures (right) obtained for the different LDSD buildings.

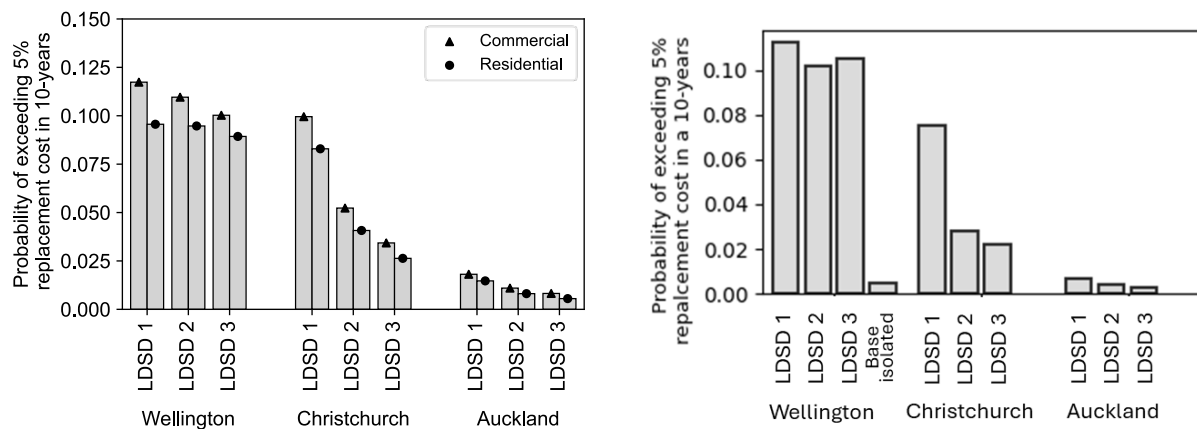


Figure 4. Probability of earthquake repairs exceeding 5% of replacement cost over a 10-year period for the RC wall structures (left) and steel MRF structures (right) obtained for the different LDS categories.

Conclusions

LDSD guidelines are being prepared in New Zealand to provide the industry with a seismic design approach that improves building performance; limiting a building's expected annual losses and likelihood of damage in future earthquakes. The guidelines include three volumes: the first for developers, owners, and tenants to motivate LDSD, the second is for engineers and architects, setting out performance objectives for LDSD, and the provides detailed technical guidance on how to achieve a low-damage design. Two verification pathways will be available to comply with the LDSD Performance Goals: a Direct Design Method and a Prescriptive Design Method.

The Prescriptive Design Method has been gauged via a benchmarking study in which a number of case study buildings in different parts of New Zealand have been designed according to the LDSD guidelines and then assessed in line with the FEMA P-58 performance assessment framework. Comparing the performance assessment results it is generally observed that the Prescriptive Design Method successfully reduces the likelihood of damage and losses. The EAL values appear to reduce by a factor of 1.5 to 3.0 depending on the local seismicity. In regions of low-seismicity, such as Auckland, it would appear that there is no significant benefit of designing for LDSD category Levels 2 and 3, with Level 1 being sufficient. In contrast, in regions of very high seismicity, such as Wellington, the introduction of tighter drift limits led to the design of stiffer buildings with larger peak floor acceleration demands. The damage to the acceleration-sensitive components in the steel MRF structures in Wellington almost completely offset the reduced damage and losses to drift-sensitive components. To address this issue, the LDSD team are currently investigating the introduction of PFA limits as a means of achieving desirable performance via the Prescriptive Design Method. Another alternative would be to require the specification of mechanical and electrical equipment that has undergone qualification testing to show that larger acceleration demands can be resisted without damage. Finally, the results also demonstrate the benefits of using base-isolation in regions of very high seismicity, as an alternative approach.

