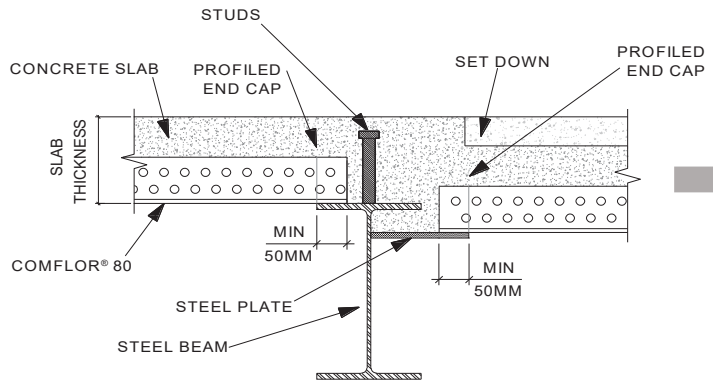

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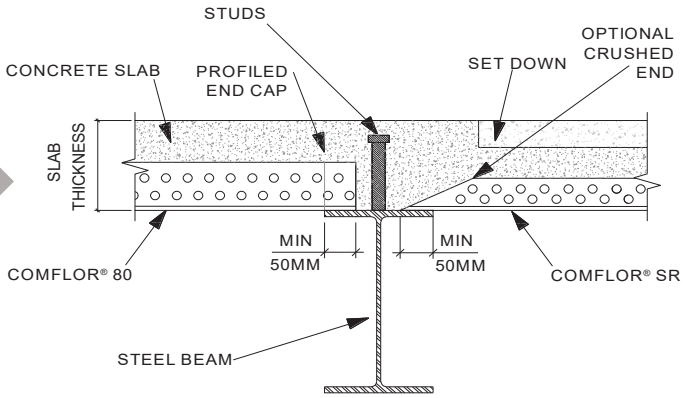
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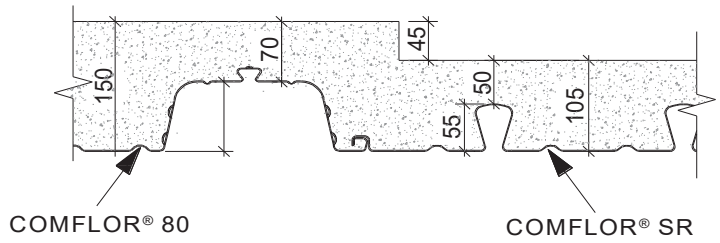


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Structural Engineering Society New Zealand (Inc.)
JOURNAL September 2026

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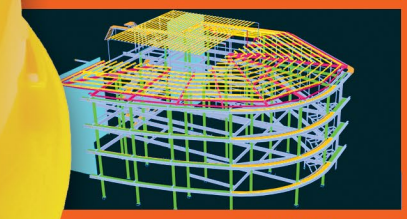














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Please see footnote page 7 regarding submission of papers.

SESOC Membership

Membership of the Structural Engineering Society New Zealand Inc. (a technical group of Engineering New Zealand) is open to any person practising the profession of structural engineering or in any way interested in the discipline.

A membership application form is included at the back of the Journal.

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Ian Billings	2002	David Brunsdon	2016
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Barry Davidson	2009	Charles Clifton	2017
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Contributions to the SESOC Journal

Contributions to the Journal are welcome. Please forward a copy of the manuscript to the Editor at the address above. Procedures for submitting articles, letters to the Editor or any other papers to the Journal can be found on the SESOC website. Please read before compiling your paper for the Journal, as any material that does not follow these guidelines will be returned to the submitter. You will find full guidance on typing and formatting, graphics and photographs, equations, final draft and checking, author qualifications, tone of writing and referencing style. https://www.sesoc.org.nz/library/sesoc-journals. Your help with this is greatly appreciated. Final decisions on any item submitted rests with the Editorial Board. Papers as printed will be available on the SESOC website shortly after publication.

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SESOC
PRESIDENT'S REPORT



It has been another busy period for the Society. Working in collaboration with the Bridge Engineering Technical Society (BETS), we have completed a review and update of the structural engineering practice field definition for Engineering New Zealand. We extend our thanks to everyone who took the time to provide feedback on the proposed updates. Looking ahead, Engineering New Zealand has signalled its intention to begin developing new CPEng Structural Engineering Classes later this year. The primary objective of these classes is to better recognise and regulate high-risk areas of practice, and to provide greater clarity around who is qualified to undertake such work. SESOC supports this initiative and will be working closely with Engineering New Zealand throughout the process to help ensure the new classes are practical, effective, and reflect our members' needs.

In partnership with the Timber Design Society, NZSEE, Engineering General Practitioners, and Engineering New Zealand, we published interim guidance on bracing design for three-storey residential structures. These buildings are outside the scope of the Acceptable Solution cited standard NZS 3604 and require specific engineering design. The interim guidance was developed in response to a recent technical review of townhouse designs undertaken for the Natural Hazards Commission. That review identified a number of significant concerns with current structural design practice, including the inappropriate use of P21-rated bracing elements within specific engineering design, incomplete load paths, and inadequate connection detailing. The interim guidance is intended as a placeholder document while further research and more comprehensive guidance is developed.

In response to the frustrations many of our members have experienced with the public consultation process for DZ NZS 3604 and DZ NZS 3404 Parts 1 and 2, SESOC has held several conversations with Standards New Zealand. This included a joint technical societies letter, a copy of which has been included in this journal, and a subsequent meeting with the National Manager of Standards New Zealand in September. In those discussions, we raised concerns about the limitations of the web tool and associated submission form, and asked whether Standards New Zealand could provide PDF versions of the draft documents, under a non-disclosure agreement if necessary, to help expedite the consultation process. Standards New Zealand advised that this was not possible, citing intellectual property risk, potential loss of sales revenue, and the additional administrative workload it would create.

SESOC's view is that this situation stems primarily from a funding shortfall within MBIE, and that the current process is untenable, placing unreasonable demands on the time of volunteers contributing public feedback. As a result, SESOC advised Standards New Zealand that we would not participate in the public consultation process for DZ NZS 3404. The review of DZ NZS 3604 was largely complete by the time we met with Standards New Zealand, and on balance we considered it to be in our members' best interests to see that process through. Feedback on the proposed changes to that standard was therefore provided. SESOC intends to correspond further with both Standards New Zealand and the Minister for Building and Construction with the aim of improving the public consultation process for future standards.

The Management Committee has also made good progress on governance improvements for the Society. Draft updates to the SESOC Rules have been prepared, covering three key changes: the introduction of term limits for elected and co-opted members of the Management Committee, a shift to a two-year election cycle, and the establishment of a new Software Manager role to oversee our suite of design tools. A Conflict of Interest Policy has also been developed and is now published on our website. Further details of the proposed Rule changes will be circulated with the invitation to the 2027 SESOC AGM, at which they will be put to members for ratification.

SESOC has formed a joint working group with the Concrete New Zealand Learned Society to develop updated guidance on concrete curing. The guidance responds to increased observations of poor concrete curing practice across the industry and the risk that some engineers may be unwittingly exposing themselves to increased liability as a result. The issue is compounded by the growing use of supplementary cementitious materials (SCMs) in concrete mixes. Unlike standard mixes, concrete containing SCMs gains strength more slowly and is therefore more dependent on good curing practice to maintain sufficient moisture for the hydration process to proceed adequately.

Planning for the 2027 SESOC Conference is progressing well. The conference will be held at the Cordis Hotel in Auckland from 30 June to 2 July. The conference website is currently being finalised, and we will be announcing the conference theme and keynote speakers in the coming weeks.

Stuart Oliver
SESOC President
September 2026

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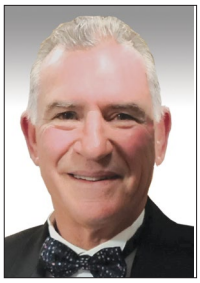


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NOTE FROM THE EDITOR



Few issues have tested confidence in New Zealand's construction industry more severely than leaky buildings. Owners have faced substantial repair costs, prolonged disputes and considerable personal distress. Alongside the technical lessons sits an unresolved question: how should the financial consequences be divided when several participants have contributed to a failure?

The present joint and several liability system gives owners a means of recovering their loss from a liable party that can still pay. Frequently, that party is a council. Where other participants have become insolvent or ceased trading, a council's payment can greatly exceed the proportion of responsibility attributed to it. The difference ultimately becomes a community cost.

The proposed move to proportionate liability would change that arrangement. Each liable participant would meet the portion of the loss allocated to its contribution. This has an understandable appeal. Responsibility for design, construction and regulatory oversight should carry financial consequences that reflect the failures within each role. Ratepayers cannot reasonably be expected to provide unlimited financial backing for private construction activity. I am also a big fan of developers not being able to "walk away".

Yet the argument looks different from the kitchen table of an owner whose house needs extensive repairs. Allocating responsibility among defendants does not ensure that each defendant has the resources to pay. An unrecoverable share could leave the owner unable to complete the work, despite having established a valid claim.

That is the central difficulty. Reducing councils' exposure may improve fairness between defendants, while transferring greater financial uncertainty to owners. Any assessment of the reform must consider both outcomes.

The proposals for home warranties and professional indemnity insurance for designers will consequently require careful examination. The practical questions concern the breadth of protection, exclusions, available funds and the process for obtaining payment. Cover must remain useful when defects emerge years later or when a business has closed. Weathertightness deserves particular attention, given the experience that has brought us to this debate.

Structural engineers also have a direct interest in how the new arrangements develop. Our contribution begins with competent design and extends to clearly defined responsibilities, coordination with other disciplines and construction monitoring appropriate to the engagement. Good records are essential, especially where responsibility passes between designers, contractors and inspectors. Insurance supports these practices; it cannot compensate for their absence.

For our profession, the challenge is to support arrangements that combine fair accountability with practical recovery. Owners need sound buildings and an effective remedy when those buildings fail. Both should remain central as the legislation develops.

PAPER AND LETTER SUBMISSIONS

Please take note of the following close off dates for the submission of papers and correspondence to the Editor:

Journal Issue	**Advice of Pending Paper**	**Material Deadline**
April 2027	Up to end January 2027	20 February 2027
September 2027	Up to end June 2027	20 July 2027
Advertising	**Booking Deadline**	**Material Deadline**
April 2027	30 January 2027	20 March 2027
September 2027	30 June 2027	20 August 2027

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Inside front cover	**$750 + GST**	**Outside back cover**	**$675 + GST**
Inside back cover	**$675 + GST**	**All internal pages A4 advertising**	**$600 + GST**

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Bookings: *Bookings should be made in advance for advertising spots. Some spots such as covers are booked some time in advance. Please book your advertising by the dates noted above.*
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Submissions: *Please note that although we may be advised of a potential paper, this does not necessarily guarantee acceptance into the next journal. Generally, we work on a first in first served basis so please advise us at your earliest convenience of a pending paper. Further, the quality, suitability and relevance of papers is subject to the Editor's discretion as to whether it is accepted for publication. Note that at times we may run a theme which could postpone your paper to the next journal. It is important to recognise that we require your papers in Word format and all embedded photos, graphics and images must be supplied as separate high quality files, preferably as jpeg or high quality pdf images. No embedded hyperlinks should be included in text for submissions.The editor's decisions regarding proofreading and paper amendments are checked by the SESOC Editorial Board and are final.*
All letters submitted to the Journal are to be passed to the SESOC Editorial Board for review.

Letters should meet the following criteria:
- *Content submitted must align with the purposes of the Structural Engineering Society;*
- *Letters must be no more than 800 words;*
- *Letters must not contain offensive or defamatory material;*
- *Letters received as a response to articles need to constructively contribute to the further understanding of the topic.*

Publishing of letters is at the discretion of the SESOC Editorial Board.

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- Tessa Beetham - *Vice President*
- Geoff Bird
- Jenni Tipler
- Adam Langsford
- Carl Ashby
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- John Snook
- Charlotte Toma
- Stuart Oliver
- Tessa Beetham
- Jenni Tipler

Editorial Board
- Nicholas Brooke
- Jason Ingham
- Charlotte Toma
- Laura Whitehurst

LETTERS TO THE EDITOR

Community building through virtual earthquake reconnaissance

As I approach the personal milestone of 10 years in this industry since graduation, I've been reflecting a bit on the importance of our community. Community as a source of encouragement and camaraderie; a space for the cross-pollination of ideas, inspiration and opportunities to give back - along with many other benefits. In essence, it can act as something of a forcing function, helping us all go further and faster than we could on our own.

At the start of my career, I was fortunate to work in superb consultancy environment where we were encouraged to engage in our professional community right from day one and supported with the time and financial resources to do so. Looking around my first industry conference in 2018, I quickly realised that this was not the normal experience for an early-career practitioner. While there were some other younger faces in the room the vast majority were postgraduate researchers and there were very few on my path at my stage.

The intense sense of 'imposter syndrome' I felt as a young practicing engineer at that conference has dissipated over the years, though it does still surface from time to time. That initial feeling has stuck with me and become part of my motivation to improve the way we build our early-career community, and to foster ideas and connection as that initial conference sparked in my own career .

In recent years our community has become increasingly diverse and I think it's so much better for it. Personally, I've had ideas prompted during discussions with recently arrived engineers from Iran, Türkiye, Japan and beyond. This diversity change means that we need to even more deliberately invest effort in community-building. After all, if I - as a pākehā male - felt that intense sense of imposter syndrome at the 2018 conference when the majority of the room looked like older versions of myself, then surely the experience would be even more challenging for young women, for those for whom English is a second language, and for those from adjacent or peripheral professions, among others.

There are many initiatives that have developed in recent years which are helping to build our community and touch the early-career space to varying degrees: the annual seismic design competition, AWEC, the SESOC ESE chapters, various mentoring programmes, and many others. Each of these has required a leap of faith from those who kicked them off, and then a lot of hard work, with the aim – at least in part – of improving inclusivity for early-career professionals. What I didn't expect when I became involved in a wider virtual earthquake reconnaissance group was that this might also prove to do the same.

The importance of earthquake reconnaissance was first identified, from an Aotearoa New Zealand perspective, back in the early 1970s because of the need to learn from real-world events outside the engineering laboratory. Large earthquakes affecting heavily populated areas in our own country are relatively infrequent (when scaled against a professional career), so heading offshore creates more frequent learning opportunities. My great-uncle was part of the mission to Lima, Peru, in 1974, and there have since been many other overseas volunteer missions which have created valuable insights for our profession but these also come at significant financial cost and volunteer time burden.

More than 50 years later, 2026 has proven to be a year of significant earthquakes occurring internationally near densely populated areas. At the time of writing there have been three mobilisations of Kiwi volunteers: the recent Catia La Mar (Venezuela), one in Kumamoto (Japan) and the San José del Palmar (Colombia) earthquake, co-ordinated by NZSEE.

Virtual reconnaissance is still relatively new globally and has been made possible by our increasingly digitally connected world and the amazing tools now available to us. It means many more volunteers can be included in a reconnaissance effort, which can now be delivered with very little direct financial cost beyond volunteers' time. With each mobilisation the processes have improved and efficiency has increased, and there will continue to be various publications around some of these aspects. Virtual reconnaissance is not intended to replace physical reconnaissance but can be used to inform a physical mission and can generally be delivered more efficiently.

Considering the people side of the recent virtual reconnaissance, there have been 42 volunteers involved in one or more of the three mobilisations so far. More than 80% of this group are under 35 years old. So, within the early-career section of our industry, there seems to be a real desire to become involved in efforts like these.

Just as importantly, the volunteer cohort includes a group of extremely experienced engineers – both above and below the 35-year line – who have been able to act as technical sounding boards and bring insights from historic physical reconnaissance missions. Some volunteers who are less experienced from a technical engineering perspective have brought fantastic skillsets in the digital space, which have further propelled the efforts. The collective learning taking place, and the knowledge transfer in both directions, will continue to have a positive effect into the future.

One of the real challenges with virtual reconnaissance, however, is that it is, by definition, 'virtual' not in-person. Every mobilisation sees new connections made online and hours spent working together on a shared task, with various online calls to co-ordinate . Most importantly, people from

other organisations and backgrounds form common ground, from which professional relationships can flourish. But "virtual" can only go so far, and nothing beats meeting face-to-face.

We recently put the word out to the volunteer group about the prospect of an informal gathering one evening during the QuakeCoRE Annual Meeting in Dunedin and managed to get 12 people together. Most of the group – myself included – had only met one or two of the others in person before. But the common ground of earthquake reconnaissance acted as an icebreaker for many wider conversations, which will perhaps be the seed for bigger and better things in the future too.

Within that group of 12, we had nine different countries of birth, a variety of technical backgrounds, ages and passions – and one shared experience of virtual reconnaissance. I think it was perhaps that shared experience that helped make it one of the more natural first meetings of engineers I've been part of.

Throughout the various mobilisations, a consistent theme has been the sense of connectedness from afar for those with ties to the region affected, particularly where friends and family have been impacted. Aotearoa New Zealand can feel very far away at times, and virtual reconnaissance gives members of our expat community an opportunity to contribute to an effort that can have a tangible impact on the ground.

Effective co-ordination of people and resources following a disaster can be a real challenge, and engineers on the ground are often overloaded. In the case of the Colombian event, the Colombian volunteers within our NZSEE team were able to get the virtual reconnaissance dataset into the right hands in Colombia, helping to direct physical response resources more effectively. Website analytics since the event suggest the dataset has also been widely used across Colombia.

I hope this piece can serve as an encouragement to the industry to keep searching for new ways to enhance the inclusivity of our community, and as a reminder that sometimes this can happen as an unexpected byproduct of other efforts too. Creating opportunities for early-career professionals to get involved in a variety of ways builds crucial skills, confidence, and connections which can then propagate outwards and compound over many decades of a career.

Looking around the room that evening in Dunedin, I found myself feeling excited for the future of our profession, and for the value that those connections will create in the years to come.

Yours faithfully
Ben Exton
CPEng CMEngNZ IntPE(NZ)
CEO, Seismic Shift Ltd

MEMBER ENGAGEMENT

- Position Statement on Representation, Diversity, Equity and Inclusion

The SESOC Management Committee would like to thank members who have engaged with the Society regarding the Joint Technical Societies Position Statement on Representation, Diversity, Equity and Inclusion.

We have received a range of feedback from members, predominately supportive comments and a small minority of constructive suggestions. We appreciate the time members have taken to share their perspectives and contribute to the discussion.

SESOC remains fully committed to advancing structural engineering knowledge, promoting technical excellence, and supporting a strong and sustainable profession. We believe these objectives are strengthened when all engineers have equitable opportunities to participate, contribute, and succeed.

The position statement was developed collaboratively with NZSEE, NZGS, and BETS, reflecting a shared commitment across the participating technical societies. It is intended to support a profession that is both technically excellent and inclusive, recognising that diversity, equity, inclusion, and professional excellence are complementary objectives.

We acknowledge that members may hold differing views on how these objectives are best achieved and on the role of technical societies in this area. The Management Committee welcomes respectful and constructive discussion and values the diversity of perspectives within our membership.

The feedback received to date will help inform future reviews of the position statement. The statement will be reviewed periodically in collaboration with NZSEE, NZGS, and BETS to ensure it remains relevant, effective, and aligned with the needs of both the profession and our members.

We encourage members to continue sharing their views and ideas with the Society. To support meaningful engagement and dialogue, we ask that feedback is provided with the member's name and contact details. While anonymous submissions may be noted, the Management Committee is unable to engage in further discussion or seek clarification where the author is unknown.

Thank you to everyone who has contributed to the conversation.

The Joint Technical Societies Position Statement on Representation, Diversity, Equity and Inclusion is available in the "Media Releases & Position Statements" section of the SESOC website here:

https://www.sesoc.org.nz/archives/category/media-releases-position-statements

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- Maintain detailed and verifiable production and testing records.
- Undergo annual data reviews and biennial on-site audits.
- Meet performance criteria such as mean concrete strengths, aggregate quality, mixer efficiency, laboratory equipment calibration, and technician training.

Additionally, unannounced audits can be conducted to verify ongoing compliance. Failure to meet these standards can result in certificate withdrawal, reinforcing the scheme's credibility.

SPECIFY WITH CONFIDENCE

By specifying concrete from a certified plant, engineers mitigate risks associated with material non-compliance, which can lead to costly project delays, remedial work, or structural failures. Without the scheme, engineers and clients bear the responsibility of implementing independent testing regimes, which can be both expensive and logistically challenging.

In a sector where structural integrity is paramount, choosing certified concrete is a decision that delivers confidence and long-term value.



The Plant Audit Scheme audits Concrete NZ Readymix Sector Group member's concrete plants as defined in *NZS 3104:2021 Specification for Concrete Production*.

The Scheme provides a rigorous audit of the quality systems in place at a ready mixed concrete plant.

FOR A FULL LIST OF SCHEME MEMBERS AND THEIR AUDITED READY MIXED CONCRETE PLANTS VISIT THE SCHEME WEBSITE **www.rmcplantaudit.org.nz**

ADVANCES IN DESIGN FOR IMPROVED SEISMIC PERFORMANCE

By Stuart Oliver (SESOC President) and Kelly Cobeen (SEAOC)

There are currently efforts underway in both Aotearoa New Zealand and the United States to develop above-code seismic design provisions with the intent to improve structure performance and post-earthquake return to function. This article provides a brief overview of these efforts and their current status.

Aotearoa New Zealand Low Damage Seismic Design Guidance

Guidance documents for Low Damage Seismic Design (LDSD) have been published in Aotearoa New Zealand. A key goal of LDSD is to deliver buildings that are less likely to be damaged and thereby limit disruption and losses in future earthquakes. The adoption of LDSD on projects is non-mandatory. The guidance documents were developed in response to the 2010-2011 Waitaha Canterbury earthquake sequence and the 2016 Mw 7.8 Kaikoura earthquake. Both events caused significant damage to numerous modern buildings, with many eventually being demolished as a result. In light of such events, industry has recognised that a life safety focus is insufficient to meet community expectations about the likely performance of buildings in earthquakes.

LDSD emphasises thoughtful early decisions in design, including site selection and structural regularity. Stiff, regular buildings on good ground have been shown to perform well in earthquakes when secondary and non-structural elements are adequately restrained (e.g. partition walls and ceiling systems). In many cases it will not be necessary to include novel anti-seismic devices into structural systems (that is, seismic isolators or supplemental dampers) to achieve LDSD performance goals. LDSD performance goals can often be achieved using conventional structural systems, applied in a thoughtful way, in regular, well-coordinated buildings. An exception to this is buildings located in areas of very high seismicity when very high levels of seismic performance are being sought. For such buildings novel anti-seismic devices may be required.

The LDSD guidelines provide for both prescriptive and performance-based verification pathways to demonstrate a building meets specified LDSD performance goals. It is envisaged the simpler Prescriptive Design Method will be applicable to most buildings. For design teams using this pathway, the LDSD design process is similar to the current design process in the Aotearoa New Zealand Building Code system, albeit there is an increased requirement for design collaboration and reporting. Provided the prescribed design parameters and/or methods are used, then the Prescriptive Design Method is deemed to satisfy the LDSD provisions.

The performance-based Direct Design Method is used for building systems or components that don't meet the requirements of the Prescriptive Design Method pathway. It is for those building systems or components that are considered innovative, new or alternative solutions via the Aotearoa New Zealand Building Code system. Demonstrating compliance using this method requires the design team to follow recognised analytical methods, or experimental testing in accordance with recognised standard testing procedures, to validate the performance of potentially affected building systems or components and show that they will not compromise specified LDSD performance goals.

All three LDSD volumes are available on the design.resilience.nz website.

United States Functional Recovery Design Guidance

In a parallel effort in the United States, a first-generation version of functional recovery (FR) design guidance is nearing completion. To develop these requirements, a Functional Recovery Task Committee (FRTC) under the Provisions Update Committee (PUC) of the Building Seismic Safety Council (BSSC) was formed in late 2022. The PUC FRTC's overarching goals included reducing vulnerability, minimising losses, improving access to essential services, avoiding significant population displacement, and, ultimately, improving recovery time of new buildings and non-building structures. Over the course of approximately three years, the PUC FRTC developed first-generation seismic functional recovery code provisions and related commentary, which achieve quantitative targets for a new performance objective of functional recovery. These provisions and commentary were developed as a proposed non-mandatory appendix for inclusion in the 2026 *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures*. The provisions and commentary were developed, debated, balloted, and revised by the FRTC and the PUC through a deliberate process and forwarded to the ASCE 7-28 Seismic Subcommittee (SSC) for consideration as a proposed non-mandatory appendix for inclusion in the ASCE 7-28 standard *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*.

The functional recovery provisions are written as a stand-alone appendix but one that shares many aspects of current code-based seismic design. It is intended that new building design that implements FR will follow a standard seismic design process in accordance with the adopted building code and ASCE 7, and then an additional design process using the appendix. Both a prescriptive and a performance procedure are included; conventional as well as innovative structural systems (e.g., seismic isolation) are supported by the appendix.

Important aspects of the functional recovery provisions include the assignment of buildings and structures to Functional Recovery Categories ranging from A through E based on the service the structure provides to the community. Rather than relying on past categorisation of buildings, significant studies were conducted by subject-matter experts regarding the role that different services play in recovery following earthquakes and similar events.

A primary design measure used to achieve increased FR performance is design for reduced deflection and drift. Drawing on several decades of development of performance-based seismic design procedures and tools developed to evaluate performance-based design outcomes, as well as new FR specific studies, seismic design parameters for many common seismic force-resisting systems have been developed for FR performance.

Another key measure used to help ensure intended performance is a very extensive set of quality assurance provisions. This recognises that improved performance can only be realised if construction implements the intended designs. Added measures include establishment of a Functional Recovery Coordinator, conducting functional recovery meetings and walkthroughs, and more extensive special inspection requirements, among other provisions.

At the time of writing, versions of the functional recovery provisions are anticipated to be published in the 2026 NEHRP Provisions and ASCE 7-28. Thank you to Ryan Kersting and Reid Zimmerman for their leadership in the FR effort and review of this article.

BRAIN TEASERS

1. Where should the supports go?

A uniform beam, 10 m long, carries a uniformly distributed load over its entire length. It rests on two simple supports, each positioned the same distance from its nearest end, leaving equal overhangs.

Where should the supports be placed to minimise the largest absolute bending moment anywhere in the beam?

2. Does the hole shrink?

A circular hole is cut in a steel plate. The whole plate is then heated uniformly, without any restraint, while remaining flat.

Does the hole become larger, become smaller, or stay the same size?

3. Sharing the load

Two vertical rods—one steel and one aluminium—have equal lengths and equal cross-sectional areas. Their upper ends are fixed, and their lower ends support a rigid horizontal plate that is guided so it can move vertically but cannot rotate. Both rods are initially unstressed.

A downward load of 27 kN is applied to the plate. Taking the elastic moduli as 200 GPa for steel and 70 GPa for aluminium, how much load does each rod carry? Assume elastic behaviour and neglect self-weight and friction in the guides.

4. The painted cube

A solid cube is painted on all six faces and then cut into 64 identical smaller cubes, arranged four along each original edge.

How many small cubes have exactly two painted faces?

5. An unusual stopwatch

You have two ropes and a lighter. Each rope takes exactly 60 minutes to burn completely when lit at one end, but its burning rate varies along its length. The ropes need not burn at the same rate as each other.

Assuming the local burning rate is the same whichever direction the flame approaches from, how can you measure exactly 45 minutes? You have no clock and cannot use rope lengths to measure time.

Answers on page 102

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LETTER TO STANDARDS NZ AND MBIE

Letter to Standards NZ and MBIE giving objective critique of the Consultation Process for DZ NZS 3604 Timber-framed Buildings and DZ 3404 Parts 1 and 2 Steel Structures Standard and co-signed by representatives of SESOC, NZSEE, TDS, EGP and ENZ

24th August 2026
Malcolm MacMillan
National Manager
Standards New Zealand
Via email: malcolm.macmillan@mbie.govt.nz

CC: Dave Gittings – Manager Building Performance and Engineering, Ministry of Business, Innovation and Employment (MBIE)
Via email: Dave.Gittings@mbie.govt.nz

Dear Malcolm,

Re: Consultation Process for DZ NZS 3604 Timber-framed Buildings and DZ 3404 Parts 1 and 2 Steel Structures Standard

We write jointly regarding the consultation process currently underway for two significant draft standards: DZ NZS 3604 Timber-framed Buildings (open until 8 September 2026) and DZ 3404 Parts 1 and 2 Steel Structures Standard (open until 22 September 2026).

We recognise the substantial effort that has gone into both drafts, and we value the opportunity to comment. We are reaching out to express our concerns with a consultation process that, as currently designed, works against the quality of feedback these documents need and deserve.

The issue

Both of these standards form part of the means of compliance with the New Zealand Building Code (NZBC). Engineers, designers and building consent authorities rely on them daily to demonstrate compliance, and the Building Code's foundation is a life safety performance objective. A standard with an undetected error, omission, or internal inconsistency can flow directly into buildings that people live and work in. Robust, thorough technical review via a public consultation process before publication serves as a critical quality assurance measure to ensure such errors or omissions are caught before standards are passed into the built environment. A public consultation process that makes robust review difficult is, in effect, a risk to the quality of the final standard.

Why the current format works against a robust review

Both drafts are available only through an online viewer, displaying two pages at a time. The online viewer, as currently implemented, does not allow a reviewer to:

- hold two related parts of the document open at once, for example a clause and its commentary, or a clause and the cross-reference it depends on
- jump directly to a known page or clause, there is no page-number or "go to" function, so navigation is only sequential, one screen at a time
- follow a chain of cross-references without losing their place and having to re-navigate from the start each time
- rely on the tool holding up under sustained use, with the viewer reportedly crashes with some regularity, risking lost review time

This is compounded by the submission form itself, where if a reviewer logs out and back in, they must click through every comment they have already entered before reaching the point where they can add the next one.

These issues create a serious practical barrier on documents of this size and technical density, and it is compounded by the 20-comment limit per submission on the online portal.

To illustrate the difficulty we face in completing a proper technical review, an example is the seismic design section of the DZ 3404 steel standard. Section 12 (Seismic Design) scope clause (12.1.1) directs the reader to Sections 3 to 9 inclusive and Section 13 for the underlying design and material provisions it builds on. Individual clauses within it (for example 12.1.3.2 and 12.8.5.4) further require the reader to cross-reference Appendix M (structural ductility factor limits) and Appendix K (bolted moment-resisting connection requirements). A competent review of this one section alone therefore requires simultaneous, easy access to at least six other parts of the standard.

Intellectual property considerations

We understand the stated basis for the viewer-only, non-downloadable format is to protect intellectual property, including against unauthorised copying or AI-enabled use. Considering that both DZ NZS 3604 and DZ 3404 are funded by MBIE and, on publication, are provided to the public free of charge, we are not clear on how restricting access to the draft can be justified by protecting the value of a document that will shortly be given away.

Our position

The technical societies contributing to this process, including our members, provide this technical review on a voluntary basis, on top of full-time professional workloads, because we believe robust building standards are critical for New Zealand. We are not asking Standards New Zealand or MBIE to make our job easier for our own convenience; we are asking for a format that allows the time our members are donating to be used effectively, so that the outcome is a standard New Zealand can rely on. We expect that the

current process will materially reduce both the volume and the quality of technical feedback the consultation receives.