Acknowledgements

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DEVELOPMENT OF ADVANCED STRUCTURAL HEALTH MONITORING SYSTEM

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Abstract

In Japan, there is growing interest in determining the safety of buildings after earthquakes due to the increasing risk of a major earthquake and the damage caused by recent major earthquakes, such as the Noto Peninsula earthquake in 2024. A rising number of buildings, especially large-scale facilities, are adopting structural health monitoring (SHM) systems that immediately determine whether or not users need to be evacuated after an earthquake. However, it is now known that damage accumulates in buildings due to multiple major earthquakes, such as the Noto Peninsula earthquake in 2024, where repeated major earthquakes are concentrated in a short period, and the foreshocks and aftershocks seen in the Kumamoto earthquake in 2016, although such multiple severe earthquakes are not considered in the current building code in Japan. In a typical SHM system, accelerometers are used to estimate the maximum inter-story drift angle and to determine only the overall damage level to the building. In addition to such general SHM systems, we have developed a system that directly monitors the damage that accumulates in the components of a building over a long period using several types of strain gauges, etc., and uses the collected data to support early restoration when damage occurs in the event of a severe earthquake.

Introduction

In recent years, Japan has repeatedly experienced extreme earthquakes equivalent to or exceeding the 500-year return period specified in building regulations. Building regulations in Japan focus on protecting human life in the event of a severe earthquake but do not clearly specify the required performance of operational, immediate occupancy, and the securing of property in a building. In addition, there is no certainty in protecting human life in the event of serial severe earthquakes.

The current Japanese seismic design methodology, based on the 1980 regulations, needs to catch up with the subsequent economic development and urban densification. There is a clear need for building performance design that considers damage from repeated severe earthquakes and the accumulation of such damage. Moreover, economically developed societies require comprehensive earthquake countermeasures, including immediate emergency response and post-earthquake damage recovery. In order to respond to such social needs, Nikken Sekkei and The University of Tokyo are developing a structural monitoring system (hereafter referred to as “SHM”) that enables not only a rough understanding of building damage immediately after an earthquake and the necessity of evacuation but also damage assessment of each structural components to assist with early building repairs and recovery. In this report, we describe the development status of SHM for accurate damage assessment.

Pervasion of SHM in Japan

SHM based on accelerometers. SHM has been increasingly introduced in Japan after each major earthquake, such as the 2011 Great East Japan Earthquake and the 2016 Kumamoto earthquakes. Existing SHM utilizes floor response acceleration measurements to understand the entire building's response behavior and vibration properties and estimates damages in structural and non-structural components. SHM based on acceleration measurements has already proven useful in measuring the degree of damage

on each floor of a building. However, to confirm detailed damage conditions at the structural element level, it is necessary to "presume" the response of individual members by combining the measurement results with structural analysis. Especially in the case of steel structures where all the structural members are covered by finishes or fireproofing, even though a damage was presumed by a SHM system, still it is necessary to look for the exact damaged elements by removing the finishes and visual inspection. There are limitations in evaluating the performance and damage of individual structural members only by measuring response acceleration.

SHM with strain gauges. In Japan, strain gauges and optical fiber sensors have been used to measure strain in steel bridges, where SHM has been widely applied. The measured data has been used for traffic load measurement and for detecting member damage by capturing changes in strain and stress distribution. On the other hand, there are very few examples of strain measurement in buildings. The reasons for the lack of actual strain measurement in building structures are that the strains in structural elements daily are extremely small and difficult to measure accurately, that dynamic strain measurement devices are expensive, that the measurement of very localized strains is easily affected by the location of the attachment and the surrounding environment, and that, as a result of these reasons, measurement and analysis methods have not yet been established. However, in terms of measurement accuracy, it is possible to measure extremely small strains at the level of constant microtremors using high-precision semiconductor strain gauges. Regarding cost, it is reasonable to expect IT/IoT technology to reduce costs. If it is possible to measure response strain and the resulting stress in addition to response acceleration, it will be possible to grasp the stress of structural members quantitatively. Then, comparing the measured stress and the structural calculation results can verify the validity of structural calculations and the reliability of construction. In addition, it is possible to detect stress imbalances due to damage from earthquakes and other causes. The authors aim to establish a technique to quantitatively determine the location and extent of damage by measuring the strain in each member of a steel structural building. We verified this technique in the “Dynamic Characterization of Buildings Using a 10-Story Office Specimen” experiment. Figure 1 shows examples of the behavior and damage that can be detected by this measurement methodology.

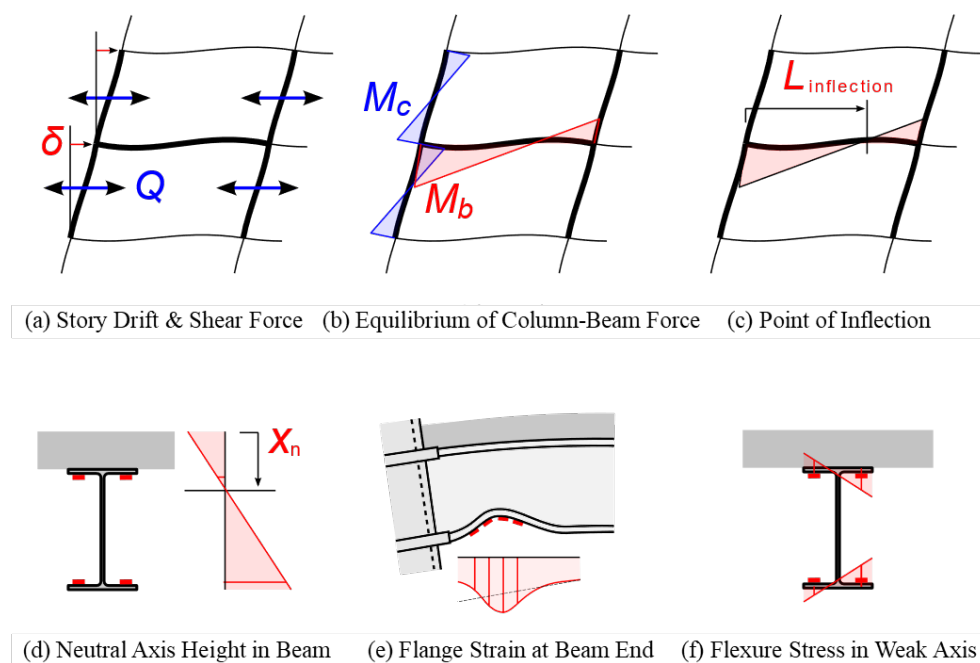


Figure 1. Behavior and damage that can be detected by strain measurement

Example of Acceleration and Strain Measurement in an Actual Building

Outline of the target building. In this report, an outline of measurement and analysis of the data measured in the Nikken Sekkei Tokyo Building (Figures 2-5, constructed in Chiyoda-ku, Tokyo) is introduced (Sano et al. 2024, and Yamahachi et al. 2024). This building was completed in March 2003 and is a steel structure of 14 stories with one basement floor adopting a vibration control structure. The columns are concrete-filled tubes, CFT, and measuring the actual stresses in the columns is difficult. However, since the stresses in the steel beams can be measured, comparing the measured stress and calculation results is expected to make it possible to judge damages that occurred in the surrounding area when the balance of these stresses changes. In this structural calculation, the column stiffness, including the filled concrete, is determined using the Euler-Bernoulli beam theory. The bending stiffness of the beam is assumed to be 1.0 to 1.65 times that of the original steel beam section as per the composite effect with the slab. For the target beam, the value of 1.65 times is used. The first natural frequency in the Y direction estimated by the floor acceleration record in past earthquakes is 0.7Hz.