Without rigorous testing, comments and feedback that picks up errors omissions or provides any needed clarity, Standards risk these important documents either not being fit for citation under the New Zealand Building Code or having amendments in the citation process.

What we would like from Standards New Zealand and MBIE

We request that Standards New Zealand provide a complete, downloadable PDF (or equivalent full-document, searchable format) of both DZ NZS 3604 and DZ 3404 Parts 1 and 2, for the remainder of the current consultation periods, so that technical reviewers can carry out the cross-referencing and comparative review these documents require. Given the closing dates are approaching, we would appreciate a prompt response and are happy to discuss any reasonable safeguards (for example, a watermarked or licensed release to registered submitters) that would address the stated IP concern without compromising the review.

Furthermore, we request that Standards New Zealand adopt a similar approach for the public consultation of similar standards in the future.

We would welcome the opportunity to discuss this directly and to work with you on a consultation approach that serves everyone's shared goal of a robust standard that supports the life safety objective of the Building Code.

Regards,

Stuart Oliver
President, New Zealand Structural Engineering Society (SESOC)

Julian Benito
President, New Zealand Society for Earthquake Engineering (NZSEE)

Lisa Oliver
President, New Zealand Timber Design Society (TDS)

Kelly Pilkington
Chair, Engineering General Practitioners Group (EGP)

Tania Williams
Chief Engineer / General Manager, Engineering New Zealand (ENZ)

SESOC SOCIAL MEDIA UPDATE

Prepared by:

Purpose

To provide an update on the state of the SESOC social media communications to date.

- SESOC's LinkedIn page has steadily grown in followers and engagement, indicating increasing interest from both SESOC members and the wider engineering community.
- New Emerging Structural Engineers (ESE) page has been growing, and future initiatives have been discussed.
- The page has been a clear asset in distribution of webinars and seminars.

Current Metrics

- Followers: 2,878, getting close to the total number of our members.
- 12,987 impressions in 1 month period. Increased over 100% in one 30-day period.
- Average engagement per post: between 900 and 2900 impressions.

Julio Ortiz

◄ **SESOC LinkedIn page**



Ancon Masonry Support Systems

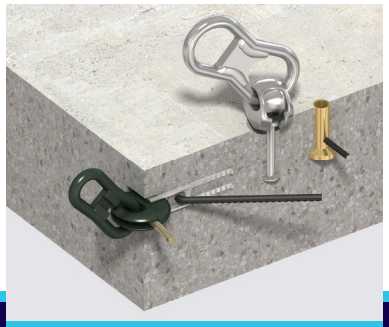
Ancon

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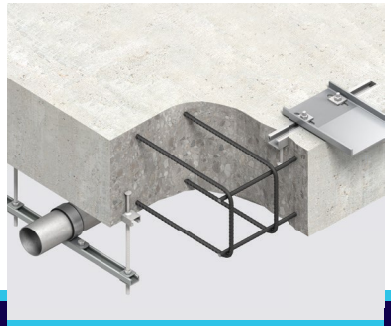
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AN OVERVIEW OF PROPOSED REVISIONS IN DZ 3404 STEEL STRUCTURES

Thompson, A.[1], Cowie, K.[2], Fussell, A.[3]

Background

Standards New Zealand is currently consulting on a proposed revision to NZS 3404:1997 *Steel Structures Standard* Parts 1 and 2 (normative and commentary). Although amended in 2007 to align with the AS/NZS 1170 suite, it is almost 30 years since a major revision occurred. Members are encouraged to familiarise themselves with the proposals and provide comment via Standards New Zealand's consultation page (linked in footnote)[1] although the publishing date of the SESOC Journal post-dates the end-of-submissions period which closes 22 Sep 2026.

This brief article outlines several of the more substantive areas of proposed change (but is not exhaustive).

Materials and Brittle Fracture

Improvements have been made to Section 2 *Material Selection* requirements in usability and the breadth of prequalified materials and referenced product standards, including API 5L line pipe and ASTM 709 weathering steels. Proposals have benefited from work by Professor Hobbacher (commissioned by HERA) to apply fracture mechanics methodologies used in Eurocodes to update NZS 3404 material selection requirements. This was partly initiated by use cases in the Antarctic. However the breadth of coverage and reduced conservatism in lower temperatures (especially in ordinary plate thicknesses $t < 40$mm) will enjoy much wider applicability. Lamellar tearing considerations in the steel selection process are now included, to harmonise with AS/NZS 1554.

Seismic material requirements are consolidated into Section 2 for ease of reference. The introduction of conforming steel types should simplify sourcing for seismic applications. This pathway no longer requires checks on a material test certificate for extra-over requirements. Regionality-based overstrength factors have been removed — they are now based on grade.

Seismic member Category 3 *Material Requirements* have been relaxed, like proposals in the withdrawn NZS 3404.1:2009 edition. It also becomes easier to specify EN 10219 CHS column sections in moment resisting frames due to the introduction of a Category 2 'secondary' classification.

Structural Analysis

Changes to Section 4 *Structural Analysis* are largely organisational, with introductory clauses on *structural stability of frames* introduced to align more closely with AISC 360 and Eurocodes. These clauses clearly identify the basic phenomena that need consideration in structural analysis for stability: imperfections (global and local), and second order effects (P-$\Delta$and P-δ). Regardless, the conventional elastic analysis techniques that exist in the current revision remain essentially unchanged because they already include these considerations.

A new section 4.7 *Advanced Analysis* is proposed, recognising growing use of such methods and similar treatment in other international standards. This includes material on direct analysis (4.7.1, for non-seismic applications), and non-linear dynamic analysis (4.7.2, for seismic applications). It is the direct analysis provisions (4.7.1) that benefit most from the clearer articulation of stability phenomena mentioned above.

The entirety of Section 4.7 *Advanced Analysis* is intended to be excluded from citation under B1/VM1, reflected in the wording and splitting of content between clauses 4.7 and C4.7 (commentary). For seismic applications, this is consistent with the treatment of time-history analysis provisions in NZS 1170.5:2004 and recognises the current period of evolution in seismic design practice. For both sets of provisions, it allows road testing of provisions under an alternative solution compliance pathway.

Connections and Fatigue

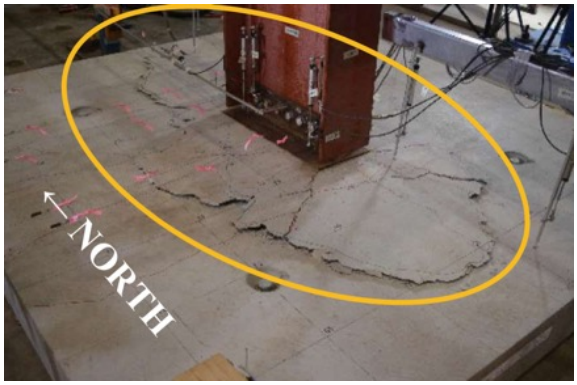
Notable updates to Section 9 *Connections* include block shear provisions, inclusion of property class 10.9 bolts, requirements for simple beam connections to concrete walls (Figure 1), and introduction of an equivalent complete penetration butt weld (ECPBW, Figure 2). The ECPBW is applicable to a tee joint (e.g. end plate) with a pair of balanced fillet welds or compound welds, where the sum of their design throat thickness exceeds 1.0 to 1.35 times the plate thickness (depending on the application and material). In certain plate thickness ranges, this could offer improved fabrication economy.

The fatigue section has been edited reasonably extensively to modernise and expand the provisions.

[1]https://consultations.standards.govt.nz/draft-standards/dz-3404-part-1-and-2-steel-structures-standard/

PAPER CLASS & TYPE: NON-REFEREED

[1] *BE, CPEng, Envelope*
[2] *BE, CPEng, SCNZ*
[3] *ME, CPEng, Tangent Consulting*



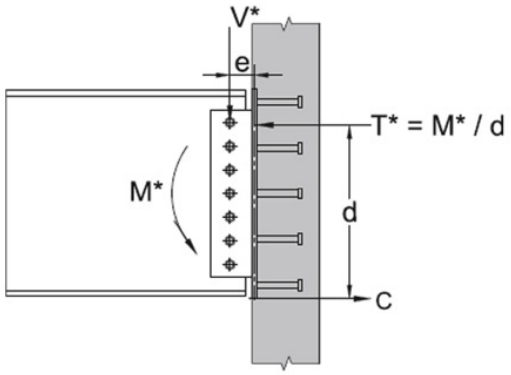
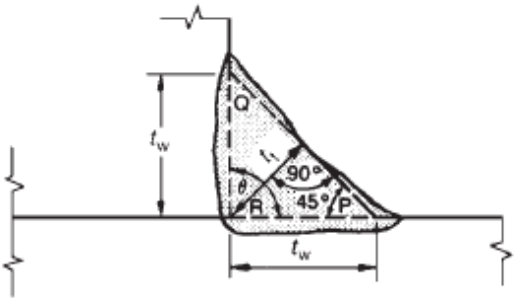


Figure 1: New provisions are included for the design and detailing of simple concrete wall-steel beam connections informed by work by Pascua, Henry et al (2023).



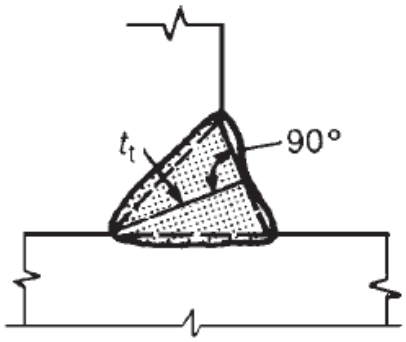


Figure 2: Provisions for equivalent complete penetration butt welds are introduced, comprising a pair of balanced fillet welds or compound welds that meet certain requirements.

Design for Fire Resistance

Section11: *Design for Fire Resistance*: This section is extensively revised and expanded, offering designers the choice to continue with simple/traditional prescriptive pathways, or use advanced methods that could offer improved economy (Figure 3). Additional detailing requirements are included.

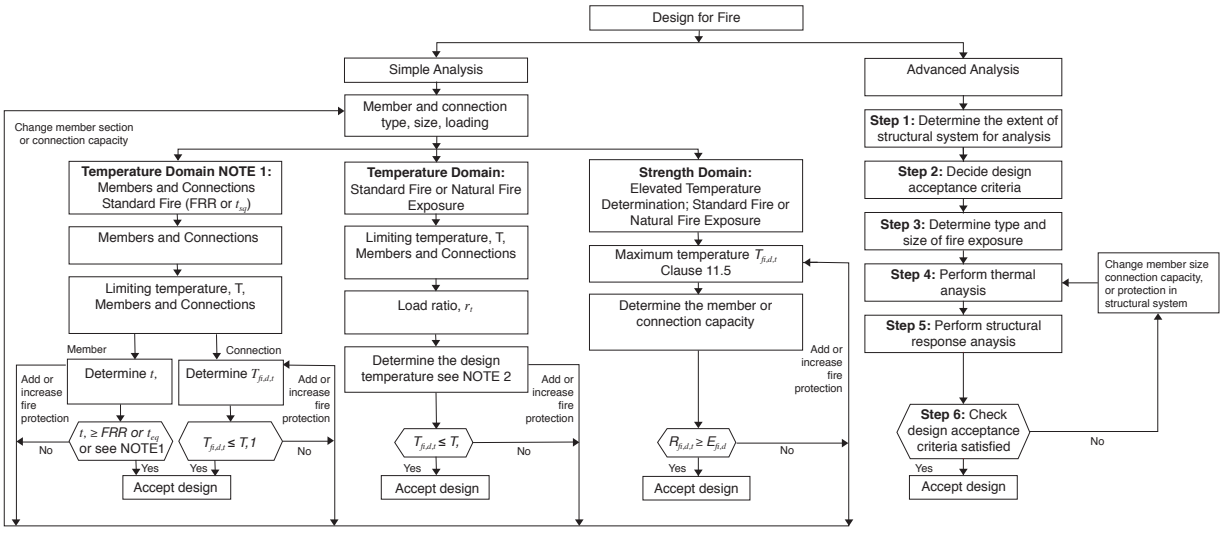


Figure 11.2.10 - Fire engineering design approach options

NOTE —

(1) For unprotected members, use Equation 11.5.3.2 or Equation 11.5.3.3. For protected members, either use Equation 11.5.3.4 or use non-reactive passive fire protection manufacturer's data in accordance with 11.3.5 (see C11.35).

(2) For unprotected members exposed to the standard fire or to the natural fire and for protected members exposed to the natural fire, $T_{fi,d,t}$ = the elevated temperature from 11.4.2 or 11.4.3 for members and 11.4.4 for connections. For protected members exposed to the standard fire, $T_{fi,d,t}$ = the critical temperature from the manufacture's charts for the given FRR.

Figure 3: Flow chart from the revision proposals for section 11: Design for Fire Resistance

Seismic Design

Changes to the seismic section are largely incremental. However, several new structural systems feature in the proposals and there are other changes worth highlighting that will have impacts on design. Importantly though, **the Section 12 proposals are not in connection with the recent release of SNZ TS 1170.5:2025**, nor the work of MBIE's Seismic Risk Working Group (SRWG) and Standards NZ on possible further (more expansive) updates to TS 1170.5 as communicated at recent industry events.

The NZS 3404 revision project was initiated prior to the 2022 NSHM, SRWG and TS 1170.5 project timeframes, and is limited in scope. The revision proposals continue to reference NZS 1170.5:2004 as the primary standard for earthquake actions. The changes improve alignment with the existing requirements of NZS 1170.5:2004, incorporate some recent developments in structural steel design and address recommendations of the Canterbury Earthquakes Royal Commission (CERC).

Improving structural robustness and redundancy

As part of improving alignment with the robustness requirements of NZS 1170.5:2004 and CERC recommendations, the proposals extend the critical height concept (> 4 storeys) across all structural systems. The robustness requirements for such structures are more onerous. Previously, the critical height only affected provisions for moment resisting frames.

Column continuity requirements are made explicit for seismic-resisting systems. Additional requirements for columns in associated structural systems (column continuity and detailing in simple 'gravity frames') reflect their important cumulative contribution to structural redundancy and spread of plasticity in severe earthquake shaking (Figure 4). This gives effect to a key recommendation of CERC regarding structural redundancy.

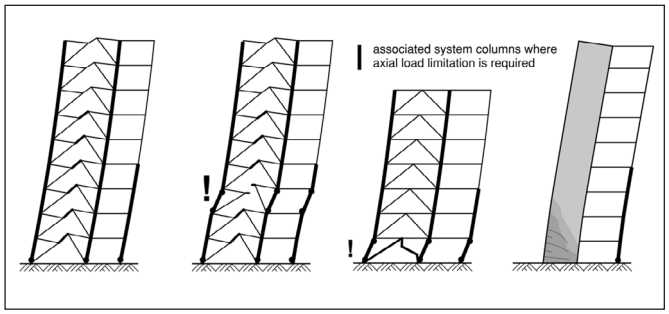


Figure 4: New provisions make the requirement for column continuity in seismic-resisting frame systems explicit and also introduce requirements for associated system columns

Adjusted provisions affect the way upper limit actions apply in a capacity design process. To highlight one important example (12.8.1.2.2), connections between all category 1-3 members in seismic resisting systems above the *critical height*, (or other systems below this height requiring capacity design) require capacity design considerations to be fully carried through in connection detailing. Use of upper limit actions is not permitted.

Unfortunately, document formatting errors in the public comment draft have affected list numbering and heading levels across several sections (including 12.8.1.2.2). This will require rectification prior to publication and does affect the legibility of the proposed provisions.

Axial load limitation provisions (12.7.3) are important for columns that could be subject to inelastic demands in severe earthquake. They ensure columns can sustain stable flexural hinges and avoid excessive axial shortening. A more recent and less conservative end-yielding criterion is included (12.7.3.2). This criterion aims to ensure flexural yielding occurs at restrained member ends instead of resulting in column buckling (due to P-δ influence) and is often a controlling criterion (Figure 5). The revised provisions are clearer on the extent of situations subject to axial load limitation (Figure 4), and how to treat columns where axial compressive load is not limited by a plastic mechanism (Figure 6).

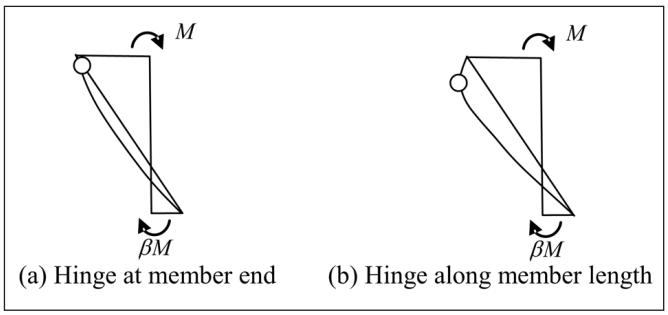


Figure 5: The 'end-yielding criterion' in axial load limitation requirements seeks to ensure flexural hinging occurs at restrained member ends, not within the clear span of a column where it can lead to column buckling (MacRae, Lu et al, NZSEE Bulletin, 43(1) 2010).

A potentially significant change affects taller, elastically designed concentrically braced frame (CBF) systems (those above the critical height). The maximum (numerical) column ductility category changes from 4 to 3. This affects column cross section geometry, but more importantly it triggers the axial load limitation criteria and connection/splicing requirements mentioned above. Both are important to column robustness and stability (Figure 6).

Increase in minimum column weight and impacts on connection detailing are likely because of this change. However, impacts on economy can be minimised or avoided by selecting optimum column section

geometry (don't use UB sections), or by using ductile braced systems. These would seem to be reasonable and appropriate incentives, and they also improve consistency in minimum required performance across different structural systems. The resulting proposals for elastic CBFs are more closely aligned with ASCE 7, which places heavier restrictions on 'ordinary' (non-capacity designed) braced frames than those that have existed in NZS 3404:1997 (Inc. Amd. 1-2) until now.

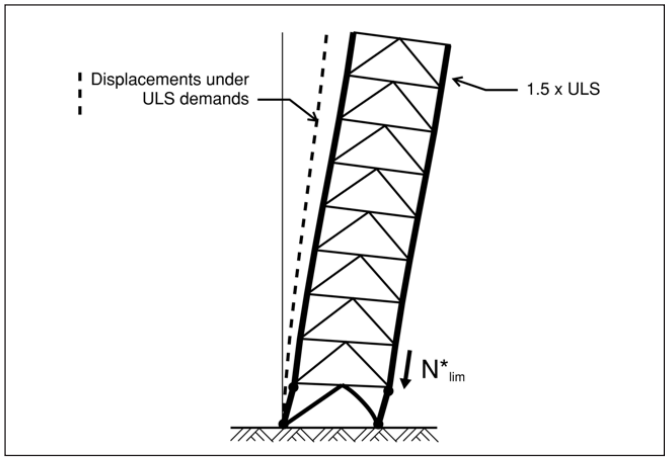


Figure 6: Diagram highlighting purpose of axial load limitation provisions as applicable to a tall elastically designed CBF in severe earthquake. The revised proposals are most likely to affect taller elastic CBF frames where it is more difficult to limit column axial load by capacity design.

Few of the above listed changes affect structures below the critical height, where most provisions remain similar to the current revision. This reflects a lack of evidence available to the committee to justify more onerous provisions for low rise structures on a risk basis (onerous in terms of both design process and construction economy).

The commentary (part 2) contains extensive discussion on all aspects discussed above.

New seismic resisting systems

New provisions cover reduced beam sections (RBS) as part of moment-resisting frame (MRF) seismic resisting systems. Eccentrically braced frame (EBF) provisions are not new, but updates include explicit treatment of built-up sections, and recognition of the higher ductility capacity of links with stocky webs.

Other systems are introduced with performance provisions (and further information in their respective commentaries) but are drafted with the intent of exclusion from citation under B1/VM1. These include:

- Dissipative connections in MRF (rigid or semi-rigid connections where the principal energy dissipating mechanism occurs within the joint).
- CBF's with friction-dissipating mechanisms.
- Buckling restrained brace frames (BRBF).

Future amendments or revisions to align with the 1170.5 project

Subject to the path of the SRWG/1170.5 project, it is likely that a future amendment to NZS 3404 Parts 1 and 2 will be required (for seismic design aspects), to align with any requirements or alterations in design process that result from a revised standard for earthquake actions.

Hybrid Concrete and Steel Structures

As composite structures are now covered by AS/NZS 2327, this section has been repurposed to address the intersection of concrete and steel materials in hybrid structures, whether or not members act compositely as a section.

Fabrication and Erection

These sections have been heavily reduced as most aspects are now covered by AS/NZS 5131 *Structural Steelwork – Fabrication and Erection.*

Case-hardened steel screws

Case-hardened steel screws are essential to mass timber construction but are potentially susceptible to both Internal Hydrogen Embrittlement (IHE) during the manufacturing process, and Environmental Hydrogen Embrittlement (EHE) post-manufacturing. The risk of IHE can be limited by using reputable screw manufacturers with code compliance reports. However, even high-quality fasteners can be susceptible to EHE which can occur when a susceptible fastener is exposed to both a source of hydrogen (usually moisture) and sustained tension.

Mitigation measures to help avoid EHE include the following:

- Align specified coatings and service class with the expected moisture exposure and corrosive conditions, both in-service and during construction.
- Ensure the fastener installation instructions are communicated, understood and implemented.
- Identify high risk connections (more likely to be exposed to both moisture and sustained tension) and plan accordingly.
- Require the contractor to submit a clear and achievable moisture mitigation plan
- Consider specifying fasteners with a lower core hardness.

For additional information, talk to your screw supplier or reference the Timber Unlimited Technical Note: Case Hardened Steel Fasteners & Mass Timber Construction.

Timber Unlimited

SEISMIC PERFORMANCE OF FIRE-RATED PARTITION WALLS WITH "NO-FIX ZONES" DETAILS IN AOTEAROA NEW ZEALAND INDUSTRY PRACTICE

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(Reprint of paper from NZSEE Conference 2026)

Abstract

Two full-scale wall specimens, each 3.55 m high, one timber-framed and the other steel-framed, were constructed between reinforced concrete slabs on a 6 m × 3.50 m plan, following standard Aotearoa New Zealand industry practice with fire-rated plasterboard linings. Specimens were subjected to floor motion records representative of a six-storey reinforced concrete building model excited by scaled El Centro ground motion records. Lateral movement at the top of the walls was accommodated in-plane through a deflection head track (DHT), while out-of-plane flexibility at wall junctions was achieved through deformation of the timber top plates in the timber wall and by the DHT in the steel wall. DHT anchors for the top plates were intentionally offset 1.20 m from wall junctions to create "No-Fix zones," allowing the intended flexibility.

The results showed that the timber-framed walls could preserve their fire integrity up to 0.60 % drift and exhibited top-plate damage at 1.00 % drift. The steel-framed wall with "fixed" ends outperformed timber-framed walls, sustaining minor damage (leading to partial loss of fire integrity) at 1.80 % drift and critical damage (separation of walls at junctions) at 2.50 %, far exceeding the 0.60 % median capacity of traditional steel walls. Displacement readings also confirmed notable out-of-plane flexibility at "No-Fix zones" junctions. These results advocate for flexible top-track detailing to enhance drift capacity of walls. However, they also reveal potential deformation incompatibilities with other non-structural elements, indicating a need for further investigation into interaction effects for improved seismic resilience.

1. INTRODUCTION

In Aotearoa New Zealand, current industry practice constrained by limited evidence regarding the seismic performance of partition wall detailing, often results in the overly conservative design of very stiff structures. This approach increases construction costs and embodied carbon without necessarily delivering proportionate performance benefits. The Aotearoa New Zealand building industry currently lacks sufficient empirical data on the performance of plasterboard lined partition walls under seismic loading conditions, especially for displacements exceeding the suggested 0.33 % inter-storey drift for serviceability limit state in Table C1 of AS/NZS 1170.0 (2002). As a result, design teams face uncertainty when specifying these systems in buildings.

Utilising University of Canterbury's Structural Engineering Lab facility our research team was able to gather critical performance data on existing and novel practices for partition walls under seismic conditions for multi-unit residential and commercial buildings to address this knowledge gap. Such data has the potential to inform code revisions, support more cost-effective and sustainable building designs, provide suitable design options for and reduce unnecessary material use in Low-Damage Seismic Design (LDSD) projects (MBIE and NHC, 2024).

As a part of this experimental campaign, three phases of tests were conducted (Bhatta, 2026):

- **Phase I:** Evaluate the seismic performance of typical partition walls with "No-Fix Zones". This phase investigates the seismic performance of these partition walls under simulated seismic demands of displacement and acceleration.
- **Phase II:** Evaluate the combined seismic performance and interaction of typical partition walls (with "No-Fix Zones" details) with perimeter-restrained concealed plasterboard ceilings and mechanical pipes, under simulated seismic demands of displacement and acceleration.
- **Phase III:** Evaluate the seismic performance and interaction of novel "rocking" partition walls with fully floating plasterboard ceilings and mechanical pipes, under simulated seismic demands of displacement and acceleration.

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This paper presents the Phase I experimental results. In this detailing, connections between the deflection head track (DHT) and the floor above are prohibited at wall junctions. This allows the system to accommodate inter-storey drifts: out-of-plane drift through the bowing of the DHT and top plates, and in-plane drift through the sliding of the wall framing and lining relative to the DHT or floor. Further details can be found in Bhatta (2026).

2. BACKGROUND

Non-structural elements (NSEs), such as partition walls, suspended ceilings, mechanical and electrical systems, cladding, and windows, are essential to a building's functionality and operation. Although these elements are secondary and are either attached to or placed near structural members, they can interact with the structure or with one another during an earthquake, leading to significant damage (Baird et al., 2014; Dhakal, 2010; Dhakal et al., 2011) Such damage often results in costly repairs, operational downtime, evacuations, and even full replacements (Bachman, 2004; FEMA, 1994; Filiatrault, 2002). Suspended ceilings, piping, and partition walls have been identified as major sources of economic loss following seismic events (FEMA, 1994; Filiatrault, 2002).

Lightweight internal partition walls are common in multi-unit residential and multi-story commercial buildings. Typically built with a steel or timber frame, comprising vertical studs, horizontal plates, and noggings, they are lined on one or both sides with gypsum plasterboard fastened with screws and/or adhesive. The walls are then finished by plastering joints, sanding, and painting.

Experimental research from various regions consistently shows that such walls begin to sustain seismic damage at very low drift levels. The onset of damage typically occurs between 0.26 % and 0.30 % inter-story drift (Davies et al., 2011; Mulligan et al., 2022; Petrone et al., 2015; Tasligedik et al., 2015; Yeow et al., 2018) with some studies reporting initial cracking as low as 0.05 % (Restrepo and Bersofsky, 2011) or at 0.38 % (Kim and Shin, 2021) underscoring their high vulnerability to even minor shaking.

The 2010-11 Canterbury Earthquake Sequence in Aotearoa New Zealand demonstrated this vulnerability, causing widespread partition wall damage such as screw failures, cracked or crushed plasterboard corners, and dislodged panels (AWCI, 2018; Baird et al., 2014; Dhakal, 2010; Tasligedik, 2014). Damage also recurred during aftershocks that exceeded serviceability limit demands (Arifin et al., 2021). Such early failures often lead to significant economic losses in minor to moderate earthquakes, due to repair costs, replacements, and operational downtime (Bachman, 2004; Bradley et al., 2008). In response, seismic-resilient partition systems have been developed to improve performance. Innovations like seismic relief joints (Tasligedik et al., 2015), flexible tracks (Davies et al., 2011; Mulligan et al., 2020), slotted and bracketed head tracks (Bhatta, Dhakal, Sullivan et al., 2023) , and "rocking" walls (Bhatta, Dhakal and Sullivan, 2023a; Bhatta, Dhakal and Sullivan, 2023b; Bhatta and Dhakal, 2026).

In Aotearoa New Zealand, "No-Fix Zone" detailing for partition walls is typically specified, in buildings with specific post-earthquake functionality requirements, when inter-storey drifts are expected to exceed the 0.33 % serviceability limit in the AS/NZS 1170.0 (2002). However, a lack of performance data for these details has led to overly conservative and stiff structural designs. This study investigates the seismic performance of these walls to inform industry practice and future code revisions, aiming to support more cost-effective, sustainable, and materially efficient LDSD (MBIE and NHC, 2024) projects.

3. EXPERIMENTAL PROGRAMME AND METHODOLOGY

The methodology involved constructing full-scale timber and steel-framed wall specimens following industry best practices, then subjecting them to incrementally increasing seismic demands (displacements, velocities, accelerations) in laboratory testing. These tests simulated realistic building drift levels, both within and beyond serviceability limit demands, to evaluate damage thresholds, failure modes, and fire integrity. Data was collected via displacement/acceleration measurements, Motion Capture (MoCap) Camera and photographic documentation.