Figure 2. Object Building

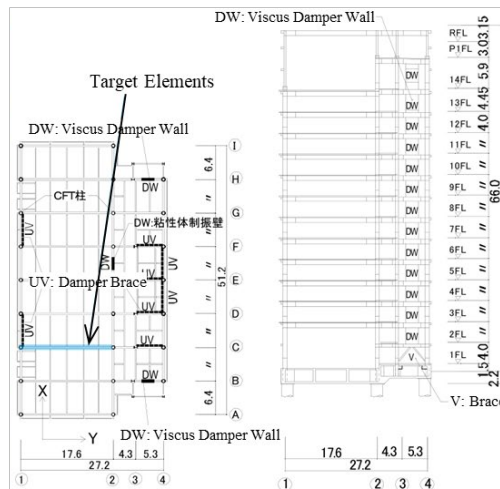


Figure 3. Plan



Figure 4. Section Grid-B

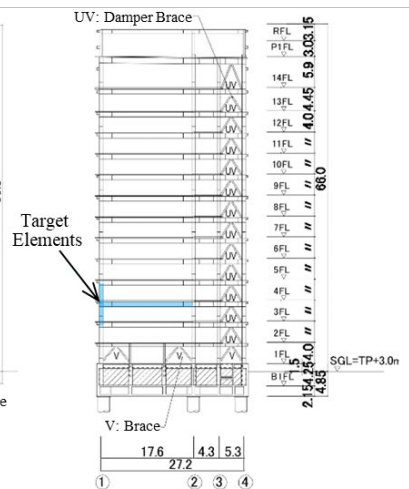


Figure 5. Section Grid-C

Outline of the measurement. The measurements were commenced in April 2023. We installed gauges on a rotated-T shape frame consisting of a primary beam with a span of 17.6 m on the fourth floor and columns above and below, as shown in Figures 3 and 5. Table 1 and Figure 6 provide an overview of the measurements. MEMS accelerometers were installed at one location on each of the three to five floors to measure floor acceleration with a sampling frequency of 125 Hz. Stresses in the columns and beams were measured using strain gauges. For the columns on the third and fourth floors, strain gauges are mounted on the two sides of opposites of the circular steel tubes at four sections (see photo in Figure 6). High-sensitivity semiconductor strain gauges were mounted beside standard metal foil strain gauges to verify the accuracy of the measurement. For the primary beam on the fourth floor, strain gauges are mounted on the inside of the top and bottom flanges at four sections. At the beam end, strain gauges are mounted on both surfaces, the upper and lower surfaces, of the bottom flange to detect the cause of local buckling, as shown in Figure 1(e). In addition, two displacement meters are installed to measure the beam end rotation angle. These strain gauges and displacement meters are connected to a dynamic strain measurement unit developed by ourselves with a sampling frequency of approximately 85Hz. The acceleration measurement unit and dynamic strain measurement unit are interconnected by wired LAN for time synchronization and data accumulation.

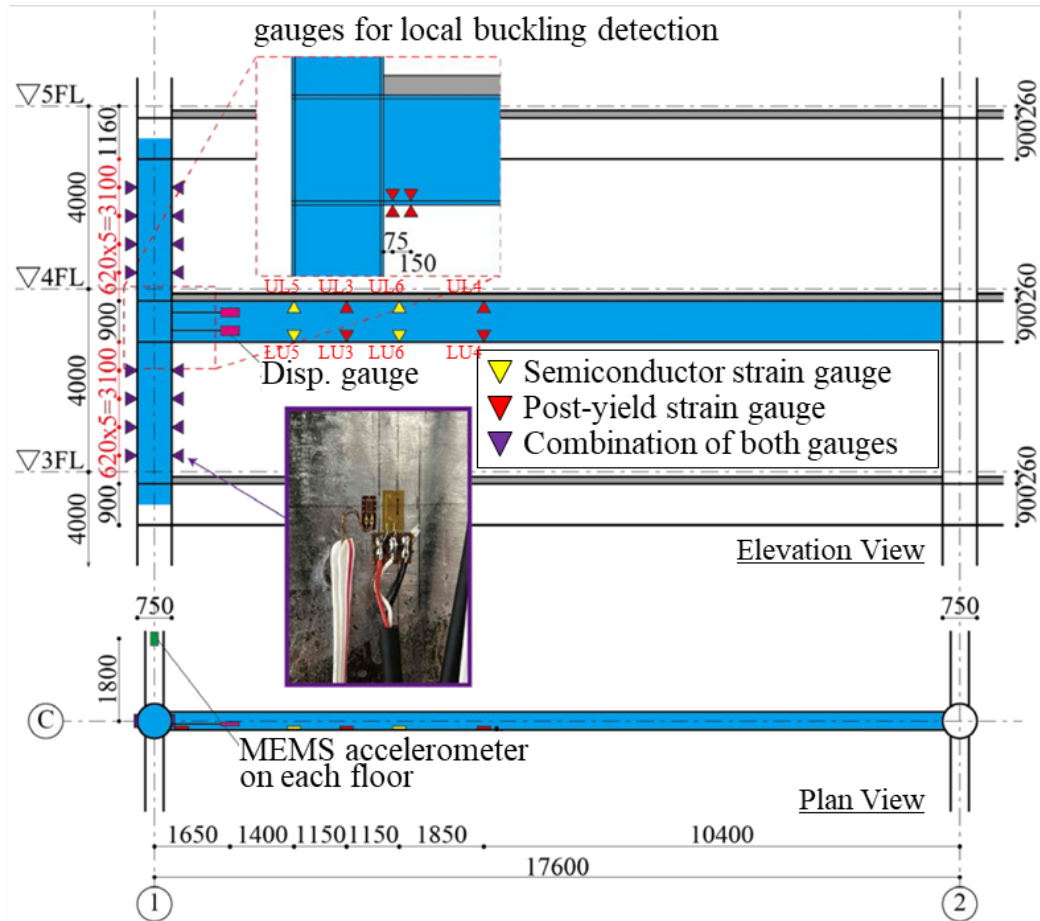


Figure 6. Installed strain gauges, displacement meters and accelerometer

Table 1. The list of sensors

Measurement Items	Sensors	Location	Numbers	Remarks
Floor acceleration	Accelerometer	3F-5F	3	
Stress in column	Strain gauge	3F, 4F	32	16 semiconductor gauges
Stress in beam	Strain gauge	4F	8	4 semiconductor gauges
Local buckling	Strain gauge	4F	4	
Beam rotation angle	Displacement meter	4F	2	

Actual record of ground motions. This paper presents the measurement result to the ground motions shown in Table 2. Record No.1 was not an earthquake, but an environmental vibration of microtremor. Record No.2 was the 2024 Noto Peninsula Earthquake that occurred on January 1, 2024.

Table 2. The list of ground motions

Record	Accrual Date and Time	Intensity (JMA Scale)
No.1	2024/01/01 16:01~16:06	(Environmental vibration)
No.2	2024/01/01 16:11~16:20	Intensity 2

Frequency analysis. The building's response acceleration and response strain to the ground motions in Table 2 were measured by MEMS accelerometers and strain gauges, respectively. This report uses only the predominant frequency components where the Fourier spectrum shows a peak as shown in Figure 7. Variables denoted with "^" in the following sections refer to its Fourier transform. (f_{peak}) denotes the predominant frequency. The specific measurement results are shown in the next section. Since strain gauges were more sensitive than accelerometers in this measurement system, (f_{peak}) was determined by the peak frequency of the strain gauge.

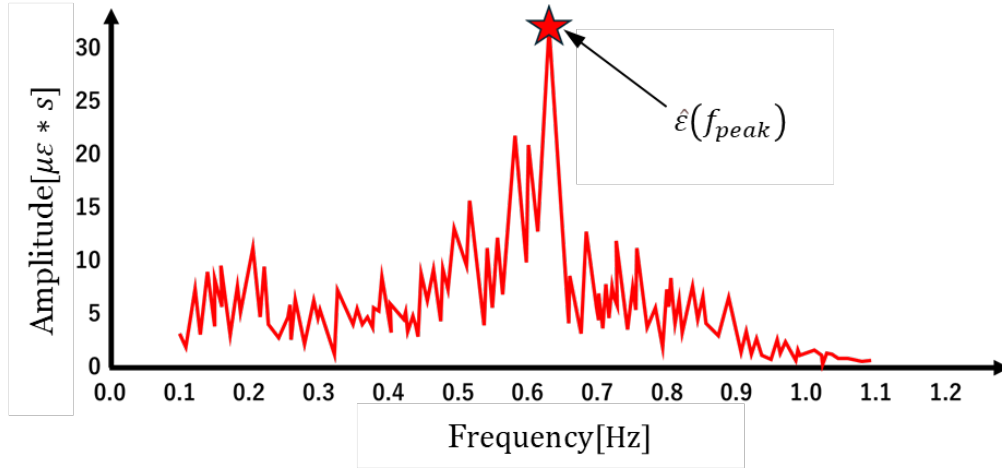


Figure 7. Fourier transform results and peak values in the frequency domain (strain)

Calculation method of section forces. From the strain gauge measurements, the axial force and bending moment of a structural element at the predominant frequency can be obtained by equations (1) and (2), respectively. ε_t and ε_b are the measured values from the two strain gauges facing each other on the column and the beam, respectively. Also, E represents Young's modulus, A represents the cross-sectional area, and Z represents the section modulus of the member.