3.1 Test Setup

A photo of the test setup and wall specimens is shown in Figure 1. The test setup comprised two 100 mm thick reinforced concrete slabs (3.50 m × 6 m) positioned at the top and bottom of steel columns (200 UC 46.2). The clear vertical spacing between the slabs was 3.55 m. The columns were pin-connected to the slabs, allowing free rotation during lateral movement in the X-direction. The bottom slab was elevated 500 mm above the laboratory floor using concrete blocks. The blocks were mounted on a linear ball bearing and guideway assembly that permitted sliding of the bottom slab in the X-direction. Lateral movement of the slabs was applied through 250 kN actuators with ±500 mm stroke capacity. Both actuators could be operated simultaneously, enabling coordinated movement of the slabs under the prescribed loading protocols. Stability in the Y-direction (perpendicular to loading) was provided by tension-only braces. In the X-direction (loading direction), these braces were removed during

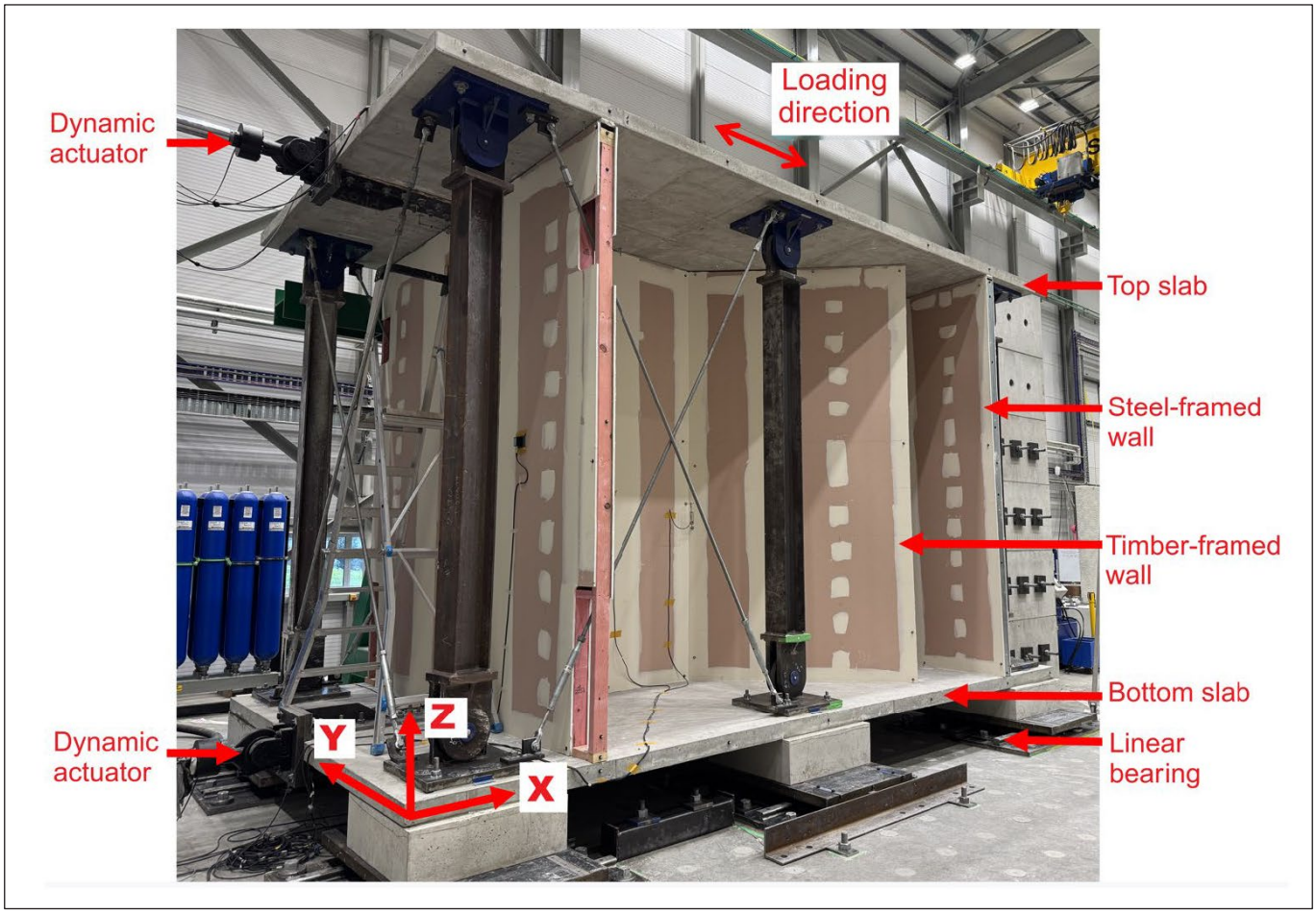


Figure 1: Photo of the test setup showing partition wall specimens

testing and reattached once the tests are complete. Refer to Pledger (2026) and Bhatta (2026) for additional information regarding the test setup components.

3.2 Wall specimen design, configurations and details

3.2.1 Walls with "unrestrained" ends ("unrestrained" specimens)

The two wall specimens, one timber-framed and one steel-framed, were constructed between the concrete slabs of the test setup as shown in Figure 2, Figure 3, and Figure 4. Their key design details were as follows:

- The walls were permitted to slide in-plane along the top deflection head track (DHT).
- Anchors connecting the DHTs to the top slab were omitted within 1200 mm of the wall T-junctions, defining the "No-Fix Zones" as shown in Figure 3.
- Out-of-plane flexibility at the junction was achieved through the bending of the top timber plate (in timber wall) and the DHT (in steel wall).
- At the wall junctions (with inclined walls and return walls), DHT flanges were cut approximately 40 mm short to prevent premature interaction with adjacent DHTs.
- Frames were lined with 13 mm GIB FYRELINE plasterboard per GIB EzyBrace® System specification GS2-N (GIB, 2016), but the plasterboard linings were not fastened to the top DHTs, as shown in Figure 4.
- The frames meeting at the wall T-junctions were connected. The top plates of the timber wall at these junctions were secured with a steel strap. The return intersection details in timber and steel framed walls are shown in Figure 4.

3.2.2 Walls with "restrained" ends ("restrained" specimens)

The wall ends of the "unrestrained" wall specimens were later prevented from rotating (to represent an approximately fixed boundary condition) by placing 2/45 × 90 timber elements tightly against the wall linings as shown in Figure 5. These timber elements were secured in position using threaded rods anchored into the top slab.

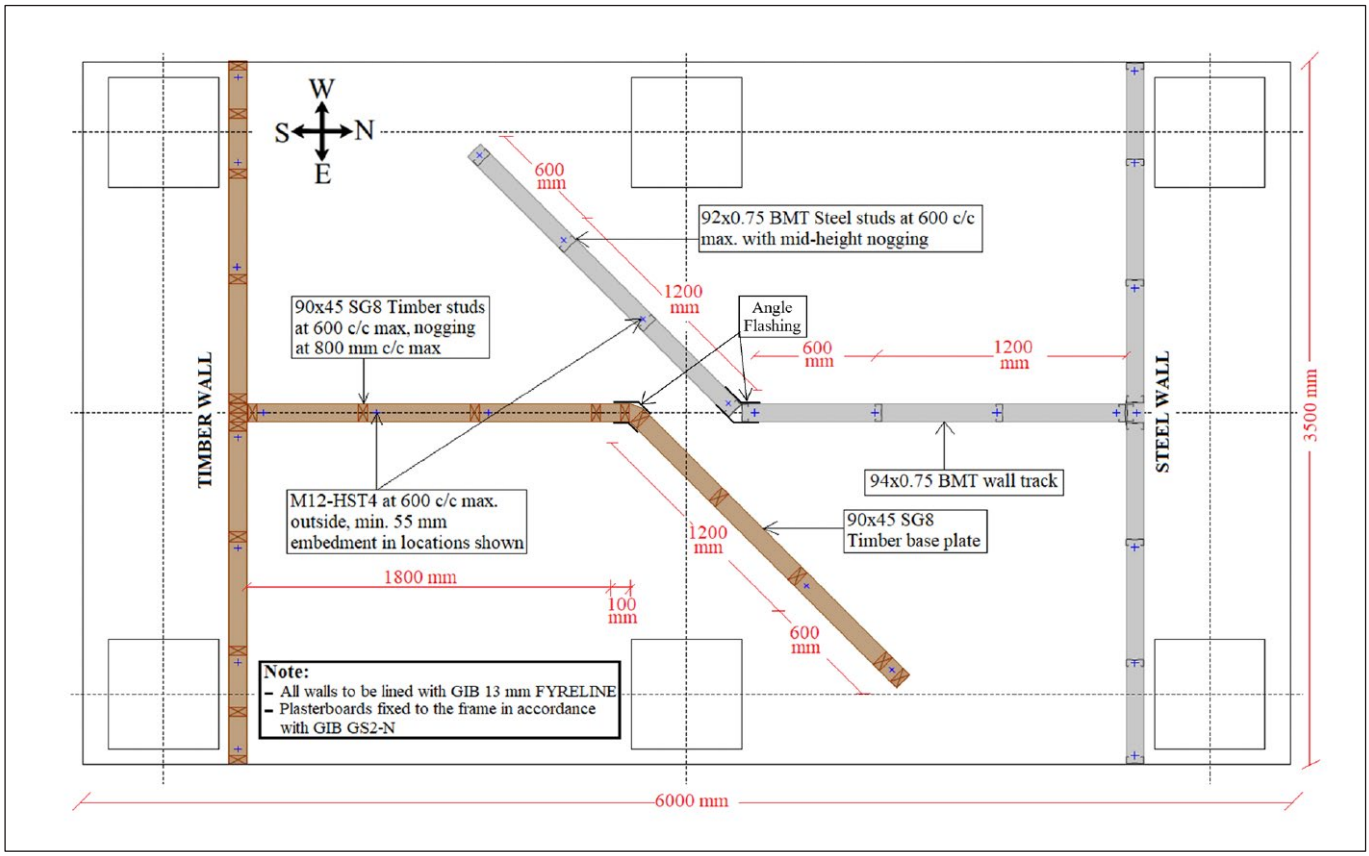


Figure 2: Timber and steel stud framing plan – "unrestrained" specimens

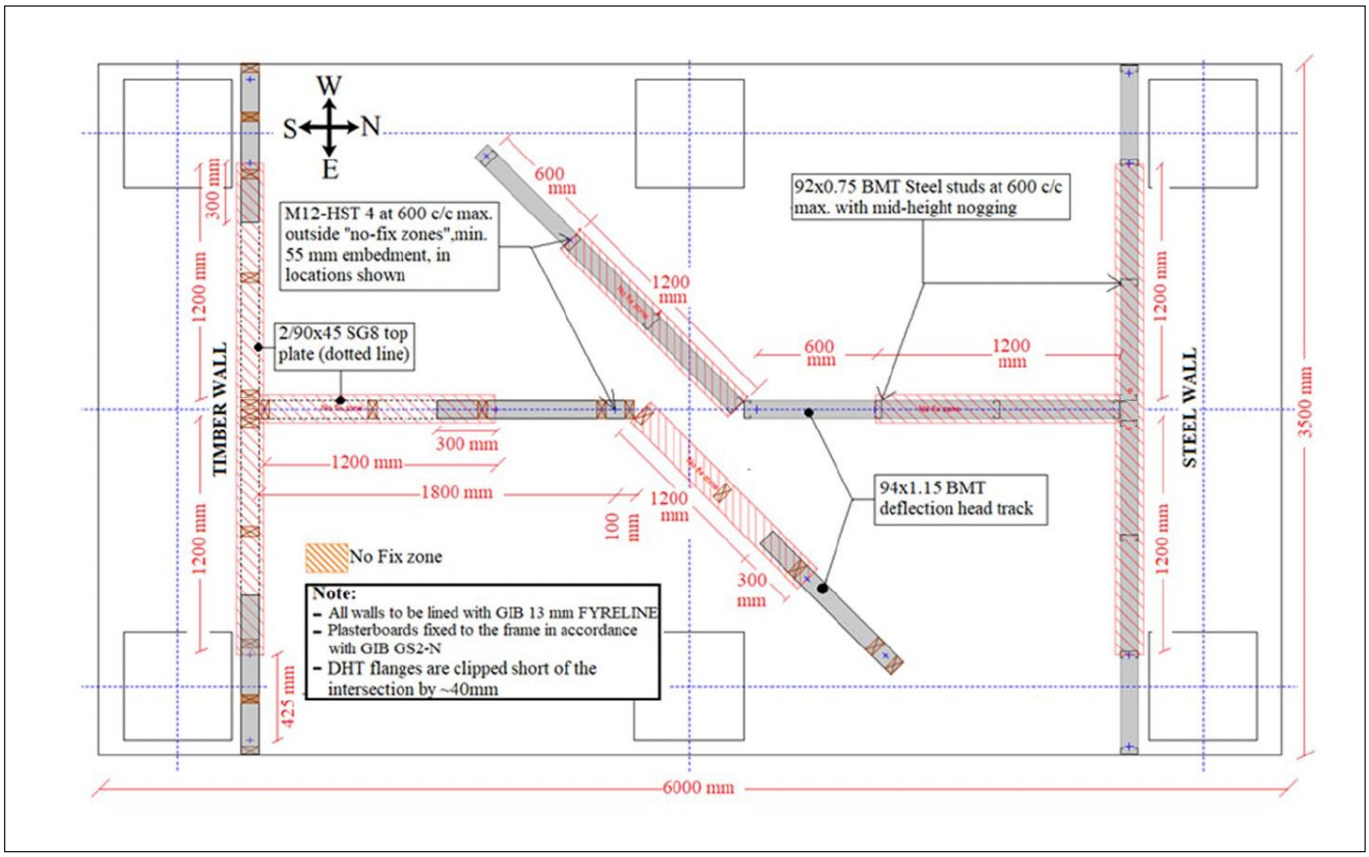


Figure 3: Timber and steel stud reflected ceiling plan – "unrestrained" specimens

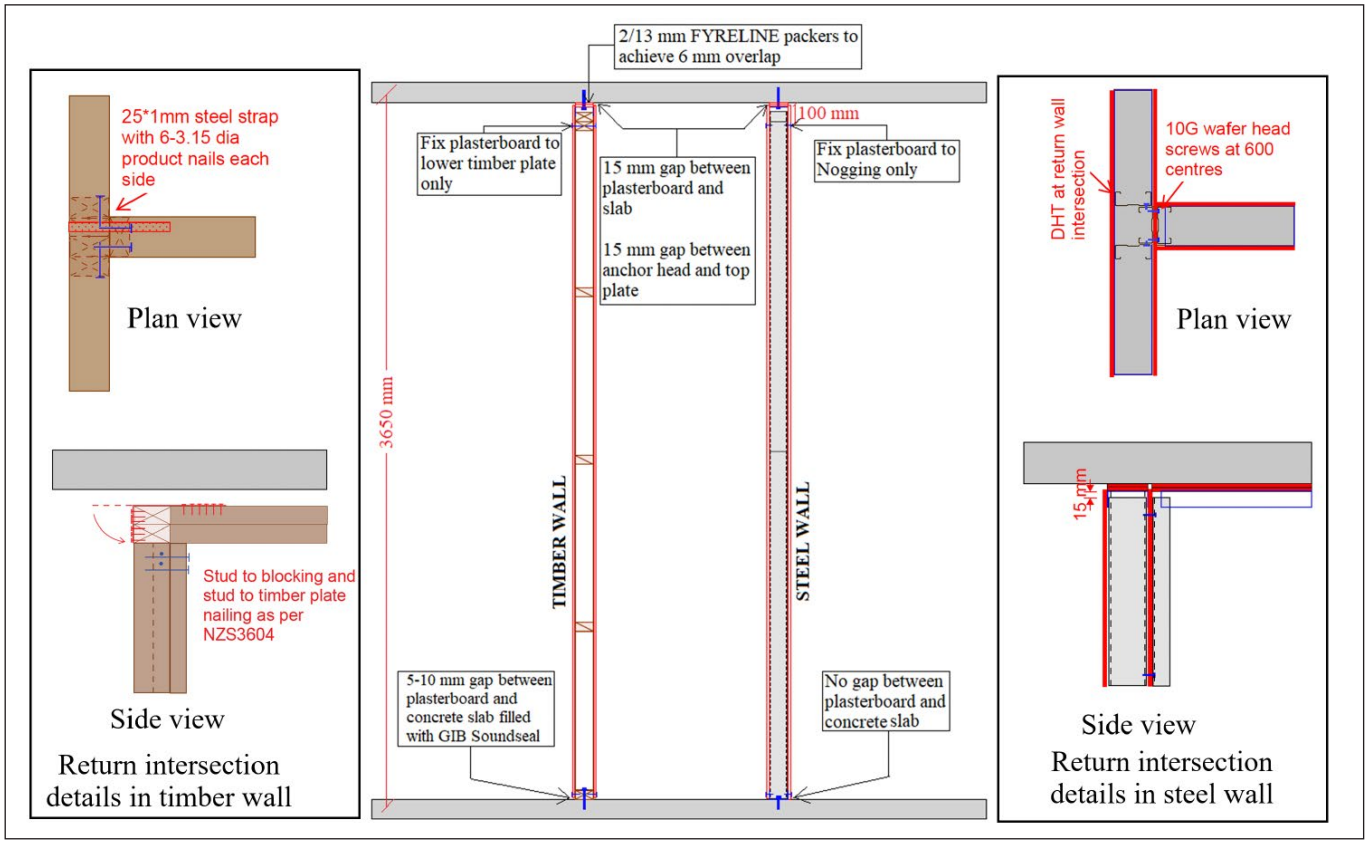


Figure 4: Wall section elevation and return intersection details (typical) – "unrestrained" specimens

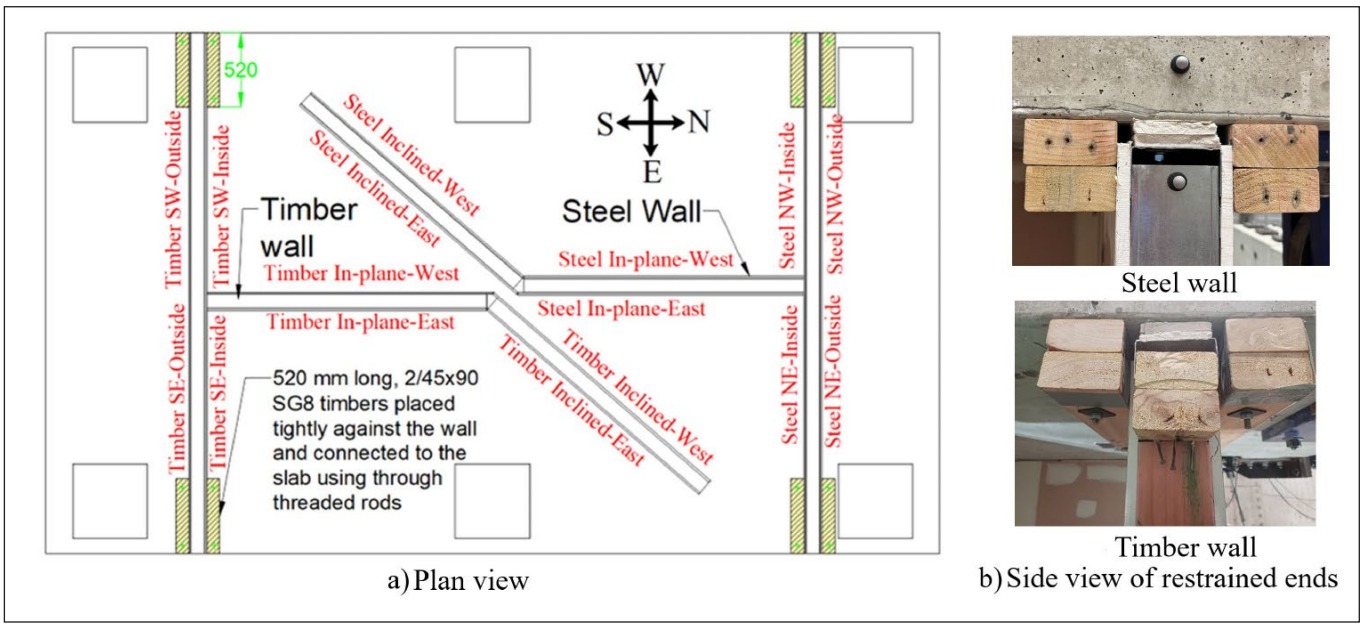


Figure 5: Wall specimens with "restrained" ends with timber members

3.3 Loading protocols

Displacement time histories (referred to as "Motions") for the second and third floors were extracted from a numerical model of a five-storey reinforced concrete building subjected to varying levels of the Imperial Valley earthquake (El Centro, 1940, Array #9) using different peak ground velocities (Pledger, 2026). These histories were then scaled to achieve the target inter-storey drifts specified in Table 1. Finally, the scaled motions for the third and second floors were applied to the top and bottom slabs of the test specimen, respectively. Figure 6 shows the scaled displacement input for Motion 14 as an example.

Testing of the rotationally "unrestrained" wall specimens (tested twice) concluded at Motion 10 (1.00 % inter-storey drift), while the "restrained" specimens were subjected to the full suite of motions in Table 1. Unless specified otherwise, the results and discussion in this paper focus on the "restrained" specimen.

3.4 Instrumentation

The test structure was instrumented with accelerometers placed on the top slab and beneath the bottom slab to record accelerations along the loading direction (the North-South, or X-axis). To measure structural deformations, the setup was equipped with Motion Capture reflector points, monitored by five cameras positioned facing the east elevation during testing. Additionally, the wall specimens themselves were instrumented with displacement potentiometers, accelerometers as shown in Figure 7 and MoCap reflectors at key locations to capture their detailed response.

3.5 Data processing

Recorded accelerations were filtered to reduce noise. A 4th-order low-pass Butterworth filter (50 Hz cutoff, 180 Hz sampling) (Butterworth, 1930) was applied to capture the specimens' local high-frequency response. A 4th-order 1-D median filter (window size 4) then smoothed impulse peaks by replacing each point with the local median, preserving signal edges with zero-padding. Displacement data were used unprocessed. For analysis, acceleration and drift data from Motion 0 were excluded due to negligible magnitude. In displacement plots, Motion 0 data are shown for completeness but omitted from discussion, as the measured values were insignificant.

Table 1:Input motions to the test setup

Motion	PFA[1] Bottom slab (g)	PFV[2] Bottom slab (m/s)	PFD[3] Bottom slab (mm)	PFA Top slab (g)	PFV Top slab (m/s)	PFD Top slab (mm)	Target ISD[4] (%)
0	0.01	0.01	2.70	0.02	0.02	3.40	0.03
1	0.04	0.04	9.64	0.05	0.05	12.14	0.10
2	0.08	0.08	19.29	0.11	0.11	24.29	0.20
3	0.12	0.12	28.93	0.16	0.16	36.43	0.30
4	0.16	0.15	36.55	0.21	0.21	46.21	0.40
5	0.20	0.19	45.69	0.27	0.26	57.76	0.50
6	0.24	0.23	54.83	0.32	0.31	69.31	0.60
7	0.28	0.27	56.00	0.32	0.37	72.47	0.70
8	0.32	0.31	64.00	0.37	0.42	82.82	0.80
9	0.36	0.35	72.00	0.41	0.48	93.18	0.90
10[5]	0.37	0.37	77.88	0.44	0.50	100.00	1.00
11	0.48	0.46	106.34	0.59	0.59	121.27	1.25
12	0.71	0.60	152.90	0.85	0.74	157.74	1.50
13	0.90	0.77	184.56	1.08	0.82	187.43	1.75
14	1.17	0.94	228.43	1.34	0.95	242.16	2.00
15[6]	0	0	0	0.36	0.42	82.13	2.25
16[6]	0	0	0	0.40	0.47	91.25	2.50
17[6]	0	0	0	0.44	0.51	100.38	2.75
18[6]	0	0	0	0.48	0.54	109.50	3.00

[1]Peak Floor Acceleration, [2]Peak Floor Velocity, [3]Peak Floor Displacement, [4]Inter Storey Drift,
[5]Final test for "unrestrained" wall specimens, [6]Bottom slab is locked into position

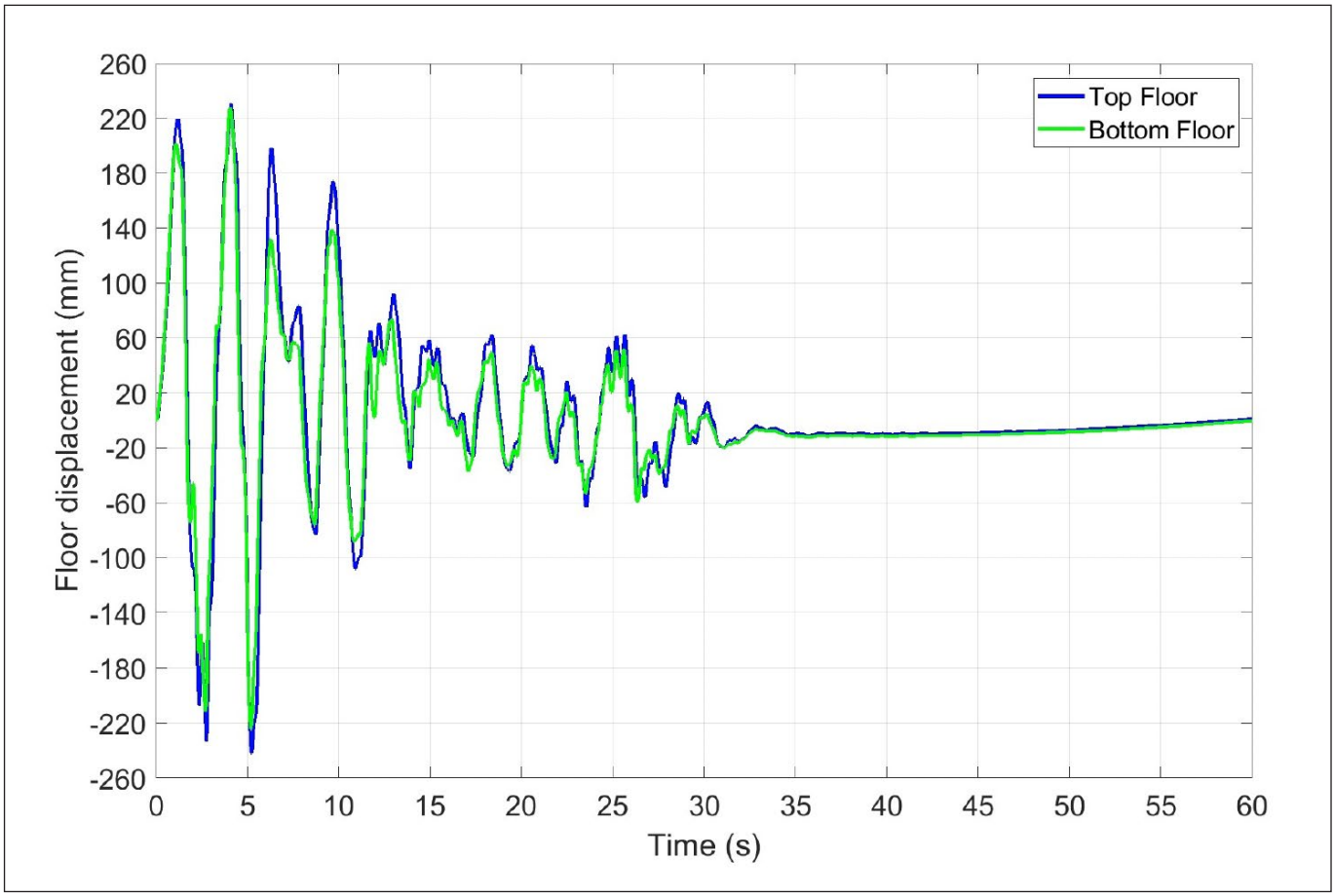


Figure 6. Floor displacement command time histories of top and bottom slabs for Motion 14

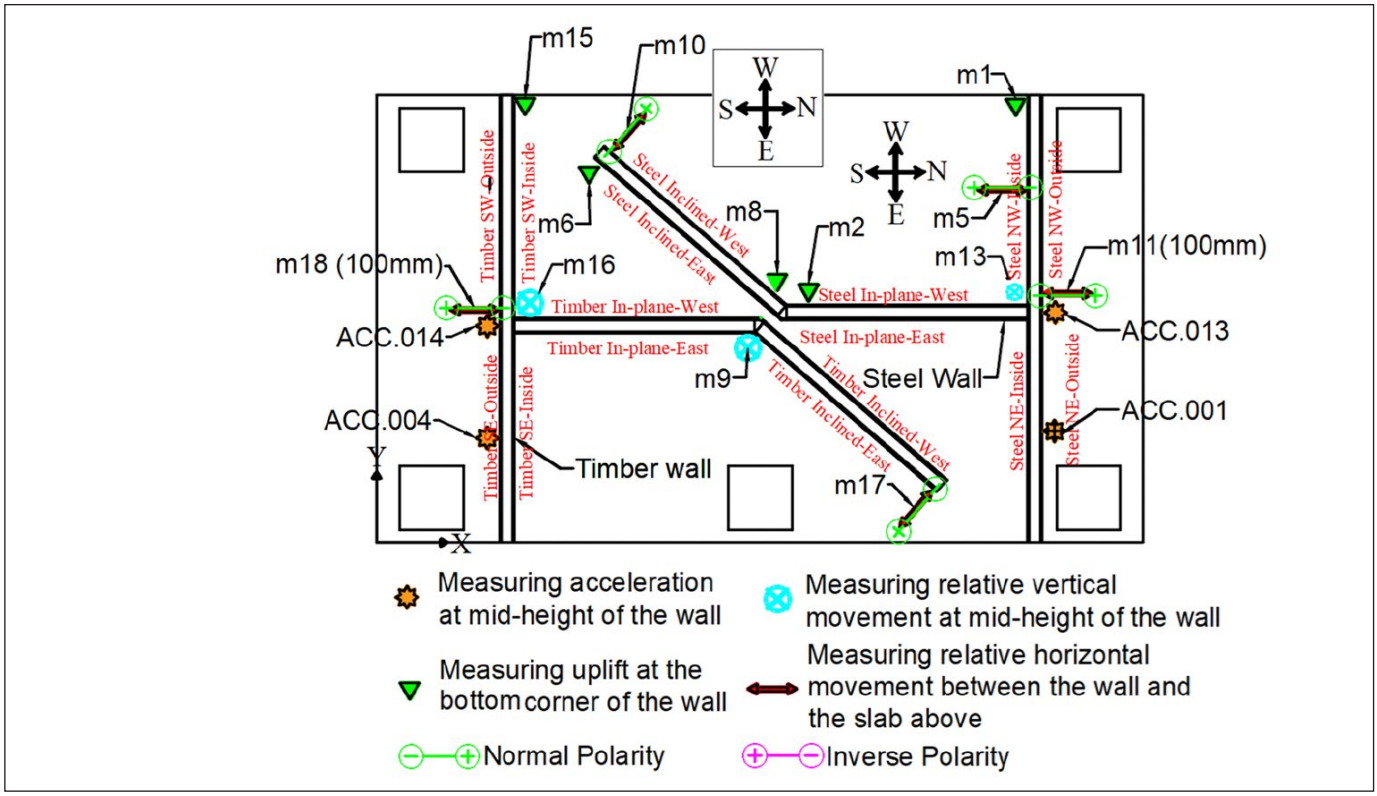
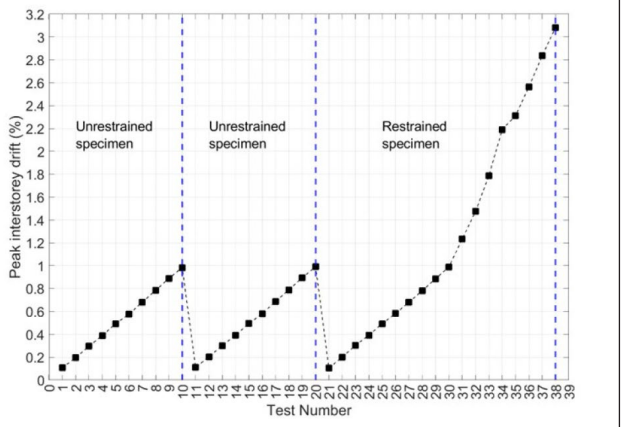


Figure 7: Instrumentation plan

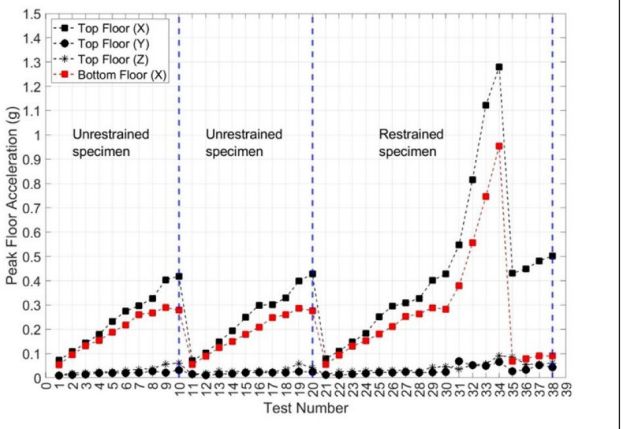
4. EXPERIMENTAL RESULTS

4.1 Response of the test structure

The recorded inter-story drifts (ISDs) and peak floor accelerations (PFAs) of the test-structure are shown in Figure 8. The measured peak inter-storey drifts derived from actuator data reached 0.98 % during two initial tests of the "unrestrained" specimen. Following the addition of out-of-plane restraints, the "restrained" specimen was tested up to a drift of 3.08 %. Acceleration data, averaged from post-processed floor-mounted accelerometers, showed that the top floor consistently experienced higher peak horizontal accelerations than the bottom floor. For instance, during Test #34, the top and bottom floor accelerations reached 1.28 g and 0.95 g in the loading direction (X), respectively.



a) Inter-storey drifts



b)Floor accelerations

Figure 8: Recorded test structure response

4.2 Component response

4.2.1 Damage observation to specimens and their damage state classification

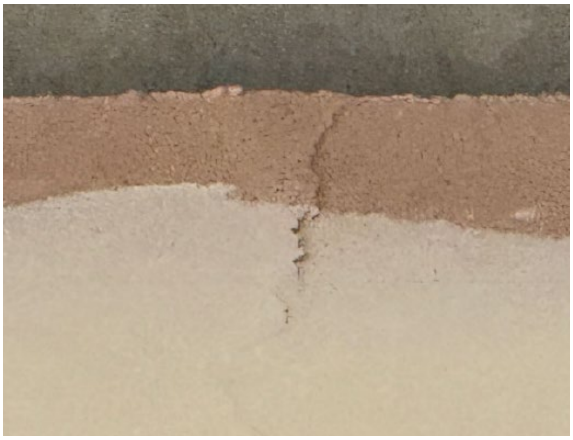
4.2.1.1 Timber Specimens

Damage in the "unrestrained" timber specimen, which was tested up to 1.00 % drift demand, initiated at a very low inter-storey drift of 0.11 % with the formation of hairline cracks at the wall T-junction. As drift increased, screw impressions became apparent at 0.39 %, followed by minor cracking in the plasterboard of the out-of-plane wall at 0.78 % drift; the cracking occurred where the DHT terminates running from the wall end - see Figure 3). This progression indicates that the specimen experienced gradual damage, with cosmetic and minor issues emerging early in the loading protocol.

The "restrained" timber specimen, which was tested to a much higher drift of 3.00 %, exhibited a different damage pattern influenced by prior testing. While hairline cracks in the plasterboard in the out-of-plane wall (where the DHT terminates running from the wall end) were noted at 0.30 % drift, these were considered progressive from the earlier "unrestrained" test. But more definitive new damage at that location, in the form of a crack in the plasterboard that could adversely affect the fire seal, was observed at 0.58 % drift (see Figure 9 (a)). This onset of damage may have been influenced by prior loading cycles associated with the "unrestrained" wall configuration i.e., flanges of DHT may have sustained some plastic deformation during the earlier loading cycles. Consequently, it is postulated the deformation capacity of 0.60 % could be considered a conservative estimate. The most significant damage occurred at 0.99 % drift, with the breaking of the top timber plates at the T-junction, a failure confirmed by loud sounds during testing and post-test inspection (see Figure 9 (b)). However, the effect on the wall response was not noticeable until afterwards. This sequence of damage shows that while restraint slightly delayed the onset of cosmetic damage, it ultimately led to a breaking of timber plate at a drift level comparable to where the "unrestrained" test ended.

4.2.1.2 Steel Specimens

For the "unrestrained" steel specimen, tested up to 1.00 % drift demand, initial damage appeared simultaneously at 0.20 % drift in the form of hairline cracks in the plasterboard (at T-junction) and screw impressions (the latter attributed to a construction error). The damage remained minor, with only a small tear in the joint paper observed at 0.89 % drift. The damage largely confined to finishes up to 1.00 % drift.



a)
Minor cracking in the plasterboard at 0.58 % drift



b)
Breaking of the top timber plates occurred at 0.99 % drift (photo taken during wall demolition after Phase I was completed)



c)
Separation of walls at steel wall junctions at 2.56 % drift

Figure 9: Photos of damage oberved in the specimens

The “restrained” steel specimen, tested up to 3.08 % drift demand, demonstrated a significant delay in initial damage. Screw impressions were not recorded until 0.99 % drift (occurred in the out-of-plane wall near T-junction), and a minor joint paper tear (at the T-junction) followed at 1.23 % drift. Substantial damage began at 1.79 % drift, with cracks in the plasterboard at the top of the out-of-plane wall (opposite to the T-junction) and tears in the plaster paper at T-junction, both noted as potential compromises to the fire seal. Further cracking occurred at 2.19 % drift. The defining failure event was the separation of the steel frames at the wall junctions (can compromise occupancy) which occurred abruptly at 2.56 % drift (see Figure 9 (c)). This indicates that the restraint system suppressed early, minor damage but resulted in the accumulation of internal forces, leading to a sudden and brittle connection failure at high drift.

Davies et al. (2011) reports that in commercial H-shaped walls with slip tracks, such damage occurred at lower median drifts: screw impressions at 0.26 %, plasterboard cracking at 0.68 %, and frame damage/connection failure at 0.75 %. This benchmark highlights the significant improvement in the seismic resilience offered by the “No-Fix zones” detail in partition walls.

4.2.2 Peak component accelerations and dynamic amplification factors

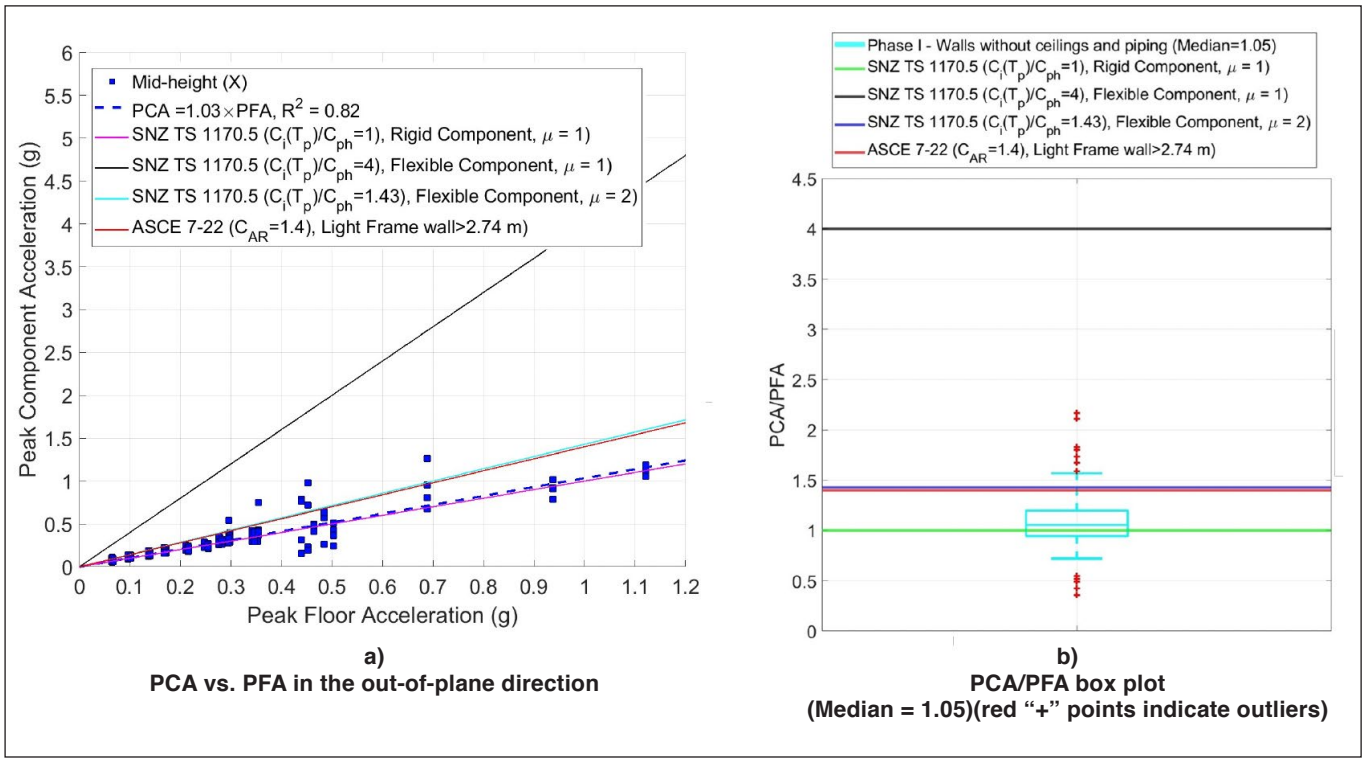
Peak component accelerations (PCA) in the out-of-plane (X) direction at wall junctions and at mid-length of the overhang are plotted against the average peak floor acceleration (PFA) (of the top and bottom slabs) for all 38 tests of the timber-and steel-framed walls in Figure 10 (a). Accelerometers were located at the mid-height of the out-of-plane walls at the junctions and at a return overhang for both specimens. The box plot of all the PCA/PFA ratio for the walls, calculated by dividing the PCA by the average PFA of both floors is shown in Figure 10 (b) .

Figure 10 also shows PCA values calculated by multiplying the average PFA from both top and bottom floors by: (1) the ratio $C_i(T_p)/C_{ph}$ for various ductility levels (μ) as per SNZ TS 1170.5 (2025), where $C_i(T_p)$ is the component spectral-shape coefficient and C_{ph} is the component horizontal-response factor, and (2) the Component Resonance Ductility Factor (C_{AR}) from ASCE/SEI 7-22 (2022). A linear fit through the origin is included alongside the experimental data in Figure 10 (a). The experimental PCA/PFA ratio (Dynamic Amplification Factor, including ductility μ) for out-of-plane light-frame walls is approximately 1.03 (R^2=0.82). This result aligns with the SNZ TS 1170.5 (2025) design cases for a Rigid component (μ = 1) or a high-ductility Flexible component ($\mu \geq 2.5$). Both standards appear to provide conservative estimates for these test results: SNZ TS 1170.5 (2025) for μ = 2 exceeds the measured ratio by a factor of 1.39, while ASCE/SEI 7-22 (2022) does so by a factor of 1.36. The median experimental PCA/PFA value for light-framed walls was found to be 1.05, as shown in Figure 10 (b).

4.2.3 Peak component lateral displacement

Figure 11 displays the absolute maximum lateral displacements of the timber wall specimen relative to the top slab, measured at various locations during the “restrained” test series.

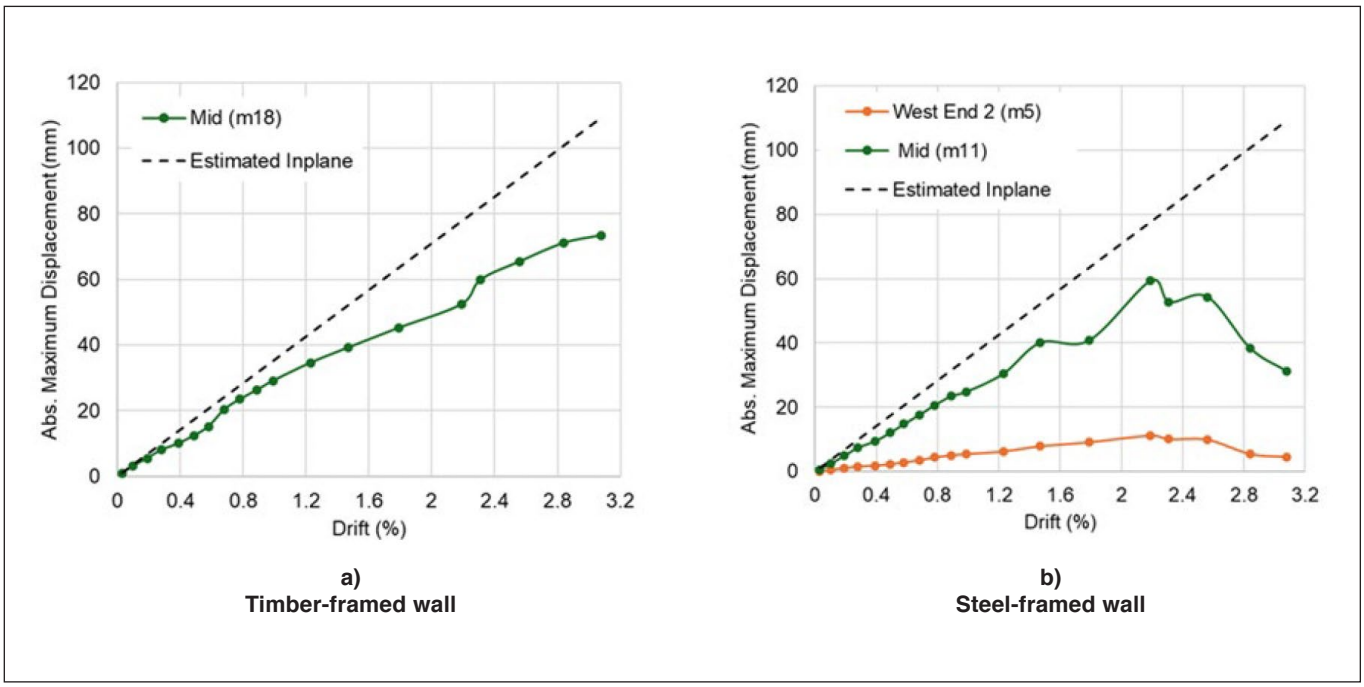
As shown in Figure 11 (a), the maximum



a)
PCA vs. PFA in the out-of-plane direction

b)
PCA/PFA box plot
(Median = 1.05)(red "+" points indicate outliers)

Figure 10: Peak component accelerations for wall specimens in out-of-plane direction (X)



a)
Timber-framed wall

b)
Steel-framed wall

Figure 11: Absolute maximum lateral displacements relative to top slab for wall specimens (approximate measurement locations shown and instrument numbers in brackets – refer to Figure 7)

displacements at the wall junction (mid-height of the out-of-plane wall) remain below the estimated in-plane displacements between the top slab and the deflection head track (calculated by multiplying the height of the wall and the drift), with a gradual divergence suggesting accumulating specimen damage.

In Figure 11 (b), junction displacements similarly remain below the estimated values until exceeding 2.20 % drift, where a sharp divergence occurs. This shift is attributed to separation between the in-plane and out-of-plane walls, allowing the out-of-plane wall to move with the top slab.

5. CONCLUSIONS

This report presents the preliminary findings from partition wall tests on the wall designs currently adopted by the Aotearoa New Zealand construction industry. Two 3.55 m-high wall specimens, one with timber framing and the other with steel framing, were constructed with fire-rated plasterboard lining. Both walls were tested under dynamic loading conditions. Lateral sliding was enabled through a deflection head track (DHT) at the top, while out-of-plane flexibility at the wall junctions was provided by deformation of the timber top plates (in timber walls) and the DHTs (in steel walls). To facilitate this deformation, anchors at the junctions were set back 1.2 m from the wall ends, creating designated “No-Fix zones”.

The experimental results indicated that walls with (“unrestrained”/“pinned”) ends deformed as intended and exhibited no damage that compromised fire rating when tested up to a maximum drift of 1.00 %, while the cosmetic damage in the form of hairline cracking at wall junctions occurred between 0.11 % (in timber wall) and 0.20 % drift (in steel wall). When the wall ends were rotationally “restrained”, effectively converting them to “fixed” ends, plasterboard crack damage compromising the fire-rating was observed at 0.60 % drift in the timber wall and at 1.80 % drift in the steel wall. Damage requiring major repair (with high priority) was deemed to occur at 1.00 % drift in the timber wall due to cracking of the top plate, and at 2.50 % drift in the steel wall due to separation at the junctions which can compromise occupancy. It should be noted that damage progression in the “restrained” walls is influenced by prior “unrestrained” testing, meaning the drift values at which damage appears are likely lower-bound estimates. When compared with traditional commercial walls, significant improvement in the seismic resilience was offered by the “No-Fix zones” detail.

Moreover, the median dynamic amplification factor for the tested wall specimens was found to be about 1.05, which corresponds to the design cases for a Rigid component ($\mu=1$) or a Flexible component with high ductility ($\mu\geq2.5$) as per SNZ TS 1170.5 (2025). The SNZ TS 1170.5 (2025) provision for a Flexible component with $\mu=2$ and ASCE/SEI 7-22 (2022) provide a conservative estimate, exceeding the measured ratio by a factor of 1.30.

The displacement potentiometer recordings of the walls demonstrated significant out-of-plane flexibility around the junction regions (No-Fix zones). As a result, potential deformation incompatibilities are anticipated with attached or adjacent elements such as suspended ceilings, façades, doors, windows, service penetrations, and nearby structural members (e.g., columns or structural walls), requiring further investigation. A detailed dynamic characterisation and displacement analysis of wall deformations are to be carried out using Motion Capture Camera recordings in future publications.

6. ACKNOWLEDGMENTS

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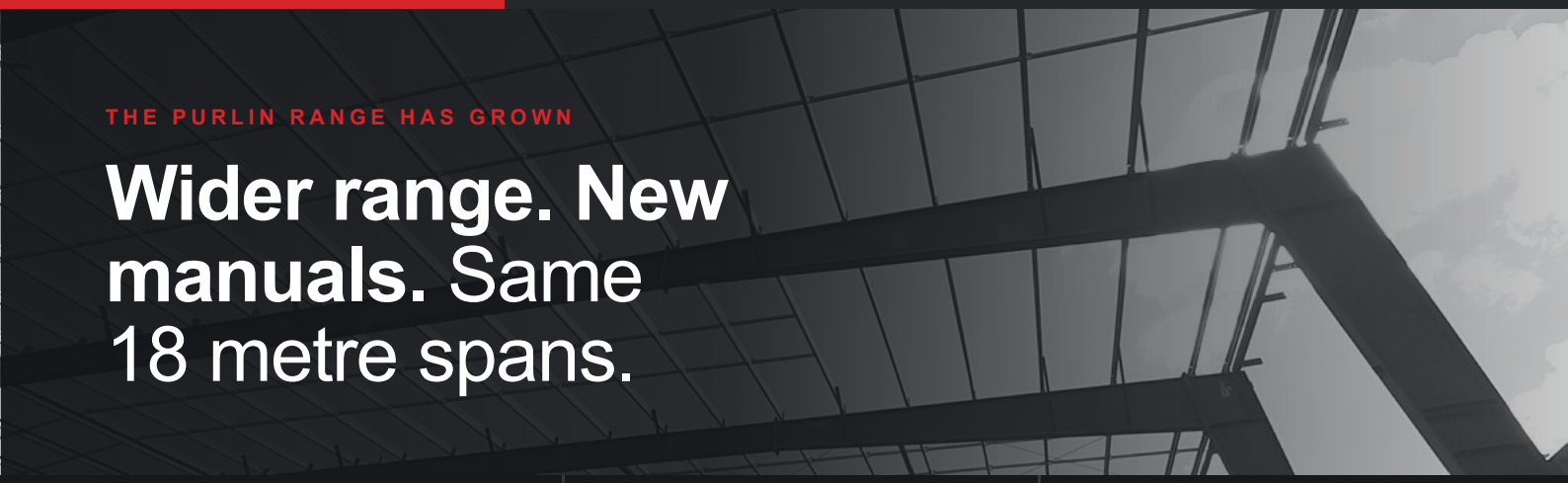
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LESSONS LEARNED AND OUTLOOK FOR TIMBER BRIDGE DESIGN IN AOTEAROA NEW ZEALAND

Douglas, C.[1], Moroder, D.[1], Buchanan, A.[1], Rogers, R[2]

Abstract

This paper presents lessons learned from recent timber bridge designs in Aotearoa New Zealand in the context of growing interest in lower embodied carbon transport infrastructure. Several recent timber bridge projects are reviewed, highlighting key design considerations and construction findings. Based on these insights, guidance is provided for engineers when designing timber bridges. The paper also includes a preliminary feasibility assessment tool to support early-stage screening of timber bridge suitability, and a discussion on opportunities for innovation in future timber bridge projects is presented.

1. INTRODUCTION

Engineered Wood Products (EWPs) are increasingly being adopted in Aotearoa New Zealand as a construction material for vertical structures. Attention is now shifting toward the use of timber in horizontal infrastructure as part of wider efforts to reduce the carbon footprint of transportation networks (NZTA, 2025). The Modern Timber Bridge focus group was established by NZ Transport Agency Waka Kotahi to drive innovation in bridge construction and promote timber as a practical additional construction material alongside traditional concrete and steel systems (Netley, 2022).

The environmental benefits of timber bridges stem from the low embodied energy of structural timber and the ability of timber components to store biogenic carbon throughout the life of the bridge. Most of the embodied energy in wood originates from solar energy captured during tree growth. Embodied energy in timber bridges results from carbon dioxide (CO_2) emissions from harvesting, drying, processing, adhesive manufacture, preservative treatment, transport, construction, maintenance, and end-of-life processes.

These emissions are much lower than those associated with comparable steel and concrete construction. Steel manufacturing requires burning of fossil fuel to provide process energy. Manufacturing of cement not only requires burning of fossil fuel but also releases CO_2 into the atmosphere as calcium carbonate ($CaCO_3$) is converted to quicklime (CaO). For this reason, the more timber replacing concrete and steel, the lower the carbon footprint of the finished bridge.

2. BACKGROUND

2.1 History

Many timber bridges were constructed in Aotearoa New Zealand prior to the 1950s (Phillips, 2024, Thornton, 2001). At least 14 of these structures remain in active service on the state highway and local road networks, with ages now ranging from approximately 80 to 100 years (NZTA, 2024a). The long-term performance of these bridges demonstrates that timber can be a durable and reliable bridge construction material when appropriately selected and maintained. However, these historic bridges were predominantly constructed from naturally durable old-growth timber that is no longer available. They differ substantially from modern timber bridges, which are typically constructed from radiata pine EWPs, relying on preservative treatment and careful detailing to achieve long service lives.

The negative reputation of poor durability in Aotearoa New Zealand's timber bridges in recent years is not entirely justified, as it is largely influenced by the old age of the remaining structures and the lack of new timber road bridge construction in recent decades. Existing timber road bridges are now approaching the end of their service lives and were generally designed to standards well below those expected of a modern bridge.

Modern engineered timber bridges differ fundamentally from historic timber bridges and can provide competitive and attractive solutions for many Aotearoa New Zealand applications, with benefits including durability, prefabrication, constructability, and structural performance (Singh and Page, 2018, Pousette et al., 2017).

PAPER CLASS & TYPE: GENERAL REFEREED

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[2] *NZ Transport Agency Waka Kotahi*

2.2 Compliance

The current NZTA Bridge Manual, Edition 3 (NZTA, 2022b) provides limited guidance on best-practice timber bridge design. Although an Interim Technical Standard for the design of timber bridges (#22-02) (NZTA, 2022a) was released in 2022, a timber specific Technical Advice Note (TAN) (NZTA, 2024c) is yet to be issued. It will be accompanied by the New Zealand Timber Bridges Design Guide, which remains in draft at the time of writing (NZTA, 2026).

This paper was prepared as a supplement to this upcoming guide and is based on recent bridge designs. Additional insights were drawn from the International Conference on Timber Bridges held in Rotorua, New Zealand in 2025 (2025).

3. AOTEAROA NEW ZEALAND DESIGN & CONSTRUCTION INSIGHTS

The discussion in this section is based primarily on papers covering these bridges presented at the 2025 International Conference on Timber Bridges (ICTB) in Rotorua. It presents key lessons from recent timber bridge designs in Aotearoa New Zealand, including material selection, detailing, construction methods, performance outcomes, and challenges.

3.1 SH 26 Onetai Stream Bridge Replacement, Waikato

The Onetai Stream Bridge, located on State Highway 26 between Paeroa and Thames, was selected as the first pilot project under the NZTA Modern Timber Bridge initiative. One of the criteria required this pilot bridge to be designed for a 100 year service life, for which there is limited available guidance, particularly in relation to durability.

The existing bridge had steel I-beam girders with a nail-laminated timber (NLT) deck. The replacement structure consists of 675 x 225 mm GL10 glulam girders spanning 9.0 m with 100 mm grade F14 re-laminated plywood deck, and serves two traffic lanes with a total width of 11.2 m. To achieve the required freeboard for flooding in the Onetai Stream, several shallower beams had to be used, which reduced structural efficiency. Stainless steel fasteners and brackets were used to transfer forces from braking, flooding, and seismic actions (Douglas et al., 2025). Demolition of the existing bridge, construction of the new scour protection, installation of the prefabricated abutments and bridge modules, and installation of the decking and final pavement were completed over a four-week period, and the bridge was opened in March 2025; see Figure 3-2. The bridge is monitored with moisture meters and a vertical accelerometer.

Construction tolerances for moisture related movement were accommodated with Drossbach ducts for abutment brackets and intentional gaps between plywood deck panels. As timber bridge construction is still relatively new to many contractors, there is still a learning curve around appropriate on-site storage and handling. In this case, storage conditions may have contributed to timber deformation, which was noted pre-installation and required additional work during installation. Storing the timber members on a level surface with sufficient ventilation would likely have prevented distortion of some elements.

The timber superstructure reduced the permanent load on the foundation, but this benefit was counteracted by the high buoyancy and overturning moment induced from flood loadings. The pavement crown shown in Figure 3 1 resulted in additional superimposed dead load and a more complex installation of the paving material, but it avoided an additional longitudinal joint in the plywood deck, susceptible to water ingress.

Top-down fasteners were used to connect the deck to the girders, requiring the holes to be plugged with an epoxy emulsion for waterproofing. To protect the girders from external moisture, the top and side faces were protected with multiple layers of bitumen emulsion.

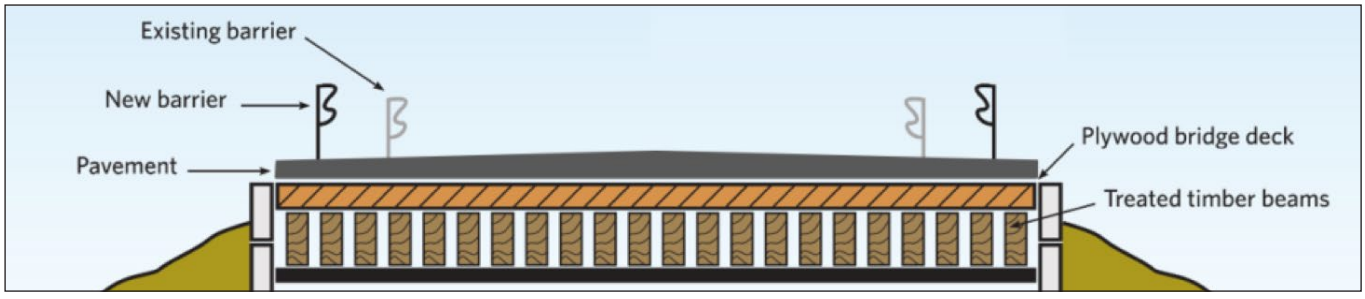


Figure 3 1. Bridge replacement cross section (modified from NZTA (2024b)).



Figure 3 2. Timber girders at the factory (left) and completed bridge (right) (Douglas et al., 2025).

3.2 Dallington Bridge, Ōtākaro/ Avon River, Otautahi Christchurch

The Dallington pedestrian and cycle bridge on the Ōtākaro/Avon River Corridor in Christchurch was opened in September 2025. It is a 30 m clear span, 3.65 m wide glulam arched girder bridge in the Christchurch "red zone", an area significantly impacted by soil liquefaction and lateral spreading during the Canterbury earthquakes.

To save costs related to block gluing of large glulam members, the girders were mechanically jointed, necessitating analysis of the effective built-up section using the γ-method in EN 1995-1-1 (CEN, 2004). The two built-up glulam girders formed from double 2000 × 225 mm GL10 billets were connected with stainless steel coach screws with glulam secondary beams supporting the permeable Fibre Reinforced Polymer (FRP) decking (see Figure 3 3). The main girders were designed to resist axial loads induced by lateral spreading at the riverbanks (Campbell and Macpherson, 2025). The bridge is monitored with moisture meters.

3.3 Kakanui Point Bridge, Waitaki

A replacement road bridge over the Kakanui River, south of Oamaru, was proposed to be constructed from timber EWPs. The concept design was a 162 m long structure with 15.6 m internal spans and 10.7 m end spans, utilising 990 x 180 mm glulam girders with a 140 mm thick plywood deck (see Figure 3 4). The girders incorporated novel triangular corbels on their side faces to provide moment continuity over the piers, for rapid installation and improved structural efficiency with reduced member depth. This approach would have avoided moment resisting connections such as epoxied rods or dowels in slotted steel plates, both of which are costly to install and might compromise long term durability. A sloping deck was utilised to minimise pavement weight. The low self-weight of timber offered no significant benefit in the foundations, because design was governed by flood loads rather than superstructure mass (Kivell et al., 2025).

This timber bridge was not built because the Waitaki District Council elected to procure a hollow-core concrete option with fewer piers, priced lower than the timber option due to fewer piers, lower foundation costs, contractor familiarity with concrete, and the perceived risk associated with timber construction (Kivell et al. (2025).

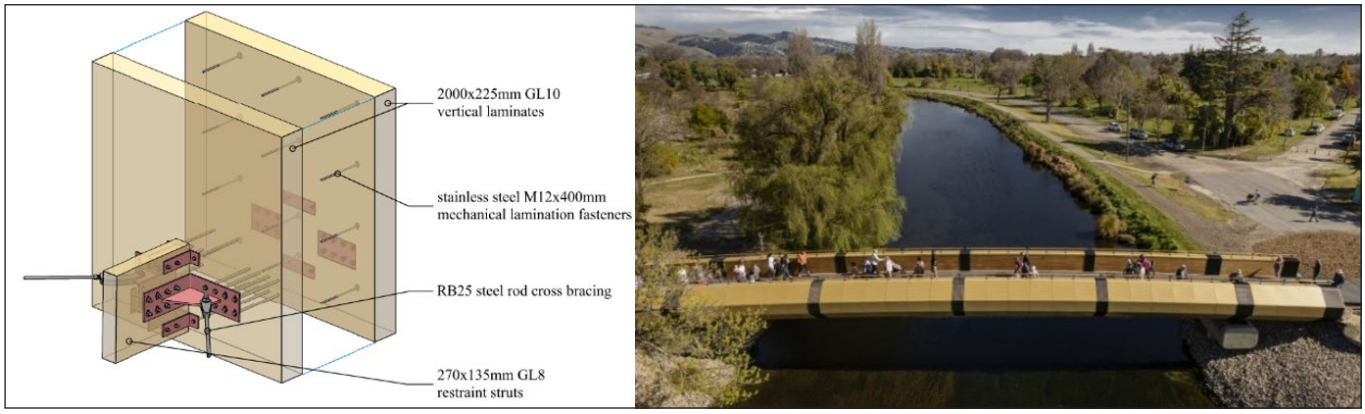


Figure 3 3. Schematic of mechanical lamination fixings (left) (Campbell and Macpherson, 2025) and Dallington Bridge in use (right) (Christchurch City Council Newsline, 2025).