$$\text{Axial Force: } \hat{N}(f_{peak}) = EA \frac{\hat{\varepsilon}_t(f_{peak}) + \hat{\varepsilon}_b(f_{peak})}{2} \quad (1)$$

$$\text{Bending Moment: } \hat{M}(f_{peak}) = EZ \frac{\hat{\varepsilon}_t(f_{peak}) - \hat{\varepsilon}_b(f_{peak})}{2} \quad (2)$$

As mentioned above, the column is a CFT, concrete-filled tube with circular steel tubes. It is impossible to determine the bending moment as a CFT column accurately, but for convenience, calculations were made using the Euler–Bernoulli beam theory for the interior concrete. In Equation (3), the subscripts "s" and "c" refer to the physical properties of the steel and the concrete, respectively.

$$\hat{M}_{cft}(f_{peak}) = (E_s Z_s + E_c Z_c) \frac{\hat{\varepsilon}_t(f_{peak}) - \hat{\varepsilon}_b(f_{peak})}{2} \quad (3)$$

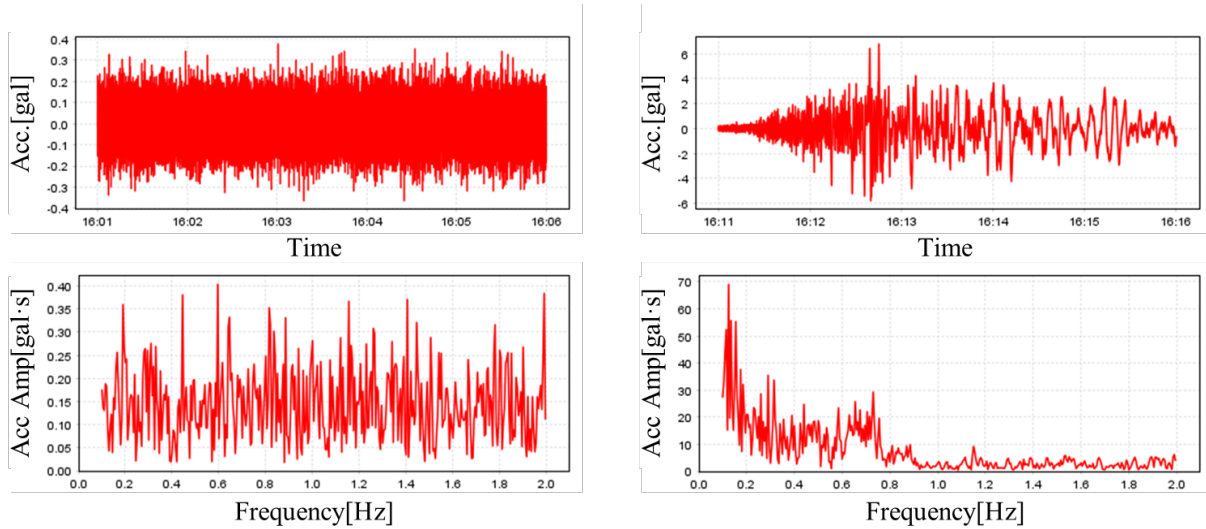
Considering the floor concrete slab on the steel beam, the bending moment of the composite beam was calculated as Equation (4) using the Euler–Bernoulli beam theory for the concrete section. Here, $\hat{M}_s$ in Equation (4) represents the bending moment of the steel beam calculated from Equation (2), $\hat{N}_s$ represents the axial force of the steel beam calculated from Equation (1), and d represents the distance between the centroids of steel beam section and concrete slab.

$$\hat{M}_{beam}(f_{peak}) = \hat{M}_S(f_{peak}) + \hat{N}_S(f_{peak}) \times d \quad (4)$$

Calculation method of story drift. The Fourier transform of the average of the inter-story displacement between the 3rd floor and 5th floor is calculated from the acceleration record $\hat{\alpha}_{5F}(f_{peak})$ of the 5th floor and the acceleration record $\hat{\alpha}_{3F}(f_{peak})$ of the 3rd floor using equation (5).