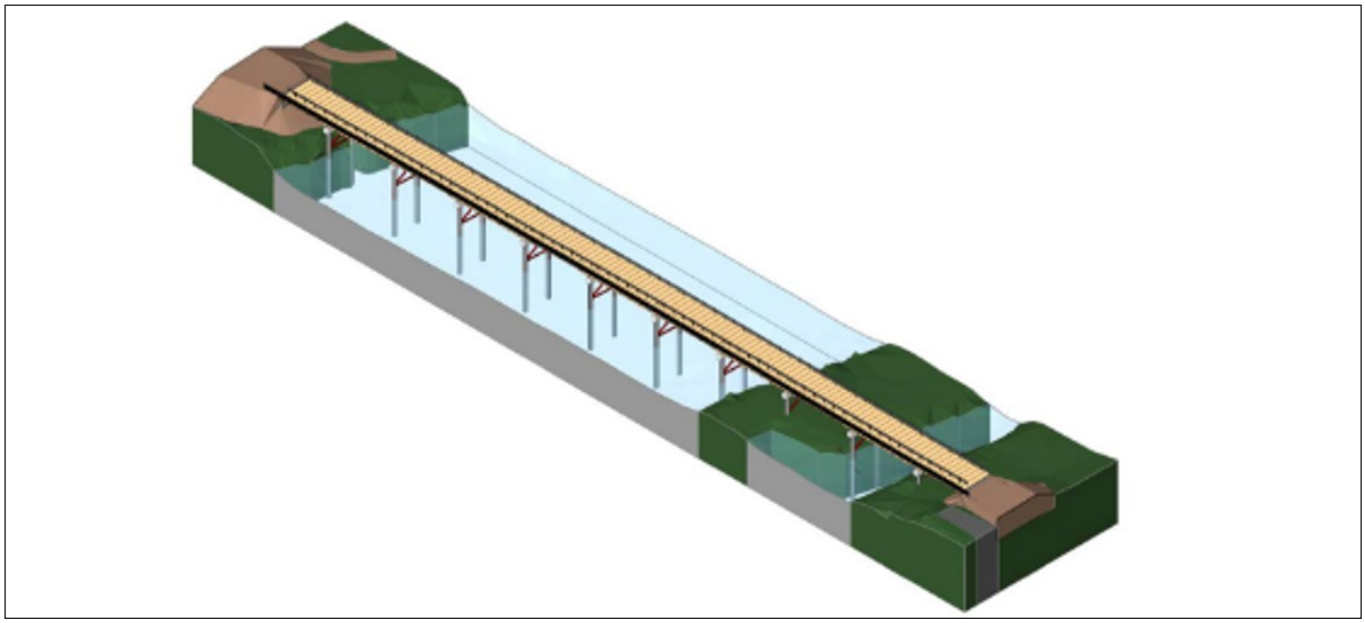


Figure 3 4. Isometric image of the Kakanui Point replacement bridge (Kivell et al., 2025).

3.4 SH 1 Papakura to Drury Shared Use Path (SUP) Bridge

The proposed Papakura to Drury (P2D) project featured a skewed multi-span 60 m timber truss bridge over SH1 at the Papakura Interchange, with a 30 m main span and two 15 m approach spans. The Shared Use Path (SUP) provides a safe separation for pedestrians and cyclists over vehicular traffic (Pham et al., 2025). The proposed design has 450 x 450 mm glulam top chords, 270 x 270 mm glulam diagonals, glulam cross-girders and sway braces, tension-only vertical members, and a steel bottom chord with 135 mm thick glulam decking panels (see Figure 3 5).

The steel bottom chord was selected to resist tension forces more effectively than timber. The main connections are slotted stainless steel fin plates and dowels. To improve durability, all gaps are sealed with a caulking compound, reinforcing screws mitigate splitting perpendicular to the grain, and cap flashings will be applied to exposed timber top faces. At the time of writing, construction of the bridge has been delayed due to financial constraints.

4. SUITABILITY OF TIMBER FOR BRIDGE CONSTRUCTION

In summary, the advantages of timber for bridge construction include the following:

- Readily available construction materials from local renewable forests, although supply, lead times, manufacturing capacity, and transport logistics should be confirmed early.
- Lightweight timber superstructure, giving savings in foundations and abutments, if these benefits are not offset by flood loading or buoyancy effects.
- Accurate lightweight prefabricated timber components requiring small construction tolerances and low-capacity cranes.



Figure 3 5. Final proposed bridge (Pham et al., 2025).

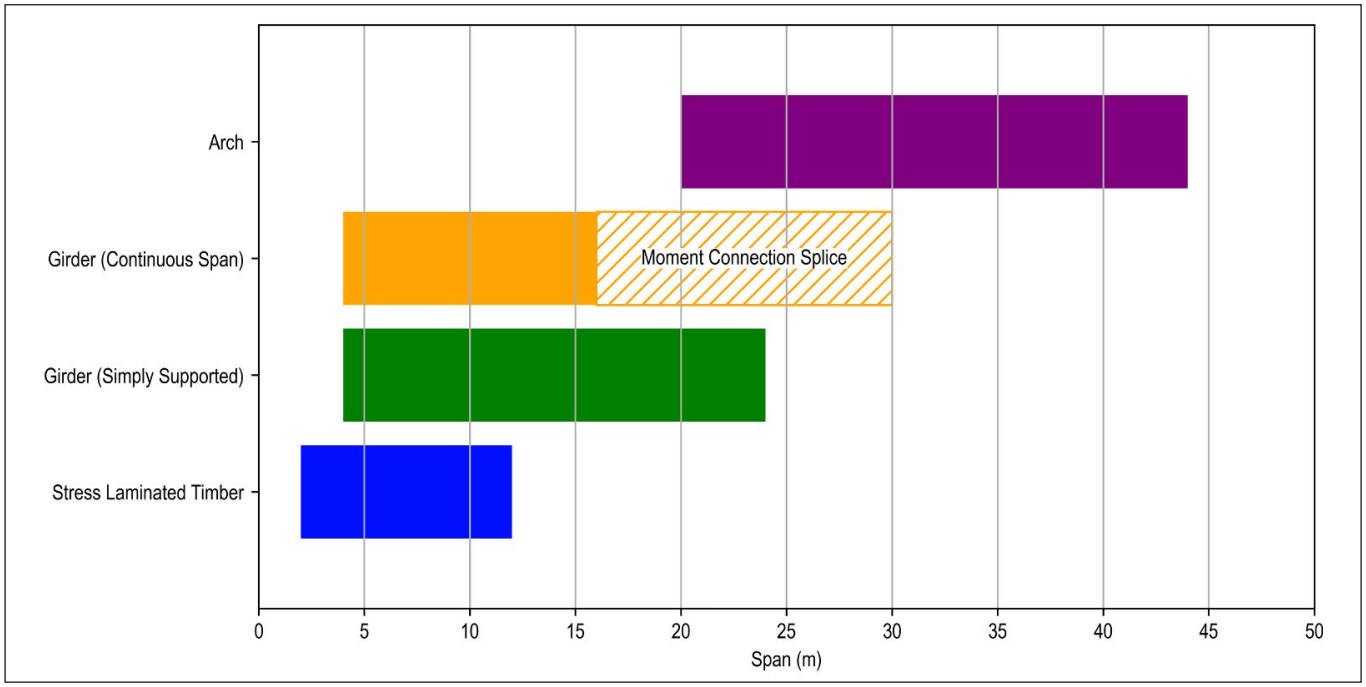


Figure 4 1. Typical timber bridge types and spans (NZTA, 2026).

Note:

- *Figure 4-1 was made based on HN/HO type vehicle loadings and does not consider pedestrian loadings. However, for preliminary design purposes, pedestrian bridge spans may be assumed to be approximately 25% greater than those shown.*
- *The maximum spans are based on glulam girder lengths of up to 32 m and glulam arch lengths of up to 22 m, reflecting manufacturing and transportation constraints. For continuous spans exceeding 16 m, connection splices are required to achieve moment continuity. Moment connections, although possible, can introduce additional cost and affect durability.*

Based on prior timber bridge designs, the following recommendations can guide engineers on the most suitable applications for timber as a bridge material.

4.1 Recommended guidance

- Use simple structural forms, such as girders or arches. Complex forms, such as trusses, are generally more difficult to construct and maintain, often require a greater number of critical connections, and may result in higher construction costs.
- Use simply supported or continuous girders for short to medium spans (typically up to 15 to 20 m in Aotearoa New Zealand practice). Longer spans can be achieved using arches (see Figure 4 1).
- Use structural timber elements primarily in bending or compression, with direct bearing wherever possible, minimising the number of shear, moment, and tension connections.
- Combine timber components with steel elements to transfer tension in arch hangers or trusses.
- Prevent timber from getting wet using decking, pavement, membranes, or flashings. Provide detailing to allow wet wood to dry naturally.
- Although timber can be used for HN and HO loading as shown in Figure 4 1, its optimal application is often in bridges with reduced loading, such as:
  - Rural bridges with reduced loading.
  - Pedestrian and cycle bridges.
- A preliminary feasibility assessment tool to support early screening of timber bridges is provided in the Appendix

4.2 Situations to Avoid

Designers of timber bridges should avoid:

- Structures relying on tensile connections in timber members.
- Structures with low structural redundancy, such as complex timber truss systems that rely on numerous timber tension connections.
- Excessive screw reinforcement in connections, which could reduce net tension capacity.

- Large spans with limited freeboard because timber's low self-weight can be outweighed by high uplift forces from potential flooding.
- Timber structures with many critical connections that will increase maintenance demands, because comprehensive data and historical maintenance records are limited.
- Connection details which act as moisture traps.
- Rigid side barriers, especially types TL4 and TL5.

4.3 Key Considerations

Durability:

- Timber can cope with wetness, but it must be designed to dry. Avoid enclosing or sealing the wood and allow for any trapped water to escape.
- Timber that is directly exposed to weathering (rain and sunlight) should be protected using sacrificial cladding to reduce deterioration and extend service life.
- Drainage should be detailed to effectively shed water away from the structure, preventing moisture accumulation and associated deterioration. Particular attention should be given to the interface between the wearing surface and the supporting timber deck.
- Chemical timber treatment should not be considered as an alternative to good detailing, but chemical treatment is necessary for non-durable species such as Radiata pine.
- Achieving 100-year durability needs to be case-specific and requires careful considerations.

Construction and handling:

- Timber must be stored in dry conditions, with members placed on level ground prior to installation to prevent twisting and warping which can affect construction tolerances.
- The design should recognise that prefabricated timber components can generally be manufactured to tight tolerances, but that moisture-related shrinkage and swelling can affect dimensions during storage, transport, and installation.
- The concrete substructure receiving these components may also have large construction tolerances. Interfaces between timber and concrete should allow for practical adjustment during installation, including tolerance gaps, slotted or oversized fixings, packers and the like.

Design:

- If fastener corrosion is a potential concern, larger diameter fasteners should be used for improved durability. However, this may affect the ductility of the joint compared with many smaller diameter fasteners. Where replacement or maintenance costs are expected to be high, stainless-steel fasteners should be specified.
- Design should account for deformation in connections rather than assuming rigid behaviour.
- Creep should be included when predicting long term deformation (i.e. k2 in NZS 3603 or j2 in AS 1720.1).
- Brittle failure modes should be checked when designing connections.
- Fatigue loading should be considered in critical connections, where there is low structural redundancy, and for heavily loaded railway-type bridges.
- For typical highway bridges, timber fatigue is generally not expected to govern member sizing, although fatigue verification may still be required for critical connections and other specific applications (Canadian Wood Council, 2024, NZTA, 2026).

4.4 Current Barriers to Timber Bridge Implementation in Aotearoa New Zealand

Based on the outcomes of several Modern Timber Bridge Focus Group meetings and material workshops led by NZTA, the following barriers to wider timber bridge implementation have been identified:

- Fundamental Limitations:

  Timber bridge structures typically have shorter span capabilities than comparable concrete or steel options, often necessitating additional intermediate piers hence more extensive foundation works, which can significantly increase project costs. Many stakeholders seek to avoid or minimise the placement of piers within waterways due to environmental, ecological, and maintenance considerations. For Māori stakeholders, this may also have cultural significance in relation to the health and mauri of the waterway. In addition, timber bridges may require more frequent inspection and maintenance over their service life, contributing to higher ongoing whole-of-life costs.

- Cost Competitiveness and Market Confidence:

  Timber bridges are often perceived as more expensive than traditional steel or concrete alternatives. This reflects established supply chains, greater contractor familiarity and confidence in conventional materials, and the decades of iteration, optimisation, and standardisation that have improved the efficiency and cost certainty of steel and concrete bridge designs. In comparison, the limited modern timber bridge market and project experience often result in higher risk allowances for timber options.

- Limited Local Precedents and Standardisation:

  Aotearoa New Zealand has relatively few examples of modern timber road bridges, which limits industry experience and slows adoption. Each new timber bridge is a bespoke design, so needs an engineer with specialised timber knowledge, whereas conventional concrete bridges can be designed based on standard span tables and established cost estimates. Development of standardised timber bridge systems would help improve consistency, reduce design effort, and lower construction risk.

- Need for Broader Engagement with Stakeholders:

  Progress in timber bridge implementation is currently largely dependent on NZTA-led initiatives. Broader engagement with bridge asset owners such as local councils as well as policymakers would raise awareness, align sustainability goals, and encourage adoption at regional levels.

- Tendering process and procurement:

  Timber is rarely considered during the design phase unless it is specifically requested in the tender documents. Recent bridge procurements have followed a Design-and-Build model, where contractors typically favour conventional construction methods over timber solutions.

- Lack of information on material compatibility

  Several aspects of material compatibility remain uncertain, including the performance of membranes, expansion-gap treatments, and pavement materials used on timber surfaces.

5. OPPORTUNITIES FOR INNOVATION

Timber presents a unique opportunity for innovative and sustainable bridge design. This section highlights areas where timber bridges can deliver environmental and economic benefits through emerging technologies, advanced materials, and modern monitoring practices. It also summarises areas where further research and development may be required to strengthen and expand the use of timber in bridge construction in Aotearoa New Zealand.

5.1 Emerging Technologies

Several technological advancements in timber bridge design and construction have emerged in recent years. A selection of these emerging technologies is outlined below.

- Modular Prefabrication:

  A key benefit of using timber is its high strength-to-weight ratio, which supports modular construction. This enables faster on-site assembly and reduces disruption to road users. For example, a 67.5 m span pedestrian bridge in Brussels, Belgium, constructed as a box girder, was fully prefabricated off-site and installed in a single night, see Figure 5 1 (Néron et al., 2025).

- Glulam Arches with Steel Frames:

  Timber exhibits its highest strength and stiffness in compression parallel to the grain. Using an arch as the primary structural system takes advantage of this, providing an efficient load path. The deck can then be suspended from the arch using vertical steel hangers, while lateral restraint is achieved through a steel frame designed to resist moment at the vertical–horizontal connection, exemplified by the Skogsrud Bridge in Norway (see Figure 5 2).

- Stress Laminated Timber:

  Post-tensioned Stress Laminated Timber (SLT) is an effective technique for combining sawn timber on-site to form a monolithic element, enhancing structural performance, as shown in Figure 5 3. SLT's suitability for modular construction adds to its versatility (Douglass et al., 2025). SLT is widely used in Sweden and Norway and has been codified in the Australian Timber Bridge Design Standard AS 5100.9 (Standards Australia, 2017). However, SLT requires ongoing maintenance, with re-stressing typically recommended every five years (Buchanan et al., 2025).

- Hybrid Construction:

  Hybrid structures using timber in association with steel or concrete can often achieve the required structural performance. Using timber in the secondary system, such as for the decking, offers a practical compromise that reduces the overall carbon footprint of the structure. Timber–Concrete Composite (TCC) systems offer another hybrid solution, combining the tensile strength of timber



Figure 5 1. Vier Armen intersection, Brussels, Belgium, 67.5 m span pedestrian bridge installation (Néron et al., 2025).



Figure 5 2. Skogsrud Bridge, Norway (Legg and Tingley, 2019).



Figure 5 3. Installation of timber planks prior to stressing of SLT bridge (Douglass et al., 2025).

with the compressive strength of concrete in the deck, which can then serve directly as part of the pavement system.

- Retrofitting/Strengthening:

  In some cases, existing timber bridges become unfit for purpose over time due to inadequate durability, undersized members, or general deterioration. Some retrofitting solutions combine Fibre-Reinforced Polymers (FRP) with timber members (WRD, 2023). FRP can be externally wrapped around elements, offering a corrosion-resistant and lightweight means of restoring or enhancing capacity. However, long term effects such as creep and moisture cycling can lead to losses in bonding or tensioning, so ongoing maintenance and inspection are required. If girder depth is critical, strength or deflection criteria can be achieved by integrating FRP between glulam lamellae or by using steel reinforcement (Gul et al., 2024, Prolam, 2025).

- Thick Structural Plywood:

  Historically, nail laminated timber (NLT) was commonly used for decking, but shrinkage and swelling of boards has allowed water ingress in some cases, leading to corrosion of fasteners and timber decay. Additionally, NLT often requires supplementary structural elements to achieve diaphragm action. In contrast, thick structural plywood decking, achieved by re-laminating thinner plywood sheets, provides a monolithic deck element that can be installed quickly, and it provides effective in-plane performance. Pre-treatment with chemical preservatives provides durability. This is an effective option for replacing decks on existing bridges, where its light weight often allows upgrades without modifying the existing foundations.

- Naturally durable Hardwood:

  Engineered wood products grown in Aotearoa New Zealand have historically been fabricated from Radiata Pine, a softwood that is susceptible to decay, necessitating treatment for all exterior timber construction. Naturally-durable hardwoods offer a promising alternative for timber bridges due to their higher density and natural oils, such as *Eucalyptus bosistoana* (Lezgi and Lim, 2025).

5.2 Monitoring

It is important to provide timely and accessible monitoring to designers and governing authorities to help optimise future designs. Key options for monitoring are outlined below.

- Moisture Content:

  The strength and stiffness of timber decrease as moisture content increases. Currently, there are only very limited in-service moisture content data for timber bridges, making it difficult for designers to accurately estimate structural performance. Better data would provide greater design certainty and allow optimisation for the intended location. Modern timber bridges, such as the Onetai Stream Bridge (Section 3.1) and the Dallington Bridge (Section 3.2) incorporate moisture meters to monitor moisture content in the timber members over time.

- Dynamic Performance:

  For timber bridge elements, a Dynamic Load Allowance (DLA) is applied to the static design load to account for the dynamic effects arising from vehicle–bridge interaction (Tingley et al., 2022). Timber's viscoelastic behaviour means its strength and stiffness increase under instantaneous loading. The NZTA Bridge Manual (2022b) specifies reduced DLA values between 0.15 and 0.30, depending on span for timber structures. In contrast, AASHTO (2020) does not require a DLA for timber structures, indicating that dynamic effects may be negligible. This difference highlights an opportunity to further optimise designs by directly measuring these effects.

- Maintenance and Repair Records:

  Only limited records exist on the performance of timber bridges constructed with engineered wood products (Singh and Page, 2018, Bier and Page, 1991). Feedback and maintenance documentation from timber-bridge asset owners is generally unavailable, meaning much of this practical knowledge is being lost over time. The most effective way to advance understanding is through systematic evaluation of existing timber bridges, identifying what has and has not performed well in practice.

6. KNOWLEDGE AND RESEARCH GAPS

The soon to be released NZTA Timber Bridges Design Guide and the accompanying TAN will together pave the way for an update to the NZTA Bridge Manual, providing a clear and robust compliance pathway for timber bridges designed for a 100 year lifespan. However, there are certain areas where more research and knowledge are needed. The following are needed:

- A suite of standardised timber bridge systems, including typologies, span tables, and detail libraries, to enable faster and more reliable business-case estimates, improve comparability, reduce risk allowances, and to streamline decision-making.
- Information on long term pavement performance, including shrinkage, waterproofing membranes, and deck detailing suitable for Radiata Pine and other common timber species.
- Development of crash tested TL 3/TL 4 barrier systems, along with reliable standard connection details to secure barriers to timber decks.
- A quality assurance pathway for 100 year durability of engineered wood products, including adhesive performance, treatment methods, and more robust delamination testing.
- Research into corrosion performance of metal components in contact with copper treated timber and effective long term isolation strategies.
- Protocols for design for deconstruction and re use of structural timber, including assessment and grading of salvaged glulam/LVL/plywood.
- Practical asset management guidance for modern timber bridges, covering inspection methods and frequencies, routine maintenance requirements, common repairs, and record keeping. A standard asset owner manual would help address limited owner experience and support consistent management over the bridge service life.

7. CONCLUSIONS

This paper presents lessons learned from recent timber bridge projects relevant to New Zealand conditions. Recommendations have been provided to guide designers on where timber bridges perform most effectively and where they are less suitable. Key barriers to wider implementation in New Zealand have also been outlined. Nonetheless, there are significant opportunities for innovation and sustainable advancement, particularly through emerging technologies, new materials, and enhanced monitoring practices.

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APPENDIX: TIMBER BRIDGE FEASIBILITY ASSESSMENT

This appendix can help asset owners, engineers, and contractors identify when timber bridges may be suitable in the New Zealand context. The assessment is informed by the lessons discussed in this paper and provides a practical basis for early project screening.

The assessment uses a weighted decision scoring approach. Users assign a score from 1 to 5 for each criterion against the relevant objective. The scores are added to give an objective subtotal percentage, which is then multiplied by the objective weighting.

The weighted objective results are then combined to give an overall assessment percentage. A higher percentage indicates that timber, used as the primary superstructure material, is likely to be more suitable for the proposed project. The interpretation guidance at the end of the assessment helps users understand the level of confidence associated with the outcome.

This assessment is a high-level decision support tool to inform, not determine, material selection. It does not fully address project specific cost implications, including procurement, supply chain maturity, construction risk, and whole-of-life maintenance. The scoring approach, criteria, and weightings should be reviewed with input from asset owners, engineers, contractors, iwi, suppliers, cost estimators, and other relevant stakeholders, and may change as further feedback, cost data, and project experience become available.

More information on timber bridges and the scoring criteria can be found in the upcoming New Zealand Timber Bridges Design Guide (NZTA, 2026).

SCORING	RESPONSE	MEANING
5	Strongly agree / Excellent fit	The condition clearly supports the use of timber
4	Agree / Good fit	The condition generally supports the use of timber
3	Neutral / Moderate fit	The condition is neither clearly favourable nor unfavourable
2	Disagree / Poor fit	The condition presents some limitations for timber
1	Strongly disagree / Very poor fit	The condition presents significant limitations for timber

A score of 0 may be assigned to any criterion where timber is not recommended based on this simplified scoring tool. A score of 0 does not preclude the use of timber, but it indicates that the proposal falls outside typical timber bridge applications and requires a project specific assessment by a bridge engineer and timber engineer before it is considered further.

PROJECT:

OBJECTIVE / WEIGHTING	CRITERIA	SCORING EXAMPLE	SCORE
Project suitability (Weight = 40%)	Is the bridge span within a practical range for timber systems (e.g., SLT, girder, arch)? Refer to Figure 4 1	Pedestrian bridge 12m = 5 (all options possible) Highway bridge 30m span = 2 (limited options available Highway bridge 45m span = 0 (not advised, requires specific assessment)	/ 5
	Is the site suitable for intermediate bridge supports without negatively affecting the waterway or traffic below? *Shorter spans are better suited for timber structures*	No piers required (based on Figure 4 1) = 5 Several piers are required, but the waterway is prone to carry debris = 2	/ 5
	Are there constraints on the depth of the superstructure to maintain clearance or freeboard?	No constraint = 5 Very limited freeboard due to high flood levels = 2	/ 5
	Is a TL4 or lower barrier system acceptable for the project? *Higher-capacity rigid barrier systems are generally difficult to achieve with timber structures. TL3 and lower is relatively easy to achieve, TL4 will need specific design consideration and TL5 is best avoided for timber bridges.*	Pedestrian and cycleway barrier required =5 Semi-rigid TL 3 barrier required = 4 Rigid TL5 barrier required = 0	/ 5
		Sub-total =	/ 20
		Objective Result = Sub-total % x Objective Weighting % =	%

Environmental conditions and seismic loading (Weight= 20%)	Would a lightweight timber superstructure provide significant benefits to the foundation costs? *Timber typically allows for simpler or smaller foundation systems.*	Poor ground conditions = 5 Good ground conditions = 3 Likely to be submerged, requiring hold-down mass = 1	/ 5
	Is the site located in a high seismic hazard region where a lightweight timber superstructure could provide significant benefits to seismic performance and potentially reduce substructure, foundation demands, and associated costs?	High seismic area = 5 Low seismic area = 1	/ 5
	Is the bridge superstructure unlikely to be loaded by flooding or debris? *Inundation of the deck due to flooding or high flows can increase design complexity and add cost for lightweight timber deck systems.*	Highway overbridge = 5 Bridge in a flood prone area = 1	/ 5
	Is the site free from adverse microclimate effects? *Constant shade, extremely high rainfall, a* *nd poor ventilation could accelerate deterioration in timber.*	Bridge located in an open area in a sunny part of NZ = 5 Bridge located in a shady valley in Fiordland = 1	/ 5
	Is the bridge exposed to corrosive agents? *Timber is not susceptible to corrosion, and stainless steel connections typically make up only a small proportion of the bridge materials. Stainless steel is recommended over hot dip galvanised steel in highly corrosive environments.*	Bridge location corrosion category C to NZS 3101 or C5 to NZS 3404 = 5 Bridge location corrosion category B1 to NZS 3101 or C3 to NZS 3404 = 1	/ 5
		Sub-total =	/ 25
		Objective Result = Sub-total % x Objective Weighting % =	%

Construction & logistics (Weight = 20%)	Are there good opportunities for prefabrication of timber elements off site?	Full prefabrication possible =5 No prefabrication possible = 1	/ 5
	Are site access, project location, and transport logistics suitable for the efficient delivery and installation of prefabricated timber components?	Site close to a main road artery with easy access = 5 Site accessed by a winding mountain road which only short trucks can access = 1	/ 5
	Is a short erection timeframe desirable? *Timber modules can be placed quickly with no curing time*	The bridge needs to be erected in only a few weeks = 5 The bridge has a generous construction timeframe = 1	/ 5
		Sub-total =	/ 15
		Objective Result = Sub-total % x Objective Weighting % =	%

Sustainability & carbon (Weight = 10%)	Are lower carbon construction materials or carbon reduction outcomes a project objective?	Sustainability is a main project driver = 5 Sustainability is not part of the project brief = 1	/ 5
	Can timber members be sourced locally or transported efficiently from a port or supplier?	The site is close to a timber manufacturing plant = 5 The site requires material to be shipped from outside NZ, and the closest port is far away = 1	/ 5
		Sub-total =	/ 10
		Objective Result = Sub-total % x Objective Weighting % =	%

Stakeholder & regulatory (Weight= 10%)	Have local community and iwi expectations been considered, and is timber viewed positively?	Local iwi has a preference to use locally sourced timber = 5 No stakeholder has a preference on material used = 1	/ 5
	Is the client or asset owner comfortable with the maintenance requirements associated with timber structures, including potentially increased inspection frequency and periodic tightening or replacement of fasteners?	Regular maintenance and inspection are accepted = 5 Low maintenance and infrequent inspections are paramount = 1	/ 5
		Sub-total =	/ 10
		Objective Result = Sub-total % x Objective Weighting % =	%
		Assessment result (sum of Objective Results) =	%

SCORE	OUTCOME
75% - 100%	**Timber should be actively considered**
50% - 74%	**Timber may be suitable subject to optimisation and project specific review**
< 50%	**Timber is unlikely to be preferred without strong project specific drivers**

Note: The score and related outcome are intended to provide only a very preliminary indication of timber bridge suitability. They should not be used as a final decision without project-specific assessment, cost review, and stakeholder input.

COMPARING HAZARD SPECTRA FROM 2004 AND 2025 VERSIONS OF THE AOTEAROA NEW ZEALAND SEISMIC LOADING STANDARD 1170.5

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ABSTRACT

A comparison of the response spectra recommended by the currently-cited seismic loading standard in Aotearoa New Zealand, NZS 1170.5:2004 and the newly released TS 1170.5:2025 is undertaken for different locations across Aotearoa New Zealand to identify what the NZS 1170.5:2004 Ultimate Limit State (ULS) demands correspond to in terms of the updated hazard and site classes specified in TS 1170.5:2025. Also, taking Te Whanganui-a-Tara Wellington Central Building District (CBD) as a case study, an equivalent Annual Probability of Exceedance (APoE) for TS 1170.5:2025, corresponding to NZS 1170.5:2004 ULS demand, is estimated considering the complex mapping between NZS 1170.5:2004 and TS 1170.5:2025 site classes, and distribution of the buildings of different heights (and periods) within each site class. The study finds that the equivalent TS 1170.5:2025 APoE corresponding to NZS 1170.5:2004 ULS Importance Level 2 (IL2) demand is approximately 1/275 when considering variation across the whole country, and somewhat higher (~1/200) when considering only the Te Whanganui-a-Tara Wellington CBD.

INTRODUCTION

A major revision of the National Seismic Hazard Model (NSHM) for Aotearoa New Zealand was released in 2022 (Gerstenberger et al., 2024). The NSHM-22 leverages advances in understanding and modelling methods, using the significant amount of data collected over the past two decades (Bora et al., 2024). The previous revision to NHSM was in 2010 (Stirling et al., 2012). Compared to the 2010 revision (NHSM-10), the NSHM-22 involves significant changes in both the modelling scheme and parameter choices, such as modelling of seismogenic fault sources in the seismicity rate model component, and the use of multiple ground-motion models in the ground-motion characterisation modelling component (Bora et al., 2024). The NSHM-10 did not result in an update to the seismic loading standard, NZS 1170.5; nevertheless, it has been used for site-specific studies. Based on the NSHM-22, on average, the calculated hazard has increased by approximately 50% compared to the NSHM-10 (GNS Science, n.d.).

The seismic loading Technical Specification (TS) 1170.5:2025 (Elwood et al., 2025; SNZ, 2025) was released in late 2025. The most significant update from NZS 1170.5:2004 in TS 1170.5:2025 was the definition of the elastic design spectra based on the updated hazard calculations of NSHM-22. While the existing loading standard NZS 1170.5:2004 (SNZ, 2004) remains the cited standard in the Building Code Verification Method (B1/VM1), the new TS 1170.5:2025 can be used (voluntarily) for the design of new buildings via an alternative solutions pathway. To assess existing buildings, the hazard based on the current standard NZS 1170.5:2004 (SNZ, 2004) continues to be used to define the benchmark loading – i.e., assessments are made relative to the demands specified in NZS 1170.5:2004.

Apart from the increase in calculated hazard based on TS 1170.5:2025, the method for defining site classes in TS 1170.5:2025 has also changed compared to

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the range of methods used in NZS 1170.5:2004. TS 1170.5:2025 follows the conventional approach adopted in international standards of defining the site classes solely based on the time-averaged shear wave velocity in the top 30 m of the ground profile, $Vs_{(30)}$. There is no one-to-one correlation between NZS 1170.5:2004 subsoil classes and TS 1170.5:2025 site classes. This change in site class definitions results in more than one TS 1170.5:2025 site class corresponding to each NZS 1170.5:2004 subsoil class. For brevity, NZS 1170.5:2004 and TS 1170.5:2025 are referred to as NZS and TS, respectively, from this point onwards.

This paper compares the difference in hazard based on NZS and TS to estimate the annual probability of exceedance (APoE) for the TS hazard, which is equivalent, on average, to the hazard from NZS 1170.5:2004 used for ULS design of IL2 buildings. This comparison is useful in a policy context as MBIE considers possible future changes in B1/VM1. Furthermore, for the assessment of existing buildings, where the benchmark hazard used to determine building ratings is based on NZS at 1/500, knowing the equivalent APoE for TS would allow a possible gradual transition to the use of new hazard data in TS without a significant shift in the benchmark hazard and hence no need to adjust building ratings.

A general methodology is presented below for finding an equivalent TS APoE corresponding to the Ultimate Limit State (ULS) demand per NZS, based on weights inferred from town population/number of buildings, the likely number of buildings in each site class, site class correlations, and building periods of interest. Using this methodology, an equivalent TS APoE is estimated for Aotearoa New Zealand as a whole and the Te Whanganui-a-Tara Wellington CBD.

METHODOLOGY

For the Importance Level 2 (IL2) category, which most buildings fall into, the ULS intensity corresponds to APoE = 1/500 for both NZS and TS. The NSHM-22 update estimates higher seismic hazard levels across much of the country than previous national seismic hazard models and the NZS. As an example, Figure 1 and Figure 2 show the ULS IL2 NZS response spectra for subsoil class D, along with the NSHM-22 spectra for $Vs_{(30)}$= 275 m/s, corresponding to different APoEs. $Vs_{(30)}$= 275 m/s is expected to be the most likely value corresponding to NZS subsoil class D, as discussed later in the paper. Notice the difference in the hazard for APoE=1/500 between Te Whanganui-a-Tara Wellington and Ōtautahi Christchurch. While the hazard based on 1/500 NSHM-22 is much greater than the NZS ULS design spectrum for Te Whanganui-a-Tara Wellington, the difference is small for Ōtautahi Christchurch. The change in hazard estimates across Aotearoa New Zealand (and across subsoil classes and spectral periods) is not uniform. TS elastic design spectra are based on the mean hazard estimates from NSHM-22, except in the low-seismicity region. This region is roughly north of Kirikiriroa Hamilton, where a floor is applied based on the 90th percentile hazard of Tāmaki Makaurau Auckland, as discussed by Hulsey et al. (Hulsey et al., 2025).

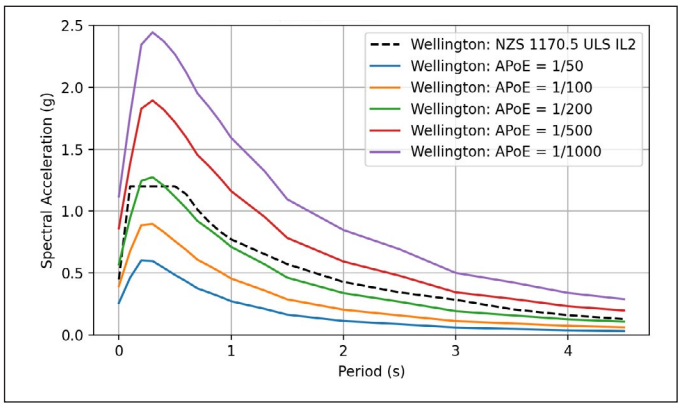


Figure 1: NZS and NSHM-22 spectra comparison for Te Whanganui-a-Tara Wellington, sub-soil class D with $V_{s(30)}$=275 m/s

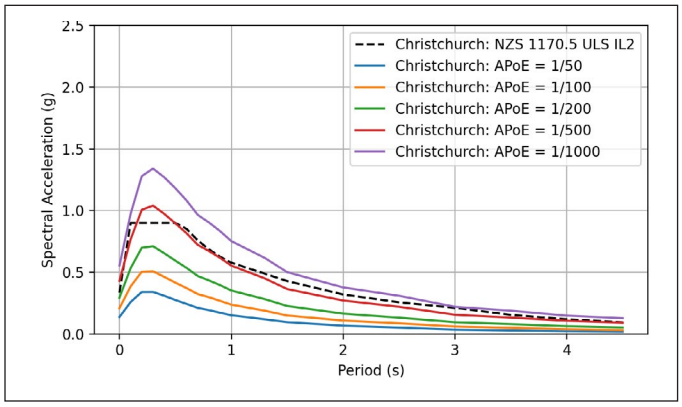


Figure 2: NZS and NSHM-22 spectra comparison for Ōtautahi Christchurch, sub-soil class D with $V_{s(30)}$=275 m/s

A general methodology is discussed to determine an equivalent (and average) APoE across the whole of Aotearoa New Zealand corresponding to the NZS hazard level. This general methodology (illustrated in a flowchart in Figure 3) is designed to allow consideration of the APoE across a desired range of subsoil conditions (site classes), periods of interest and/or building or population locations. Starting from the left, different towns (or population centres) may carry different weights, w_t . This may be based on the town's population, number of buildings, or the number of vulnerable buildings. Then, those buildings may be located on different NZS subsoil classes, n. The proportion of the number of buildings

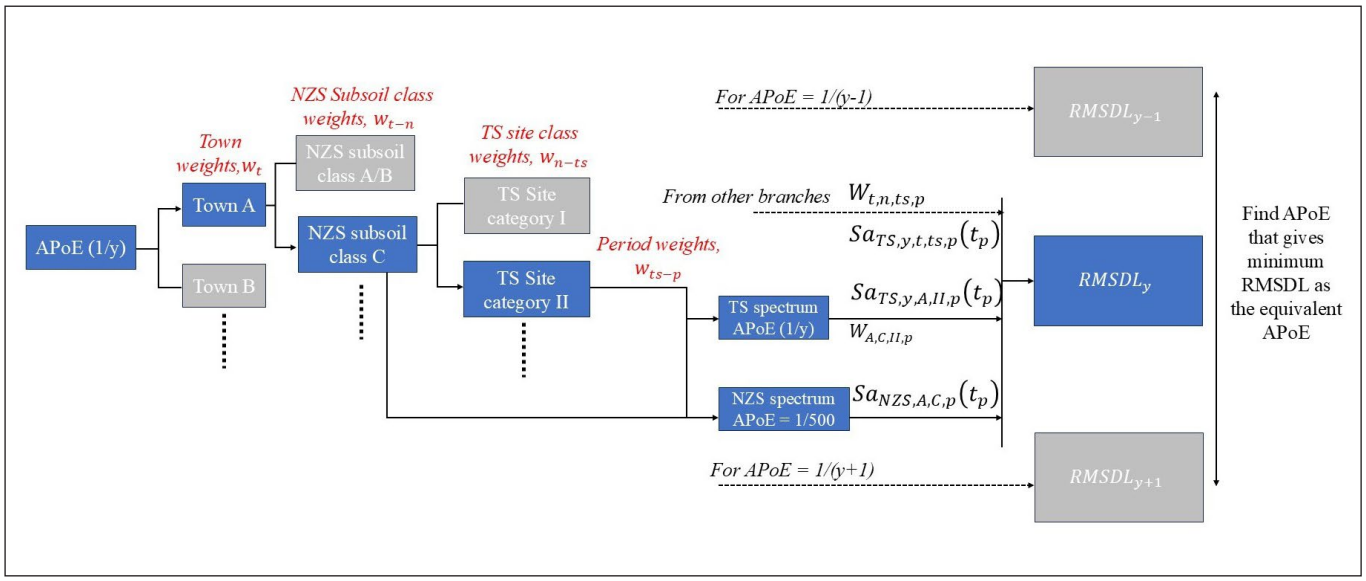


Figure 3: Flowchart showing a generic methodology for finding equivalent APoE

in each NZS subsoil class in a town determines the subsoil class weight, $w_{t\text{-}n}$. For buildings in each NZS subsoil class, the proportion of buildings in different TS site classes determines the TS site class weights, $w_{n\text{-}ts}$. Finally, the proportion of buildings in each NZS-TS site class combination over different periods, p, forms the period weights, $w_{T,S\text{-}p}$. Rather than assigning period weights based on the proportion of buildings, they may be further refined based on building occupancy, importance class, etc. However, for this study, the proportion of buildings in different storey bins has been used to estimate period weights, as explained later in the paper. Note that Figure 3 shows the comparison with TS spectra. A similar comparison can be made with NSHM-22 spectra, by using a correlation between NZS subsoil classes and $Vs_{(30)}$, rather than TS site classes shown in Figure 3.

To find an equivalent APoE, the weighted root mean square difference of the log (RMSDL) of spectral ordinates is obtained at different APoEs, as shown in equation 1. The APoE that gives the minimum RMSDL is taken as the equivalent APoE. Note that only one branch is shown in Figure 3 for illustration purposes, for a single town and a specific NZS-TS site class combination. For each town, there are four NZS subsoil classes, and each subsoil class has six TS site classes. In total, there are 24 branches for a single town. RMSDL is calculated over all branches emanating from all towns at a particular APoE. Each branch provides NZS and TS spectral ordinates, and a combined weight for each period of interest $V_{t,n,t,s,p}$, as shown in Figure 3.

$RMSDL_y$ is the RMSDL for TS APoE = $1/y$. Weights w_t, w_{t-n}, w_{n-ts} and w_{ts-p} are as defined earlier; $Sa_{NZS,t,n,p}\ (t_p)$ is the NZS ULS spectral ordinate at a period p for a specific town t and NZS subsoil class n. $Sa_{TS,y,t,ts,p}\ (t_p)$ is the TS spectral ordinate at a period p for a specific town t and a TS site class ts for APoE = 1/y.

$$RMSDL_y = \sqrt{\frac{1}{\sum W_{t,n,ts,p}} \sum W_{t,n,ts,p} \left[\ln(Sa_{NZS,t,n,p}(t_p)) - \ln(Sa_{TS,y,t,ts,p}(t_p))\right]^2} \quad (1)$$

$$W_{t,n,ts,p} = w_t \times w_{t-n} \times w_{n-ts} \times w_{ts-p} \quad (2)$$

$$RMSDL_y = \sqrt{\frac{1}{\sum W_{T,N,TS,p}} \sum W_{T,N,TS,p} \left[\ln(Sa_{NZS,T,N,p}(t_p)) - \ln(Sa_{TS,y,T,TS,p}(t_p))\right]^2} \quad (3)$$

$$W_{T,N,TS,p} = w_{TS-p} \quad (4)$$

Equivalent APoE for a specific town for a specific NZS-TS site class combination

To estimate an equivalent APoE for a particular town, the town weight, w_t for that town can be set to 1, and for the rest of the towns, set to zero. Similarly, an equivalent APoE for a specific NZS subsoil class can be estimated by setting w_{t-n} to 1 for that class, and the other NZS subsoil classes to zero. If the equivalent APoE for a specific NZS-TS site class combination is sought, then w_{t-n} and w_{n-ts} can each be set to 1 for the specific NZS-TS site class combination, with other classes set to zero. Equations 1 and 2 reduce to equations 3 and 4.

Subscripts T, N and TS denote a specific town, NZS subsoil class and TS site class, respectively. $w_{TS\text{-}p}$ are the period weights used to compare response spectra.

Equivalent APoE for the whole of Aotearoa New Zealand

The equivalent APoE for the whole of Aotearoa New Zealand is estimated based on the generic methodology discussed above. Weights are estimated as discussed next.

To estimate the equivalent APoE for the whole of Aotearoa New Zealand, the town (or region) weights are estimated based on the populations of 53 identified Functional Urban Areas (FUA) from the 2018 census (Stats NZ, 2021). They are estimated as the population of an FUA divided by the total population in all FUAs (excluding those locations where a floor is applied in TS). Thus, the sum of all town weights adds up to 1. Town weights are set to zero for FUAs with a floor applied in TS; that is, these towns are not included in the estimation of the equivalent APoE. Relative FUA populations are expected to have changed since the 2018 census but are unlikely to have changed to an extent that would significantly affect the outcome, especially given uncertainty in estimating other weights, as discussed next.

Weights w_{t-n} to w_{ts-p} are expected to differ across locations, given the different proportions of buildings in each NZS subsoil class, the correspondence between NZS subsoil classes and TS site classes, and the proportions of buildings for each period of interest. That requires extensive data for each location, most of which is not readily available. In the absence of such data, some simplifying assumptions may be made regarding the weights.

Typically, the distribution of buildings in NZS subsoil classes is expected to differ across FUA. This proportion may be obtained from the building inventory and regional or national site class maps (e.g., Perrin et al., 2015). For simplicity, the distribution may be assumed uniform (i.e., equal weights w_{t-n}=0.25 are assigned to each NZS subsoil class A/B, C, D, and E) in the absence of specific data. This estimate is crude, but the equivalent APoE estimate is found to be relatively insensitive to this assumption, as discussed later in the paper.

As noted previously, there is no direct correlation between NZS subsoil classes, $V_{s(30)}$, or TS site classes. There can be a range of $V_{s(30)}$ likely for a site classified into a particular NZS subsoil class, as demonstrated for the locations of seismic monitoring stations across Aotearoa New Zealand (Wotherspoon et al., 2024), and also for the Te Whanganui-a-Tara Wellington urban area (Kaiser et al., 2024). However, for the purposes of this analysis, a most likely correlation may be assumed between the NZS subsoil class and the TS site class, based on the distributions reported in Wotherspoon et al. (2024). Table 1 shows the most likely TS site class, and a representative $V_{s(30)}$ value for each NZS subsoil class considered for finding equivalent APoE for all towns for this study.

NZS subsoil class B is characterised as a rock site (defined by unconfined compressive strength (UCS) >1 MPa and $V_{s(30)}$ > 360 m/s), which may have a surface layer of up to 3 m of soil or highly or completely weathered rock. TS class I sites are also defined as rock sites; however, with $V_{s(30)}$ > 750 m/s. For the purpose of this study, the most likely TS site class is identified as TS II, which is characterised as very stiff soil, or very dense soil, or soft rock. This is consistent with the median $V_{s(30)}$ estimate for Class B of Wotherspoon et al. (Wotherspoon et al., 2024) of 675m/s, albeit based on limited rock $V_{s(30)}$ information. It is recognised that

Table 1: Mapping of NZS subsoil classes to the most likely TS site classes and $V_{(S(30)}$

NZS subsoil class	TS site class	$V_{S(30)}$ (m/s)
B	II	675
C	III	375
D	IV	275
E	VI	150

many subsoil class B sites can be classified as TS I. For NZS subsoil classes C and D, a representative $V_{s(30)}$, and most likely TS site class are assumed based on the median $V_{s(30)}$ values reported for the respective subsoil classes (Wotherspoon et al., 2024). There can be large variations in $V_{s(30)}$ of class E sites. For site class E, the representative $V_{s(30)}$ is assumed to be at the boundary of TS VI and TS VII, with TS VI as the most likely site class. A more refined study may consider the TS-NZS site class correlation region-specific, varying throughout Aotearoa New Zealand. However, for this national study, the above assumptions are considered reasonable.

Another assumption concerns the periods used to compare spectra. Rather than taking a building-by-building approach, a limited number of representative periods of interest may be assumed to represent all buildings in a town, with assumed proportions. Considering that the bulk of the buildings are up to 3 storeys, significantly fewer 4-6 storeys, and only a very small number of buildings more than six storeys, it was decided to consider only two periods of interest for two and five-storey buildings, respectively, representative of the first two categories of buildings.

There can be considerable variation in fundamental periods of buildings of the same height, depending upon their layout, type of lateral system, etc. Nevertheless, for this study, the elastic fundamental periods of two and five-story buildings are assumed as 0.2s and 0.5s, respectively, based on a simple rule-of-thumb relationship of 0.1 times the number of stories, N, which is also prescribed for the estimation of fundamental periods of moment frames by the American standard ASCE 7 (ASCE, 2022). However, there have been anecdotal references suggesting that buildings in Aotearoa New Zealand have longer periods than the assumed relationship of 0.1N would imply. A sensitivity analysis is conducted by using periods twice those estimated by this relationship, both for estimating equivalent APoE across Aotearoa New Zealand and for the Te Whanganui-a-Tara Wellington CBD case study presented later in the paper. The equivalent APoE estimates are found not to be significantly different for the two assumed relationships between periods and number of stories, as shown later in the paper. Period weights are assumed to be 0.8 and 0.2, respectively, for all TS site classes in all towns.

Equivalent APoE for a specific town with different NZS-TS site class combinations

For an equivalent TS APoE for a specific town, w_t is set to one for that town. Other weights w_{t-n} , w_{n-ts} and w_{ts-p} should be estimated as discussed earlier in the generic methodology and is demonstrated later for Te Whanganui-a-Tara Wellington CBD.

Equivalent APoE estimates for a specific FUA and a specific NZS-TS site class combination

Figure 4 shows the equivalent APoE obtained for 53 FUAs across Aotearoa New Zealand for four NZS subsoil classes. Note that a few locations north of Kirikiriroa Hamilton have been greyed out where a floor has been applied in TS. For most locations, the equivalent APoE corresponding to NZS ULS IL2 is greater than 1/500 (i.e. shorter return period), reflecting the general increase in calculated hazard from the updated NSHM-22. Locations with the largest equivalent APoE tend to be in the lower North Island (e.g., Masterton and Te Whanganui-a-Tara Wellington), or south-eastern South Island (e.g., Ōtepoti Dunedin and Invercargill).

Figure 5 shows the TS spectrum matched for Te Whanganui-a-Tara Wellington NZS subsoil class C, for NZS ULS IL2 (APoE = 1/500). Also shown is the mean NSHM-22 spectrum. The corresponding equivalent APoEs are 1/173 and 1/155 for TS and NSHM-22, respectively. Note that the equivalent APoE depends upon the periods of interest and respective weights. For the results shown in Figure 4 and Figure 5, a particular NZS subsoil class for each FUA is matched separately, considering only the 'most likely' NZS-TS site class correlation per Table 1, and the two periods of interest (0.2 and 0.5 seconds), as detailed earlier.

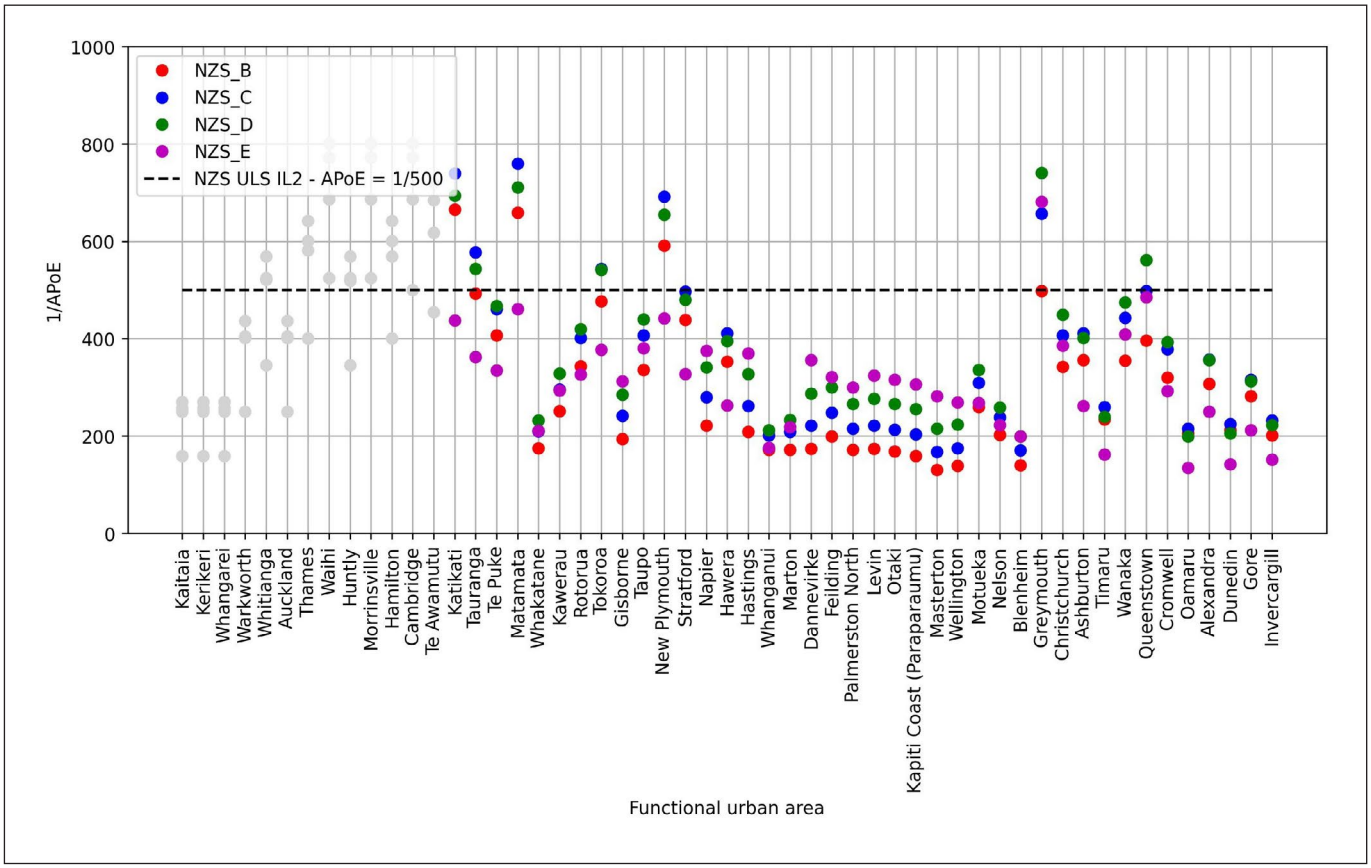


Figure 4: Equivalent APoE for matched NZS (ULS IL2) and TS spectra for different FUAs and NZS subsoil classes (Grey dots indicate the locations where a floor has been applied in TS)

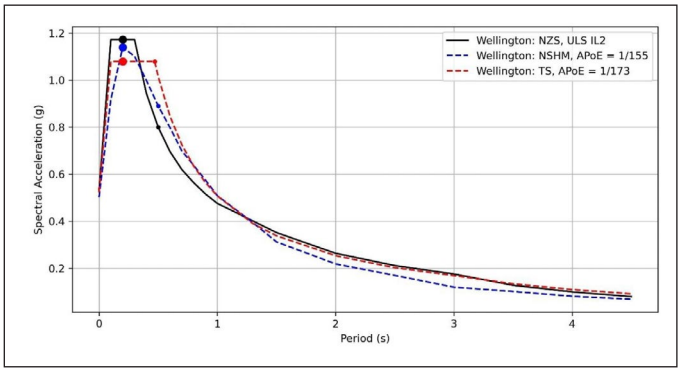


Figure 5: Matched NZS (ULS IL2), TS, and NSHM-22 spectra and equivalent APoEs for Te Whanganui-a-Tara Wellington, NZS subsoil class C. The dots mark the periods (0.2 s and 0.5s), and their relative sizes indicate the period weights (0.8 and 0.2) used to match the spectra.

Although matching is done assuming the most likely correlation from Table 1, as noted earlier, there can be considerable variation in $V_{S(30)}$ values for a given NZS subsoil class. Figure 6 shows the variation in equivalent APoE estimated for a few locations for NZS subsoil class C, using a range of $V_{S(30)}$ values from 200 m/s to 600 m/s reported for NZS subsoil class C (Wotherspoon et al., 2024) and using NHSM-22 spectra for the specific $V_{S(30)}$ values .

Note the variation of equivalent APoE for the range of NZS subsoil class C and $V_{S(30)}$ correlation, indicated by the extent of blue lines in Figure 6, also differs by location. This difference can be explained by examining the hazard curves for varying $V_{S(30)}$ values. Figure 7a and Figure 7b show mean hazard curves for Te Whanganui-a-Tara Wellington and Ōtepoti Dunedin for a period of 0.2s, which carries the maximum period weight used in calculations. Also shown in Figure 7a and Figure 7b as vertical dashed lines are the NZS ULS IL2 intensities for 0.2s, for the respective locations. Figure 7c and Figure 7d show close-ups of the same hazard curves, but with the X-axis normalised by the NZS ULS IL2 intensity and the Y-axis normalised by the equivalent APoE for the most likely $V_{S(30)}$. Notice a larger spread along the vertical axis for Ōtepoti Dunedin, as compared to Te Whanganui-a-Tara Wellington, which contributes towards a larger difference in equivalent APoE for different $V_{S(30)}$ values, as compared to Te Whanganui-a-Tara Wellington.

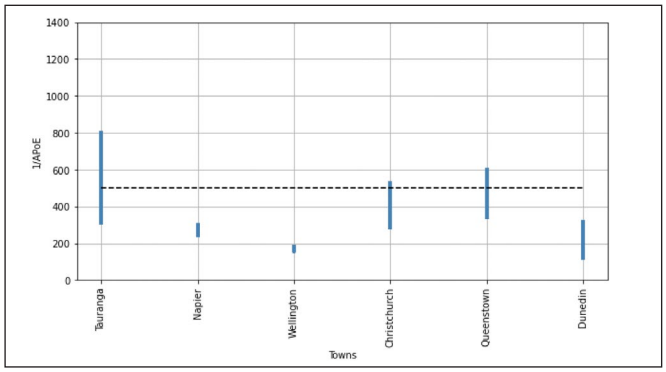
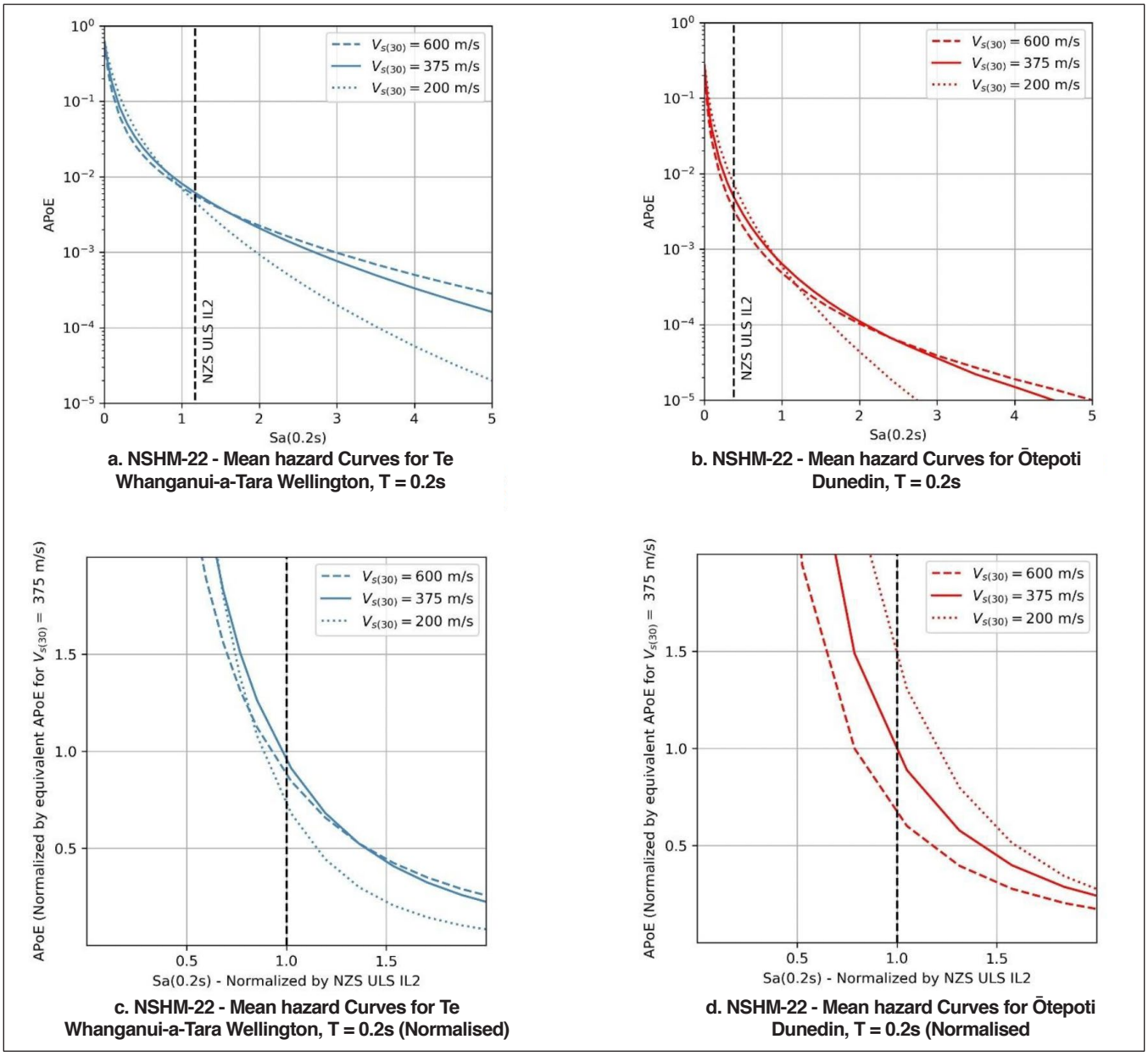


Figure 6: Variation in equivalent APoE for NZS subsoil class C for a $V_{S(30)}$ range of 200m/s to 600 m/s, estimated using mean NHSM-22 spectra for the specific $V_{S(30)}$ values

Equivalent apoe estimate for the whole of Aotearoa New Zealand

Figure 8 shows the equivalent APoE. Using this methodology, the equivalent TS APoE is found to be 1/275[1]. Generally, for the lower North Island, near Te Whanganui-a-Tara Wellington, and south-east of South Island, near Ōtepoti Dunedin, the individual equivalent APoEs for FUAs are observed to be higher than 1/275. In terms of hazard, this would mean these regions have TS APoE= 1/275 hazard higher than NZS APoE = 1/500 hazard. This is also shown in Figure 9, which shows



a. NSHM-22 - Mean hazard Curves for Te Whanganui-a-Tara Wellington, T = 0.2s

b. NSHM-22 - Mean hazard Curves for Ōtepoti Dunedin, T = 0.2s

c. NSHM-22 - Mean hazard Curves for Te Whanganui-a-Tara Wellington, T = 0.2s (Normalised)

d. NSHM-22 - Mean hazard Curves for Ōtepoti Dunedin, T = 0.2s (Normalised

Figure 7: Variation in TS hazard APoEs corresponding to NZS ULS IL2 for Te Whanganui-a-Tara Wellington and Ōtepoti Dunedin

[1]*Estimates of equivalent APoE are found to be relatively insensitive to different estimates of w_{t-n}. Omitting site class E and*
- *Considering the remaining three subsoil classes with equal weights gives equivalent APoE as 1/273.*
- *Considering subsoil classes C and D with twice the weight of subsoil class B gives equivalent APoE as 1/283.*

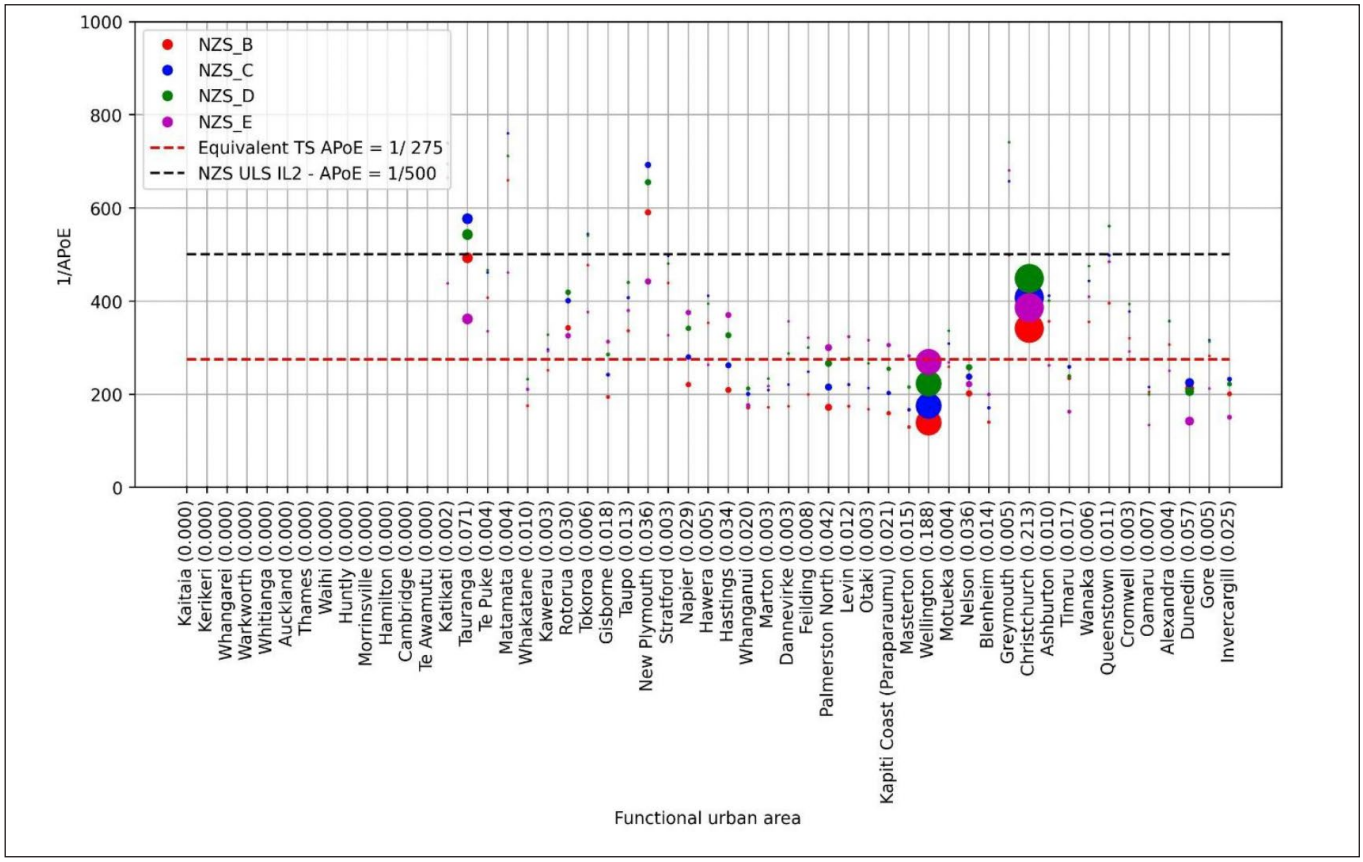


Figure 8: Equivalent APoE considering different FUAs. Values inside the brackets alongside the FUAs are weights w_t, which are based on the population estimates from the 2018 population census. Marker sizes for each FUA indicate the relative difference in w_t. Note that the weights for the locations where floor is applied in TS are taken as zero, that is, they are excluded from the analysis of equivalent APoE.