$$\text{Inter-story Drift: } \hat{d}(f_{peak}) = -\frac{\hat{\alpha}_{5F}(f_{peak}) - \hat{\alpha}_{3F}(f_{peak})}{2 \times 4\pi^2 (f_{peak})^2} \quad (5)$$

Measured response acceleration. Figure 8 shows acceleration response record, $\hat{\alpha}_{4F}(t)$, and Fourier amplitude spectrum, $\hat{\alpha}_{4F}(f)$, in the beam direction at the fourth floor for record No. 1 (microtremor) and record No. 2 (seismic intensity 2). The time history record and its Fourier amplitude spectrum show the typical waveforms for earthquake ground motions, which indicates that our system could capture the response. On the other hand, the ground motion of Record No. 1 shows no peak in the spectrum, indicating that the accuracy of the MEMS sensor is insufficient.

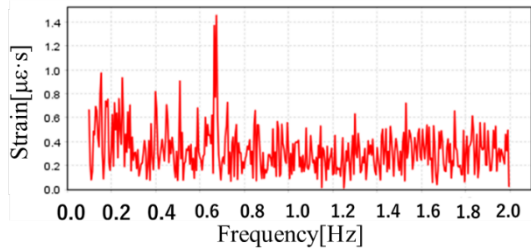
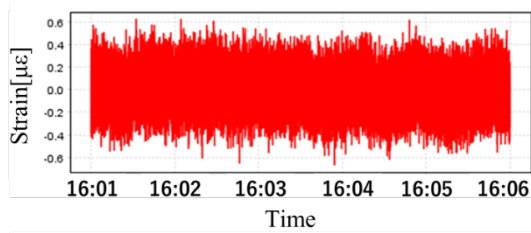


(a) Acceleration of Record No.1

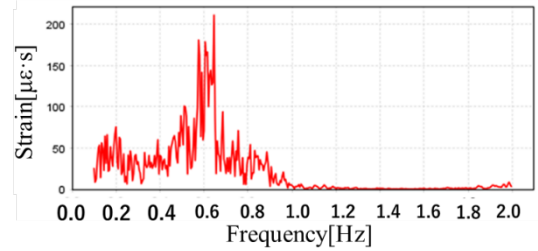
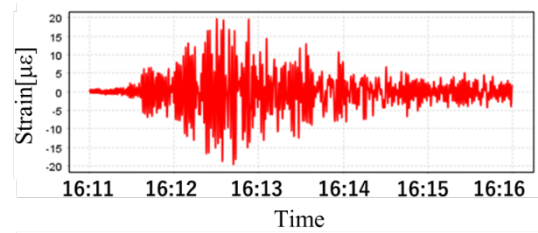
(b) Acceleration of Record No.2

Figure 8. Recorded acceleration on 4F

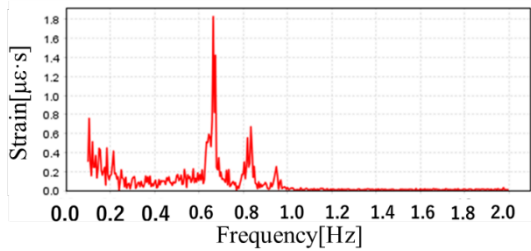
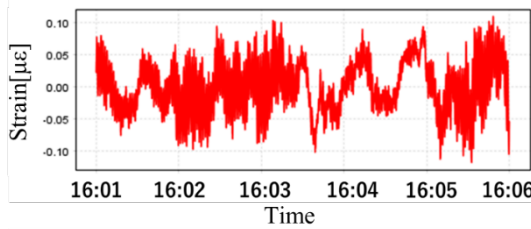
Measured response strain. Figure 9 shows the strain response records at the bottom flange of the beam. In Figure 9(a)~(d), the upper graph shows the time history, and the lower graph shows the Fourier amplitude spectrum. Figures 9(a) and 9(b) are records of metal foil strain gauges attached at the LU3 position. In the Fourier amplitude spectrum of both Record No. 2 (seismic intensity 2) and Record No. 1 (microtremor), we can observe a clear peak around 0.65Hz, which is close to the natural frequency of the frame. These results indicate that strain vibration was well detected. On the other hand, Figures 9(c) and 10(d) show records of semiconductor strain gauges attached at the LU5 position. Figure 9(c) shows that the strain amplitude under microtremors is significantly small, around 0.10μ. However, the Fourier amplitude spectrum shows a clear peak, indicating that the dynamic strain response corresponding to the natural frequency of the structure is captured. Figure 9(d) also shows that the spectral shape is similar to Figure 9(b), indicating that good measurement accuracy was achieved.