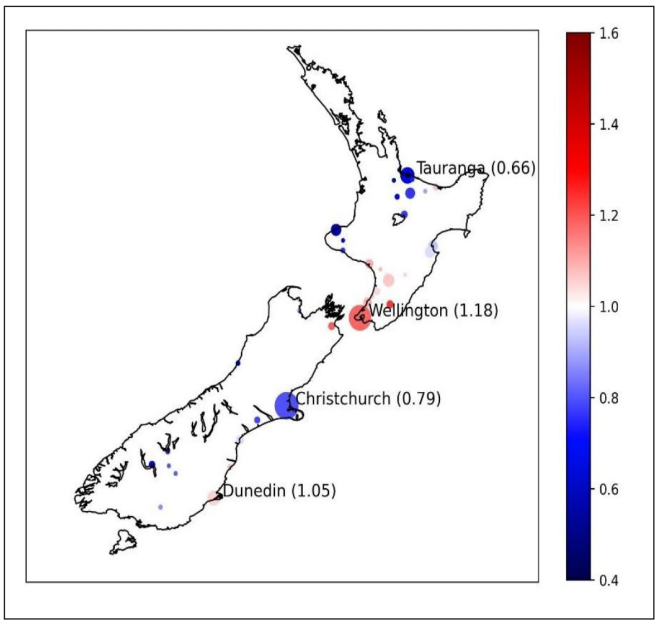


Figure 9: Ratio of short-period spectral accelerations, Sas, from TS APoE = 1/275 to NZS APoE = 1/500. The colour of each marker indicates how high or low TS APoE = 1/275 hazard is with respect to the NZ APoE=1/500 hazard, with red markers indicating higher TS hazard than NZS hazard. The size of each marker indicates town weight relatively. Values inside brackets against the names of the FUAs are Sas ratios for these locations.

the ratio of short-period spectral acceleration, Sas, at equivalent TS APoE = 1/275 to Sas at NZS APoE = 1/500 for FUAs considered in this study.

As noted earlier, due to a range of possible $V_{S(30)}$ values for a specific NZS subsoil class, there can be a variation in NZS-TS site class correlation. The TS site class can be higher or lower than the most likely class considered in Table 1, based on $V_{S(30)}$ for a specific site. Considering this possibility, a sensitivity test is conducted with two additional variations. Estimates of equivalent APoEs are made, considering the TS site class above and below the 'most likely', as shown in Table 2.

Note that for site class E, Variation 2, the TS site class remains the same, TS VI. A class down would have resulted in TS VII. TS spectra are available only up to TS VI. For TS VII, site-specific studies are required to develop response spectra, with TS VI spectral ordinates acting as the minimum. Hence, for this study, TS VI is taken as the most likely TS site class for Variation 2 as well. The resulting equivalent APoEs estimated for the two variations are also listed in Table 2.

Table 2: Variations considered for NZS-TS site class correlation and equivalent APoEs (NZS ULS IL2)

NZS subsoil class	Most likely TS site class	Variation 1	Variation 2
B	II	I	III
C	III	II	IV
D	IV	III	V
E	VI	V	VI
Equivalent TS APoE*	1/275 (1/249)	1/331 (1/343)	1/245 (1/215)

*Values inside brackets are the estimated equivalent APoEs for building periods assumed in seconds to be equal to 0.2 times the number of stories

Table 3: Equivalent APoE considering different NSHM-22 hazard percentiles (NZS ULS IL2)

10th percentile	Mean	90th percentile
1/934	1/288	1/150

Note that no change has been made in weights w_{t-n}. Also, full correspondence is assumed between one NZS subsoil class and a single TS site class. In a targeted regional study, these weights should be based on the proportion of buildings in each NZS subsoil class and TS site class, as shown in the next section for Te Whanganui-a-Tara Wellington CBD.

NSHM-22 estimated the epistemic variability in seismic hazard parameters (Gerstenberger et al., 2024). Table 3 lists the equivalent APoE across all FUAs obtained by considering the mean, 10th, and 90th percentile hazard based on NSHM-22. NZS subsoil class to$V_{S(30)}$ correspondence is assumed per Table 1. The wide range of equivalent APoE illustrates the significant uncertainty in seismic hazard estimates.

CASE STUDY FOR TE WHANGANUI-A-TARA WELLINGTON CBD

In this section, an equivalent APoE is obtained for Te Whanganui-a-Tara Wellington CBD, considering the actual distribution of buildings in each NZS and TS site class and periods of interest, taken from the RiskScape building database (Paulik et al., 2023). Central Te Whanganui-a-Tara Wellington is one of the few locations in Aotearoa New Zealand where extensive building databases have been compiled (e.g., Puranam et al., 2019), and both local NZS class and $V_{S(30)}$ mapping has been undertaken (see Kaiser et al., 2024). This allows a more detailed case study of equivalent APoE that considers local conditions, e.g. types of structures present and aspects of the local geology. From the database, a total of 1,742 buildings are considered in the Te Whanganui-a-Tara Wellington CBD, excluding light-frame timber buildings up to three stories high.

Site Class determination in Te Whanganui-a-Tara Wellington CBD

For each building, NZS and TS site classes are obtained based on the location of the building and the NZS subsoil class, and $V_{S(30)}$ mapping undertaken by Kaiser et al. (Kaiser et al., 2019, 2024) shown in Figure 10. The NZS Subsoil Class map is underpinned by extensive geophysical measurements and shallow borehole data. The CentrePort region was classified by Kaiser et al. (Kaiser et al., 2019, 2024) as subsoil class D, possible E, and the authors note that while available geotechnical measurements are indicative of Class D, there is softer hydraulic fill and historical observations of liquefaction in this area (e.g., Cubrinovski et al., 2017). This area is treated as Class D in this study.

The $V_{S(30)}$ map (Figure 10) comprises a more limited set of quality-controlled $V_{S(30)}$ measurements overlain on a background $V_{S(30)}$ model calculated from 3D representations of the subsurface lithology (Hill et al., 2022, 2023). Here, the modelled $V_{S(30)}$ values are used to estimate the TS site class. Note that $V_{S(30)}$ measurements on rock are sparse for Te Whanganui-a-Tara Wellington, as they are for all of Aotearoa New Zealand; hence the $V_{S(30)}$ map makes a generic assumption of V_(S(30)) > 750 m/s for sites mapped as outcropping greywacke rock, consistent with the commonly adopted value of $V_{S(30)}$ = 760 m/s for rock used in the international seismological community (Dobry et al., 2000). The authors note this assumption is likely an overestimate since greywacke is highly weathered near the surface and/or thin soil layers are present (as is allowable under the NZS subsoil Class B definition).

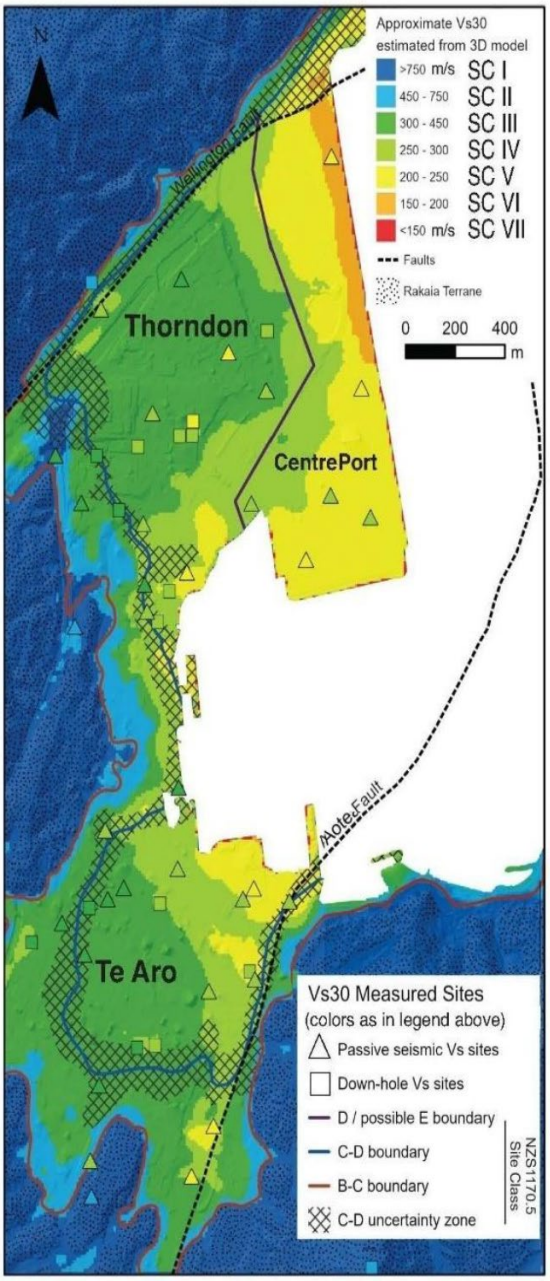


Figure 10: NZS subsoil class and $V_{S(30)}$ map for Te Whanganui-a-Tara Wellington CBD. The map shows the modelled $V_{S(30)}$ values and individual $V_{S(30)}$ measurements. As noted by the authors, the $V_{S(30)}$ model and measurements are generally in good agreement, with the exception of an area of Thorndon where the model appears to overestimate $V_{S(30)}$ values. Sites in this area of Thorndon would be classed as TS site class III based on the model, when measurements suggest TS site class IV (or in a few cases TS site class V) may be more appropriate. (Adapted from Kaiser et al., 2024)

(Note that this map is not to be used for determining site classes for design based on NZS or TS. Further, note that for TS, if $V_{S(30)}$ is not measured, an uncertainty band of 30% on $V_{S(30)}$ is applicable per Section 3.1.3.6 of TS.)

Calculation of model weights

The relative fractions of total buildings in each NZS subsoil class, or w_{t-n} are calculated from the NZS subsoil class map based on the buildings' locations. The fraction of total buildings in each NZS-TS site class combination (i.e. w_{n-ts}) is also calculated, based on the $V_{S(30)}$ model and TS site class $V_{S(30)}$ ranges (Lee et al., 2025). For a particular NZS subsoil class, the fraction of buildings in each TS site class of the buildings in that subsoil class provides weight w_{n-ts}.

Note that based on the assumptions described above in the $V_{S(30)}$ mapping, most buildings in NZS subsoil class B would correspond to TS site class I, rather than TS site class II, which is assumed as the most likely in the national analysis (Table 1). However, given the poor constraints on rock $V_{S(30)}$ and its likely overestimation (Kaiser et al., 2024), the assumption NZS subsoil class B = TS site class II in Te Whanganui-a-Tara Wellington is adopted for consistency with the national approach. This also allows for the presence of thin soil layers (<3m) and/or highly weathered near-surface material.

Also, based on the above approach, a high proportion of buildings in subsoil class D in Te Whanganui-a-Tara Wellington correspond to TS site class III. While overall for Te Whanganui-a-Tara Wellington, NZS subsoil class D areas correlate with a median site class of TS site class IV, compatible with Table 2 (see Figure 6 in Kaiser et al. (Kaiser et al., 2024)), many building locations fall within areas modelled as $V_{S(30)}$ > 300 m/s, in the lower end of the range prescribed for the TS site class III. This likely partly reflects a genuine tendency for buildings to be located on somewhat stiffer soils in Te Whanganui-a-Tara Wellington. However, as noted in Kaiser et al. (Kaiser et al., 2024), there is a tendency of the $V_{S(30)}$ model to overestimate $V_{S(30)}$ observed in parts of Thorndon (see also Figure 10), which may also introduce a degree of bias in this area.

Amongst the buildings in each NZS-TS combination, the buildings are categorised by height into five bins, as shown in Figure 11 for NZS subsoil class C. The Figure shows the distribution of the number of buildings by number of stories for different NZS subsoil Class C-TS site class combinations. A representative period is assigned for each bin, and the fraction of buildings in each bin is the period weight $w_{ts\text{-}p}$ for that specific NZS-TS site class combination, as shown earlier in Figure 3.

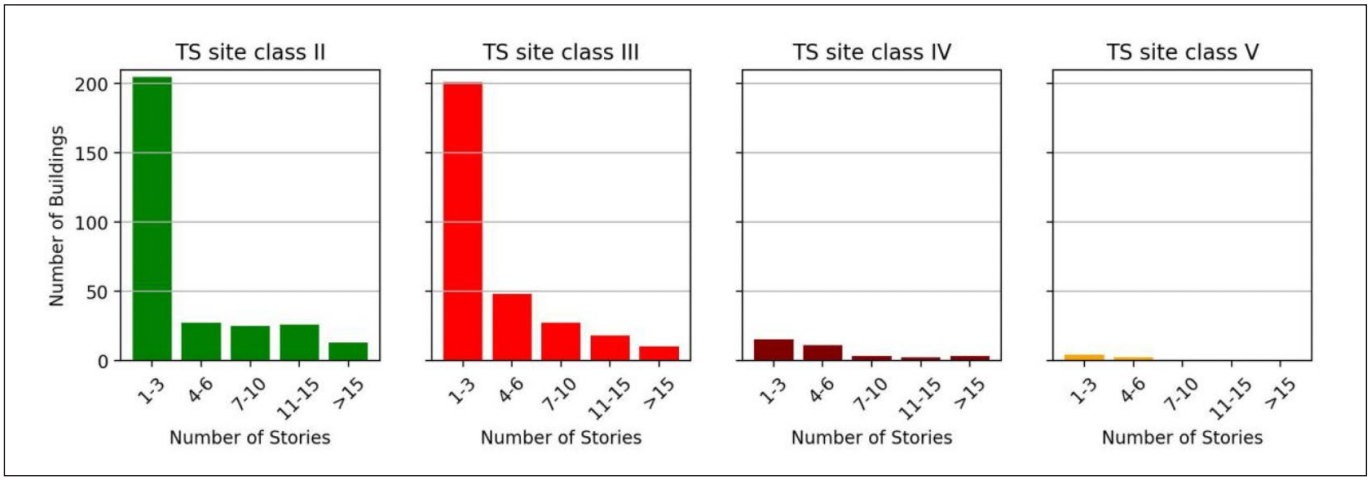


Figure 11: Number of buildings in different storey bins in Te Whanganui-a-Tara Wellington CBD, NZS subsoil class C

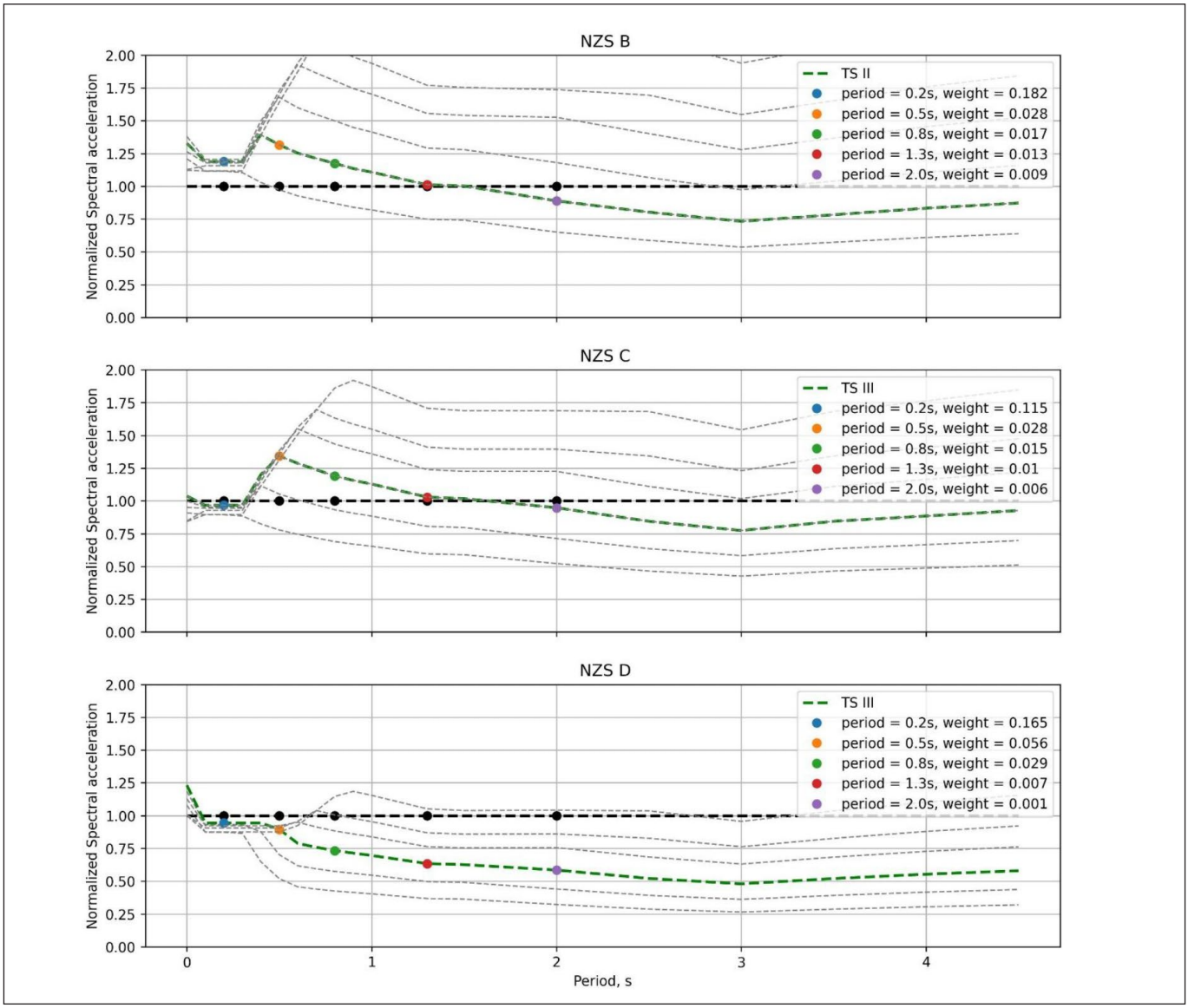


Figure 12: TS spectra normalised by 100% NZS ULS IL2 spectral ordinates at the equivalent APoE = 1/189 for Te Whanganui-a-Tara Wellington CBD. There are three subplots for the NZS subsoil classes. Each subplot shows six TS spectra (Grey and Green dashed lines) for each TS site class with five periods each, at which the comparison is made. In total, there are ninety weighted comparison points that are used to estimate equivalent APoE. Normalised spectra shown in green are the TS spectra with the highest $w_{n\text{-}ts}$ for a particular NZS subsoil class."

Te Whanganui-a-Tara Wellington CBD Equivalent APoE

Following the above methodology for Te Whanganui-a-Tara Wellington CBD, the equivalent TS APoE is 1/189 for NZS ULS IL2. Given the uncertainty in the site classes discussed above, the following sensitivity analyses are also considered: (1) TS site classes based on the $V_{s(30)}$ model assumptions for rock, resulting in a large number of TS site class I sites, and (2) include only buildings in NZS subsoil classes C and D. The equivalent APoE corresponding to NZS ULS IL2 obtained for the above two variations (1/200 and 1/212, respectively) are not appreciably different from the equivalent APoE for the base case (1/189). Considering variation in the building period estimates as well, the range of equivalent APoE is between 1/184 and 1/218.

Figure 12 shows, for B to D subsoil classes, the TS spectra at equivalent APoE for Te Whanganui-a-Tara Wellington CBD (1/189) normalised by NZS ULS IL2 spectral ordinates. These plots compare normalised NZS spectra for different subsoil classes with the TS spectra corresponding to the equivalent APoE. The figure shows that the difference in NZS and TS spectral acceleration can be quite large for some NZS – TS site combinations at certain periods, but such combinations carry far less weights than the combinations marked by dots on the normalised TS spectra with the highest $w_{n\text{-}ts}$ for a particular NZS subsoil class, shown by green dashed lines. Weights of all spectral NZS-TS ordinate pairs are considered in the estimation of equivalent APoE, but the spectral ordinates at lower periods of interest carry larger weights as compared to other periods due to the larger number of short-period buildings. This is reflected in the spectra matching shown in Figure 12.

Table 4 also shows the equivalent APoE for NZS ULS IL4 as 1/571. Note that the same weights have been used to estimate APoEs corresponding to both IL2 and IL4. Further refinement could be made in the future to estimate weights based on the proportion of buildings corresponding to that specific importance level.

Table 4: Equivalent APoE at different NZS intensities for Te Whanganui-a-Tara Wellington CBD

Intensity	NZS APoE	Equivalent TS APoE
NZS ULS IL2	1/500	1/189
NZS ULS IL4	1/2500	1/576

CONCLUSION

Comparison of NZS 1170.5:2004 and TS 1170.5:2025 spectra is made to find an APoE based on TS spectra, which, on average, matches the NZS ULS IL2 spectrum. An approach is discussed for carrying out spectral matching at periods of interest, based on the proportions of buildings with different periods located in different NZS and TS site class combinations across different towns and cities in Aotearoa New Zealand. Based on the assumption of equal likelihood of each NZS subsoil class at each location, assigning weights to different towns/cities based on resident population estimates from the 2023 census, and building fundamental period assumptions, the equivalent APoE is found to vary between 1/215 and 1/343. The most likely value is around 1/249 to 1/275 based on the most likely NZS-TS site class correlation and the two building period assumptions.

Taking Te Whanganui-a-Tara Wellington CBD as a case study, the proportion of buildings in different NZS-TS site class combinations indicates that the equivalent APoE for Te Whanganui-a-Tara Wellington is slightly higher than that for the country as a whole, ranging from 1/184 to 1/218 depending on NZS-TS site class alignment and building period assumptions. Sensitivity studies, including variation in the NZS-TS site class correlation, indicate a wide variation in the equivalent APoE.

The results of this study can inform policy discussions regarding possible future transitions from the design and assessment of buildings based on NZS to those based on TS. But engineers are reminded that current building regulations, for new and existing buildings, remain based on NZS 1170.5:2004. TS 1170.5:2025 can be used for new design on a voluntary basis via an alternative solutions pathway.

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A SELF-CENTRING LOW-DAMAGE ANCHORAGE SYSTEM FOR CYLINDRICAL STEEL STORAGE TANKS USING RESILIENT SLIP FRICTION DAMPERS

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ABSTRACT

Post-event surveys following the 2013 Seddon and Lake Grassmere and 2016 Kaikōura earthquakes have identified mechanically anchored cylindrical steel storage tanks as one of the more vulnerable industrial structures in Aotearoa New Zealand, with anchorage rupture, elephant-foot and diamond buckling, and full tank collapse reported across the wine and dairy sectors. Industry-wide retrofit of hold-downs between the two events reduced the share of anchor-related damage but coincided with a shift in the predominant failure mode to the tank barrel and cone, indicating that the intended capacity hierarchy of the system had not been preserved. This paper examines a tension-only anchorage system based on the Resilient Slip Friction Damper (RSFD). The RSFD provides energy dissipation through friction between an inner shaft and an outer cylinder clamped together by pre-tensioned bolts, and restoring force through pre-stressed disc springs, producing a stable flag-shaped hysteresis with no material yielding. Prototype devices were manufactured and subjected to component-level cyclic testing to validate the predicted device performance. The system was then compared against necked-rod and buckling-restrained anchorages through nonlinear time-history analyses of three cylindrical tanks spanning a range of aspect ratios representative of industrial practice. The tension-only system produced an average reduction in overturning-moment demand of the order of 30–40% relative to the other conventional anchorage systems analysed, with a more pronounced benefit in tanks of higher aspect ratio. Field implementations across the Aotearoa New Zealand wine and dairy sectors are also reported.

Keywords: storage tanks; seismic anchorage; self-centring; friction damper; RSFD; low-damage design; tension-only hold-down.

INTRODUCTION

Above-ground cylindrical steel liquid storage tanks are widely used across the petrochemical, winery, dairy, food and beverage, and water utility sectors. The extent of damage observed in recent earthquakes has exceeded that anticipated under conventional design assumptions for this class of structure. Post-event surveys in Aotearoa New Zealand and overseas have documented widespread anchorage rupture, barrel buckling, and operational disruption (Dizhur et al., 2017; Yazdanian et al., 2020; Fischer et al., 2016).

The seismic response of liquid storage tanks has been studied for over six decades. Housner (1957) introduced the now-standard decomposition of the contained liquid into an impulsive component, which moves with the rigid tank shell, and a convective component representing the long-period sloshing mass; this two-mass model remains the conceptual basis of all current design codes. Veletsos and Yang (1977) and Haroun and Housner (1981) extended the formulation to include the flexibility of the tank wall, showing that the impulsive pressure distribution and the natural period of the impulsive mode are influenced by shell deformation, and providing the modal analysis framework adopted in subsequent codified procedures. Malhotra et al. (2000) developed a simplified spring–mass procedure that captures the essential dynamic behaviour with a small number of parameters and is suitable for routine design; API 650 (2020) takes its impulsive-period procedure from this work (Cl EC.4.5.1). Current design provisions include Appendix E of API 650 (2020), Part 4 of Eurocode 8, and the NZSEE Seismic

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Design of Storage Tanks guideline (NZSEE, 2009), which itself updates the original Red Book recommendations of Priestley et al. (1986). NZSEE 2009 differentiates anchorage systems by their ductility and energy-damping capability, permitting reduced design forces for ductile rod-based hold-downs relative to standard rods.

In practice, ductility in rod-based anchorages is realised through material yielding, and field observations have shown that this yielding is frequently accompanied by collateral damage. The Marlborough region produces over 75% of Aotearoa New Zealand wine output, with wine exports valued at approximately NZD 1.6 billion at the time of the 2016 Kaikōura earthquake (Dizhur et al., 2017). Surveys following the 2013 Seddon (Mw 6.6) and Lake Grassmere (Mw 6.6) earthquakes covered more than 20 wineries, with around 4,000 stainless steel tanks inspected in total; approximately half of these tanks exhibited visible deformation, and over 70 tanks were reported to have ruptured. Following the 2016 Kaikōura earthquake (Mw 7.8), in the order of 1,000 tanks were damaged across the region, of which around 150 were considered irreparable, and an estimated 10–13% of pre-event tank capacity (approximately 30–40 million litres) was sidelined for the 2017 vintage.

Yazdanian et al. (2020) compiled damage statistics across the 2013 and 2016 events that are particularly instructive. In 2013, anchor damage was the most prevalent failure mode at 47%; barrel and cone damage were observed at 28% and 14% respectively. By 2016, anchor damage had dropped to 23% — reflecting industry-wide retrofit of hold-downs after 2013 — but barrel and cone damage had escalated to 54% and 43%. The intended capacity hierarchy of the system had not been preserved; stronger anchorages had simply migrated the damage up the tank wall (Figure 1).

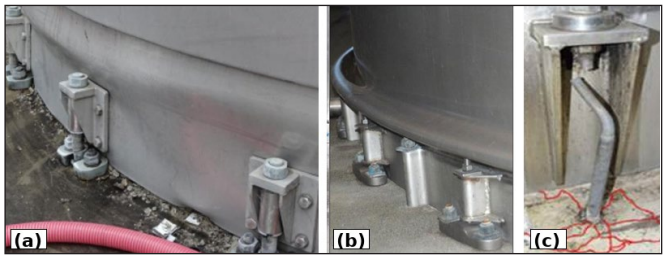


Figure 1: Tank damage observed following the 2016 Kaikōura earthquake: (a) barrel deformation at the gripping zone; (b) elephant-foot type buckling adjacent to anchor brackets; (c) ruptured anchor rod (after Dizhur et al., 2017).