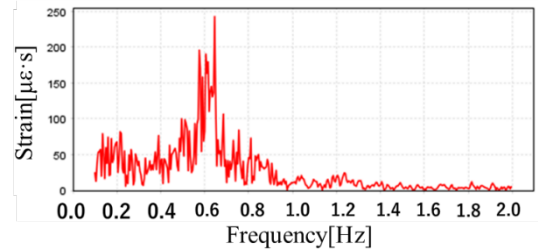
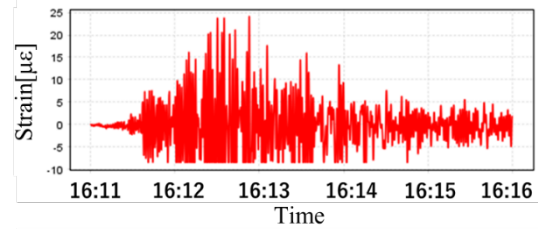
(a) Metal foil strain gauge, Record No.1



(b) Metal foil strain gauge, Record No.2



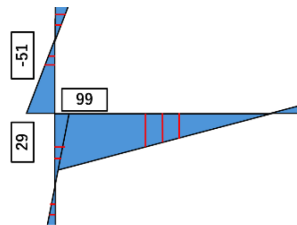
(c) Semiconductor strain gauge, Record No.1



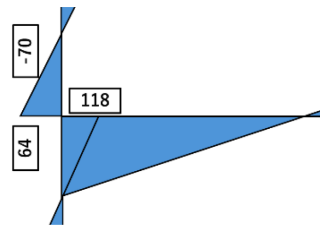
(d) Semiconductor strain gauge, Record No.2

Figure 9. Recorded of strain response at the bottom flange of 4F beam

Bending moment diagram. Figure 10(a) shows the bending moment distribution calculated from Record No. 2, and Figure 10(b) shows that obtained from the static structural analysis.



(a) Result from Record No.2



(b) Result from structural analysis

Figure 10. Local bending stiffness diagram ($kN \cdot m/mm$)

The value shown here is the bending moment calculated using Equation (6) when the average inter-story displacement between the third and fifth floors reaches 1 mm.

$$\text{Local bending stiffness: } K = \operatorname{Re} \left| \frac{\hat{M}(f_{peak})}{\hat{d}(f_{peak})} \right| \quad (6)$$

The reason for dividing the bending moment $\hat{M}$ by the deformation $\hat{d}$ is to eliminate the effect of the magnitude of the deformation and to normalize it. In addition, although $\hat{M}$ and $\hat{d}$ are both complex numbers, they are ideally in phase, so $\hat{M}/\hat{d}$ has only real components. In reality, there is a slight phase difference, but here we ignore the imaginary components and evaluate it as only real components. Figure 10(a) shows the distribution of K calculated by substituting $\hat{M}_{cft}$ calculated in Equation (3) from the measurement results of Record No. 2 into $\hat{M}$ in Equation (6). On the other hand, Figure 10(b) shows the distribution of K obtained from the static structural analysis. The values in Figures 10(a) and 10(b) are approximately the same, indicating that this measurement method is consistent with the analytical results to a certain degree of accuracy. In Figure 10(b), the sum of the bending moments of the columns above and below the joint is naturally balanced by the bending moment at the beam end. However, in Figure 10(a), the sum of the bending moments of the columns is disproportionate to that of the beam, which is about 80%. This difference may be due to the difference between the reality and the Euler–Bernoulli beam theory assumption in this report. Overall, the measured and analytical results correspond in some agreement, and the results are generally considered to be reasonable.

Conclusion

This report introduced the development status of a SHM system for steel building structure to accurately assess the structural damage using accelerometers and strain gauges. One example of measurement result in actual ground motions was presented, and it was shown that a reasonable record was observed comparable to a static structural analysis result. In addition to continuous measurement, the authors are executing further to improve reliability.

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