These outcomes reflect the limited ductility and absence of true capacity protection in conventional rod-based anchorages. Necked rods derive their ductility from low-cycle plastic elongation but buckle in the reverse cycle, producing pinched hysteresis, accumulating residual displacement, and ultimately rupturing in the most severe events (Sahami et al., 2020). Damage to the sacrificial element is the intended energy-dissipation mechanism, which means inspection and replacement are required after a design-level event, and the residual capacity to resist aftershocks is reduced. The Kaikōura earthquake was followed by an extended aftershock sequence (Dizhur et al., 2017), and the 2019 Ridgecrest sequence in California, in which a Mw 7.1 mainshock followed a Mw 6.4 foreshock by 34 hours, illustrates the same exposure internationally.

This paper proposes a different approach: a tension-only, low-damage, self-centring anchorage system using the Resilient Slip Friction Damper (RSFD) developed at Auckland University of Technology (Mohammadi Darani et al., 2021). The RSFD provides controlled energy dissipation through bolt-clamping friction and restoring force through pre-stressed disc springs, producing a stable flag-shaped hysteresis with no material yielding. The objectives of this paper are: (i) to set out a design methodology for RSFD-anchored cylindrical tanks consistent with NZSEE 2009 and API 650 frameworks; (ii) to compare RSFD performance against necked-rod and buckling-restrained anchorages through nonlinear time-history analysis of three case-study tanks of varying aspect ratio; and (iii) to summarise outcomes from four Aotearoa New Zealand implementations spanning the wine and dairy sectors.

PERFORMANCE OF CONVENTIONAL ANCHORAGE SYSTEMS

Necked-rod anchorages

In a necked-rod hold-down, a length of rod is reduced in diameter to localise yielding away from the threaded ends. Tensile yielding of the necked region provides the connection with ductility under uplift demand. Under reversed cyclic loading the necked region is subjected to compression, where buckling can occur, and the response exhibits pinched hysteresis with progressive stiffness and strength degradation (Sahami et al., 2020). Plastic strain accumulates with successive cycles, contributing to displacement demand and, under prolonged inelastic excursions, low-cycle fatigue rupture. Because the energy-dissipation mechanism relies on material yielding, the rods are inspected after a design-level event and replaced where significant inelastic deformation has occurred.

Buckling-restrained anchorages

Buckling-restrained rods enclose the yielding core in a sleeve that prevents flexural buckling under compression, similar to a buckling-restrained brace (Fahnestock et al., 2006). The system therefore retains stiffness and capacity in both tension and compression, producing a more stable, symmetric hysteresis loop with greater enclosed area than that of the necked-rod system, and

correspondingly higher per-cycle energy dissipation. One aspect of the response that may warrant further investigation is the reversed action: in theory, the compressive force could be transferred back into the tank as a concentrated demand at the gripping point.

A review of the damage statistics from the 2013 and 2016 Aotearoa New Zealand earthquakes points to a possible related effect at the population level. The reduction in anchor damage between 2013 (47%) and 2016 (23%) is attributable to industry-wide retrofit of hold-downs in the intervening period. The simultaneous increase in barrel damage (28% →54%) and cone damage (14% →43%) could be suggestive of a migration of the predominant failure mode from the anchorage to the tank shell, of the kind that a stiffer, more capacity-protected anchorage might in principle be expected to produce.

RSFD AS A TANK ANCHORAGE SYSTEM

The Resilient Slip Friction Damper (RSFD) developed by Mohammadi Darani et al. (2021) is a friction-based connection with self-centring capability. Two coaxial parts — an inner shaft and a slotted outer cylinder — are clamped together by pre-tensioned bolts to generate a calibrated friction interface. A stack of disc springs, pre-stressed beyond the friction force, sits in series with the friction interface and provides the restoring force on unloading. The result is a stable flag-shaped force–displacement response: linear-elastic uptake to the slip force, frictional plateau (with adjustable post-slip stiffness governed by the disc spring stack) up to the design displacement, and full self-centring on unloading without material yielding.

Adaptation for tank use

The original RSFD device required modification to be acceptable in dairy and winery environments. The redesigned version used in this study features (i) a slotted upper section with a robust cap and a disc-spring sleeve, allowing controlled clamping force; (ii) circulation holes to prevent moisture accumulation in the cylinder; (iii) full stainless-steel construction (cylinder, shaft, bolts, fasteners) to align with the tank metallurgy and resist exposure to wine, milk, cleaning agents, and outdoor atmosphere; and (iv) a welded base block enabling direct attachment to the tank skirt without an intermediate clip plate.

Component-level cyclic testing

A prototype with 40 kN force capacity and 15 mm deflection capacity was manufactured and tested in a universal testing machine under cyclic protocol. The intended flag-shaped hysteresis was achieved without strength or stiffness degradation across multiple cycles (Figure 4(b)). A second-generation device with 150 kN force capacity and 15 mm deflection capacity was subsequently tested. The fitted experimental backbone closely tracks the design backbone, confirming both the analytical formulation and the manufacturability of the device at production capacity (Figure 5).

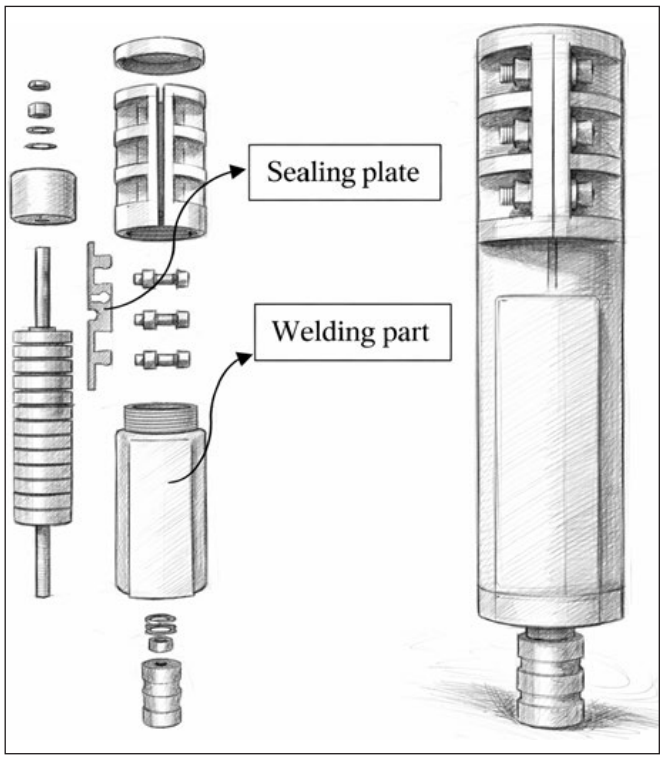


Figure 2: Exploded view of the RSFD assembly.

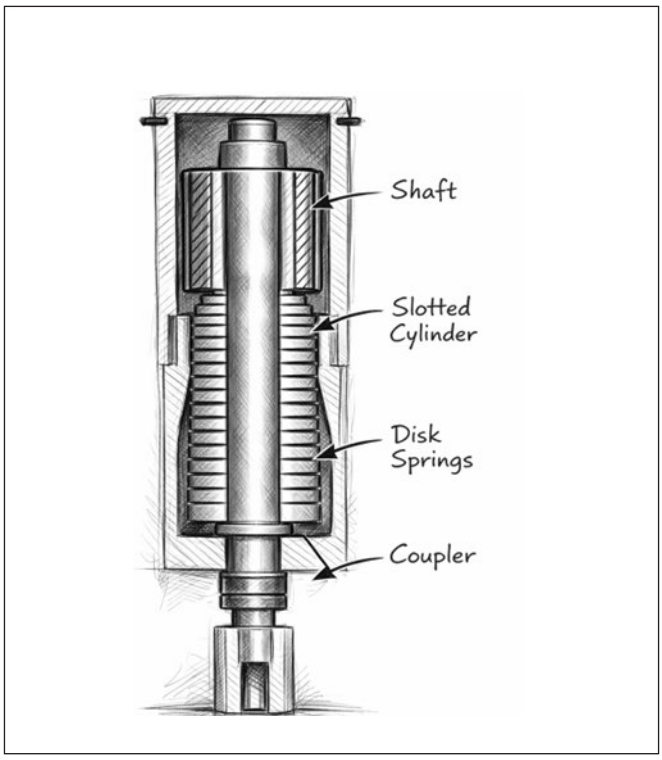


Figure 3: RSFD section illustrating the disc-spring stack pre-stressed against the slotted cylinder.



Figure 4(a): 40 kN RSFD prototype mounted in the Universal Testing Machine.



Figure 5(a): 150 kN RSFD prototype mounted in the Universal Testing Machine.

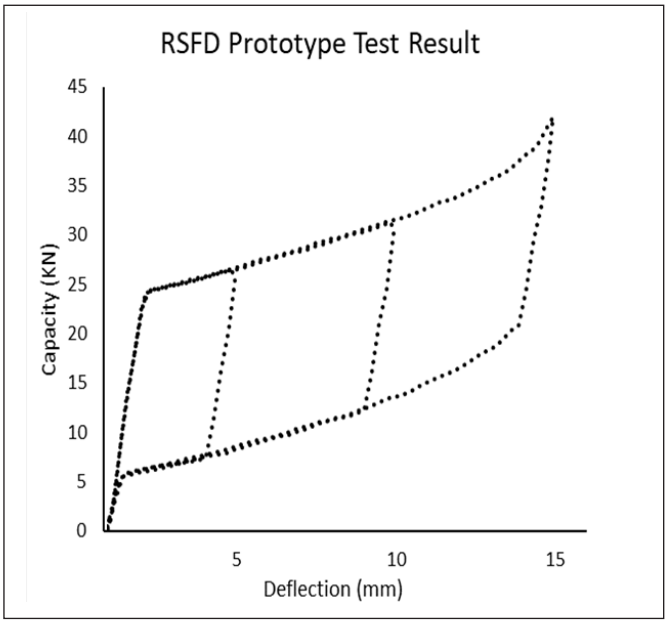


Figure 4(b): Measured cyclic response of the 40 kN prototype.

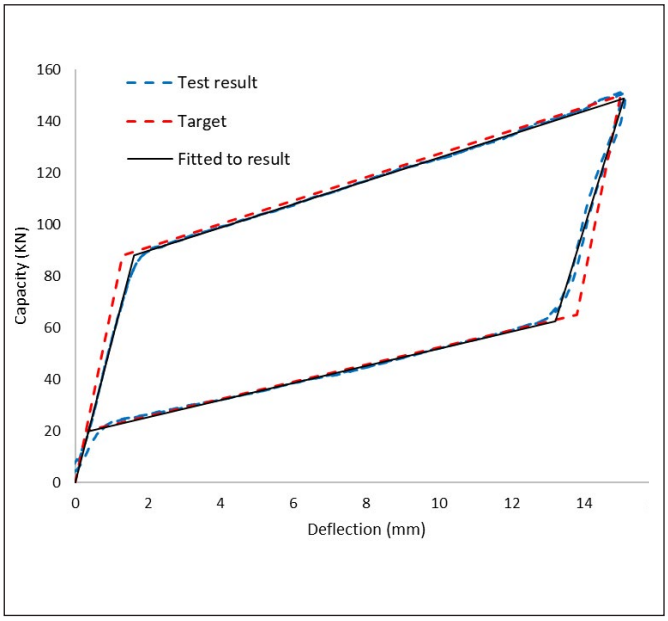


Figure 5(b): Cyclic test result for the 150 kN production-scale RSFD, with target backbone (red dashed), measured response (blue dashed), and fitted backbone (black solid).

Tension-only load path

The RSFD is configured as a tension-only hold-down. Compression is not transferred through the connection because the disc-spring stack is engaged only under tensile elongation, and the device freely closes back to zero force on unloading. Consequently, the tank shell is not subjected to localised compression at the gripping point, reducing the likelihood of the elephant-foot and diamond buckling modes associated with concentrated compressive demand. The load path during reverse cycles is carried by the tank base plate uplift–reseating mechanism, exactly as in a freely rocking unanchored tank, but with the rocking amplitude controlled by the RSFD slip force and disc-spring stiffness.

DESIGN METHODOLOGY

The proposed methodology integrates the RSFD into the existing NZSEE 2009 / API 650 design framework. The seismic base shear contribution from mode i in the NZSEE 2009 spring-mass formulation is given by:

$$V_i = C_h(T_i) \cdot N(T_i,D) \cdot m_i \cdot g \cdot S_p \cdot Z \cdot R_u \cdot k_f \quad (1)$$

where $C_h(T_i)$ is the spectral shape factor for the relevant mode and site subsoil class; $N(T_i, D)$ is the near-fault factor; m_i is the modal mass (impulsive or convective); S_p is the structural performance factor; Z is the seismic hazard factor; R_u is the return-period factor; and $k_f(\mu, \zeta)$ is the combined ductility and damping reduction factor used here as the metric to compare anchorage performance.

Following the API 650 (2020) framework, the response modification factor R combines ductility, damping, and over-strength contributions:

$$R = R_\mu \cdot R_\zeta \cdot R_\Omega \quad (2)$$

where R_μ accounts for ductility, R_ζ for damping, and R_Ω for over-strength. For a mechanically anchored tank, ASCE 7-16 (2017) sets R_Ω = 2.0. For the RSFD, the same factor is set directly by the device force–displacement geometry:

$$R_\Omega = \frac{F_{max}}{F_{slip}} \quad (3)$$

For the RSFD, R_Ω is determined directly from the disc-spring force–displacement geometry between the slip force F_{slip} and the ultimate force F_{max} at the design displacement and is therefore independent of material variability, giving R_Ω = 2.0. A separate device-level capacity-design over-strength factor $\Omega = F_{ult} / F_{max}$ governs the capacity protection of the tank barrel and foundation, where F_{ult} is the device force at the ultimate displacement $\Delta_u = 1.5 \cdot \Delta_{ULS}$; for the disc-spring geometry adopted, Ω = 1.15–1.20. The relationship between k_f and the ductility / damping decomposition is:

$$k_f = \frac{1}{R_u \cdot R_\zeta} \quad (4)$$

The design procedure adopted in this study is:

1. Determine the design-level overturning moment using the impulsive/convective decomposition of NZSEE 2009 (or API 650 Appendix E) with R selected for an RSFD ductile anchorage system.
2. Calculate the number, capacity, and arrangement of RSFD units required to resist the uplift force at the skirt, with the slip force F_{slip} / F_{ult} set in the range 50%–60%.
3. Size the disc-spring stack so that the device deflection capacity Δ_u is at least $1.5 \cdot \Delta_{ULS}$, providing margin for the maximum considered earthquake (MCE).
4. Capacity-design the tank barrel, anchor chair / skirt, and foundation using the RSFD over-strength factor Ω = 1.15–1.20.
5. Verify the SLS slip threshold against wind and operational loads to ensure the RSFD does not slip in service.

NUMERICAL CASE STUDIES

Three cylindrical tanks with aspect ratios H/D of 0.5, 3 and 5 were modelled to investigate a range of tank slenderness (Table 1). Each tank was analysed with three anchorage systems—necked-rod, buckling-restrained and RSFD—using nonlinear time-history analysis (NLTHA). The liquid response was represented using an equivalent spring–mass analogue based on the impulsive–convective formulation of Housner (1957), with effective liquid masses determined using API 650 (2020, Annex E). The key dynamic properties of each tank are summarised in Table 2. The tank walls were designed to remain elastic, with ductility provided exclusively through the hold-down system. A suite of seven ground-motion records (Chi-Chi, Imperial Valley, Kobe, Kocaeli, Landers, Loma Prieta and Northridge), scaled to the ASCE 7-16 (2017) spectrum, was applied; results below are averages across the suite. The scaled response spectra (Figure 6) are in reasonable agreement with the target spectrum for periods up to approximately 2.5 s.

The components of R were derived from the analysis results as follows. The product $R_u R_\zeta$ was calculated as the ratio of the force demand for a fully elastic hold-down system to that for the ductile system. For the necked-rod and buckling-restrained systems, R_Ω was determined from the ratio of the ultimate force demand obtained from the analysis to the yield force established at the design stage;

to determine R_Ω, the records were scaled to match the overturning moment of a fully elastic hold-down system, addressing potential discrepancies between the numerical results and the code-prescribed values. For the RSFD, R_Ω follows directly from Equation (3).

Table 1: Case-study tank specifications.

Case	V (m^3)	H (m)	D (m)	H/D
1 - Squat	395	5	10	0.5
2 - Mid-aspect	295	15	5	3.0
3 - Slender	490	25	5	5.0

Table 2: Dynamic properties of the case-study tanks

Case	Impulsive			Convective		
	Mass (t)	Height (m)	Period (s)	Mass (t)	Height (m)	Period (s)
1 - Squat	213.0	1.88	0.07	171.7	3.03	3.37
2 - Mid-aspect	273.1	7.03	0.21	22.6	13.64	2.33
3 - Slender	469.5	12.03	0.49	22.6	23.64	2.33

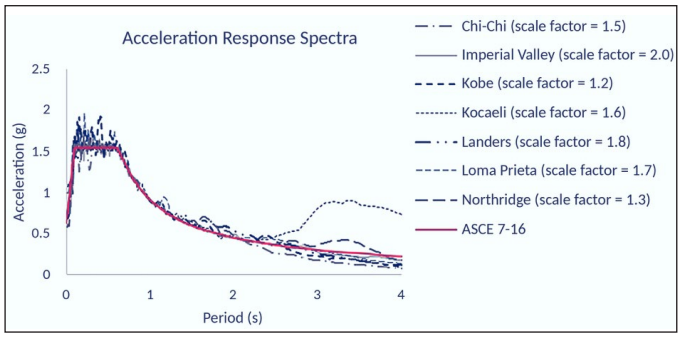


Figure 6: Acceleration response spectra of the seven ground-motion records used in the nonlinear time-history analyses, scaled to the ASCE 7-16 (2017) spectrum (red line).

Necked-rod results

Necked rods were sized following NZSEE 2009 with target R = 4 (Table 3). The component backbone was derived from FE analysis (SeismoStruct 2020) and verified against published cyclic data (Prota et al., 2009); the representative M22 Grade 300 backbone is shown in Figure 7. The combined R factor returned by the analyses was 3.8–4.0 across the three tanks, consistent with the design assumption, but the over-strength contribution R_Ω ranged from 2.46 to 3.00 — exceeding the ASCE 7 over-strength value of 2.0. The tank shell and foundation are thus under-protected relative to the rod itself. Case Study 3 tank-level hysteresis is shown in Figure 8, in terms of base shear against the horizontal displacement at the top cone of the tank.

Table 3: Necked-rod hold-down specifications.

Case	No. of hold-downs	Fy (MPa)	Rod diameter (mm)
1	20	300	22
2	28	300	40
3	46	500	40

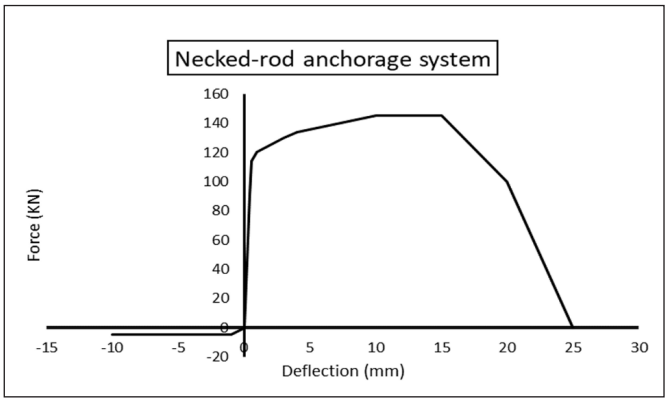


Figure 7: Assumed force–displacement backbone for the M22 Grade 300 necked-rod, derived from FE analysis.

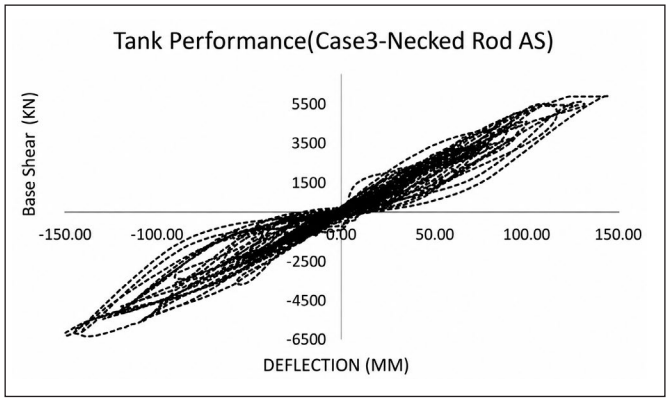


Figure 8: Hysteresis response of Case Study 3 (490 kL) tank under the 1999 Kocaeli record, anchored with 46 × M40 necked-rod hold-downs (Grade 500). Deflection is the horizontal displacement at the top cone of the tank.

Buckling-restrained results

Buckling-restrained rods, sheathed to prevent compressive buckling, were sized for design R values of 4, 5 and 6 in each case (Table 4). The 295 kN device backbone — common to Cases 2 and 3 — is shown in Figure 9: fuller and symmetric in tension and compression. Analysed R factors matched intent within tolerance, but R_Ω again exceeded 2.0 in all but the lowest-R designs. Raising R from 4 to 6 reduced anchor counts (76 →50 in Case 3) at the cost of larger residual displacement (Table 6). Case Study 3 tank-level response is shown in Figure 10.

Table 4: Buckling-restrained hold-down quantities and device capacities.

Case	R = 4	R = 5	R = 6	Device capacity (kN)
1	16	14	10	118
2	28	22	18	295
3	76	62	50	295

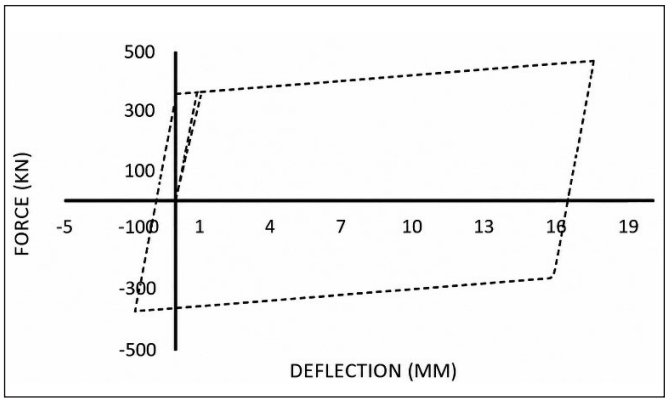


Figure 9: Assumed hysteresis backbone of the 295 kN buckling-restrained rod used in Cases 2 and 3.

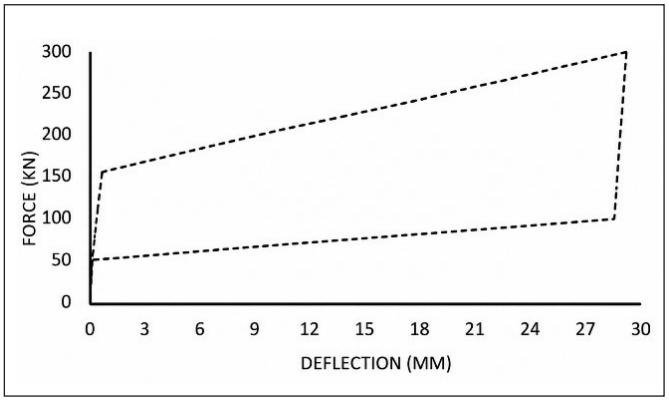


Figure 11: Assumed flag-shaped force–displacement hysteresis of the 300 kN RSFD device (slip force 150 kN, ultimate 300 kN at the design displacement of 18 mm).

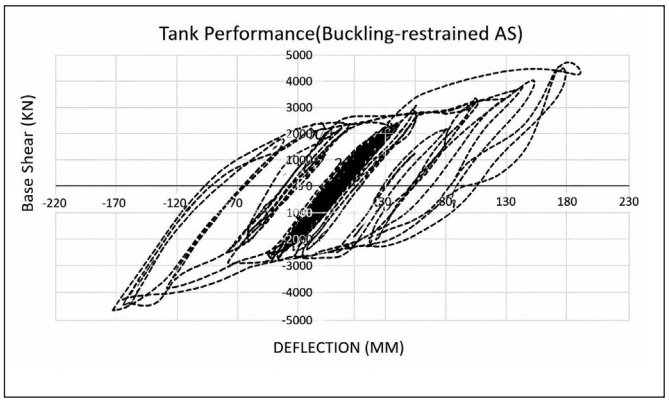


Figure 10: Hysteresis response of Case Study 3 (490 kL) tank under the 1999 Kocaeli record, anchored with 50 × 295 kN buckling-restrained hold-downs (R = 6 design).

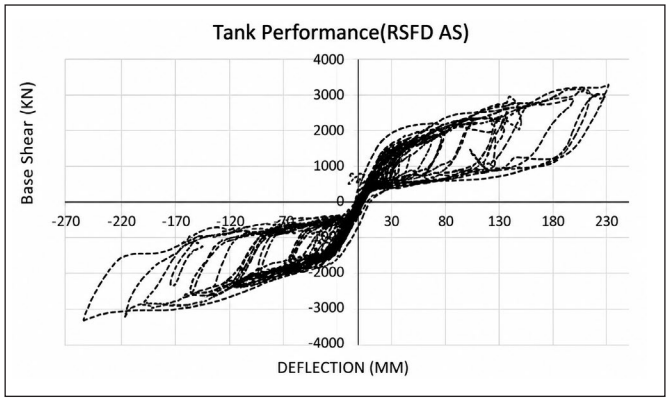


Figure 12: Hysteresis response of Case Study 3 (490 kL) tank under the 1999 Kocaeli record, anchored with 44 × 360 kN RSFD hold-downs. Deflection is the horizontal displacement at the top cone of the tank.

RSFD results

RSFD units were sized with slip force at 50% of ultimate device capacity and 1.5·ΔULS deflection capacity (Table 4). The 300 kN device backbone is shown in Figure 10 — flag-shaped self-centring response with linear-elastic uptake to F_{slip} ≈ 150 kN, frictional plateau to F_{max} = 300 kN at design displacement, and full return to zero force on unloading. Combined R factors (with the API 650 1.4 conversion applied) were 3.8, 6.7 and 7.7 for Cases 1, 2 and 3 respectively. R_Ω is fixed at 2.0 by disc-spring geometry; residual displacement is zero in every case examined. Case Study 3 tank-level response is shown in Figure 11.

Table 5: RSFD hold-down specifications.

Case	No. of hold-downs	Design capacity (kN)
1	14	200
2	22	300
3	44	360

System comparison: over-strength and overturning-moment demand and aftershock resilience

The analysed over-strength factor R_Ω is plotted in Figure 12 for the three case-study tanks across the five system variants (RSFD, necked-rod, and buckling-restrained with R = 4, 5, 6). For the RSFD, R_Ω equals 2.0 in all three cases, reflecting that this factor is determined by the disc-spring force–displacement geometry between F_{slip} and F_{max}. For the necked-rod and buckling-restrained variants, R_Ω ranges from 2.04 to 4.20 across the cases, generally increasing as the design R is raised. The highest values are obtained for Case 3, the slender tank, reaching 3.9 for the necked-rod design and 4.2 for BR-R6. These values indicate that for the conventional anchorages, the tank shell and foundation must be capacity-protected for forces above the code assumption.

The corresponding overturning-moment index (analysed overturning moment normalised against the elastic-design demand) is plotted in Figure 13. For the RSFD, the index is 0.53, 0.30 and 0.26 for Cases 1, 2 and 3 respectively, corresponding to overturning-moment reductions of 47%, 70% and 74% relative to the elastic-design baseline. For

the necked-rod system in Case 3, the index is 0.98. Across the conventional buckling restrained systems, increasing R from 4 to 6 reduces the index by 11% in Case 1 and by 11–18% in Cases 2 and 3. The relative differences between the RSFD and the conventional systems widen with aspect ratio: in Case 1 (H/D = 0.5) the RSFD index is about 30% lower than the lowest buckling-restrained value (0.70 for BR-R6), and in Case 3 (H/D = 5.0) it is about 38% lower (0.26 versus 0.42 for BR-R6).

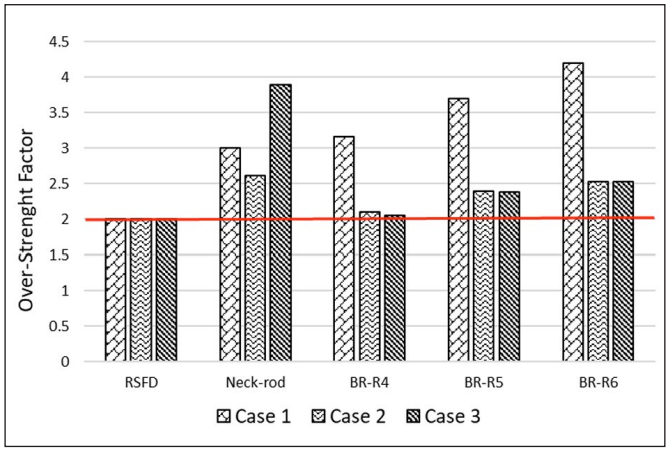
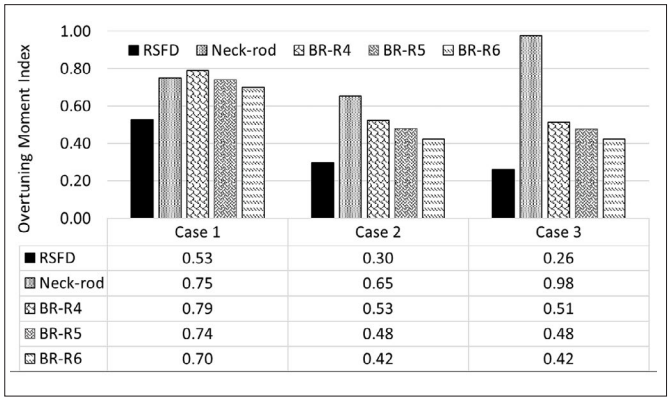


Figure 13: Comparison of the analysed over-strength factor RΩ across the three case-study tanks for the RSFD, necked-rod, and buckling-restrained (R = 4, 5, 6) anchorage variants.



	Case 1	Case 2	Case 3
RSFD	0.53	0.30	0.26
Neck-rod	0.75	0.65	0.98
BR-R4	0.79	0.53	0.51
BR-R5	0.74	0.48	0.48
BR-R6	0.70	0.42	0.42

Figure 14: Comparison of the overturning-moment index (analysed overturning moment normalised against the elastic-design demand) across the three case-study tanks for the RSFD, necked-rod, and buckling-restrained (R = 4, 5, 6) anchorage variants.

Residual displacement was recorded at the buckling-restrained connections in all three case studies. In these analyses, the reverse cycle, driven by the resisting moment from the weight of the tank and its contents, was not sufficient to overcome the compression resistance of the elongated buckling-restrained rods, so the rods did not fully close, leaving a local gap at the connection. The tank base plates are flexible in out-of-plane bending and provide little restoring force to close this gap; external force would therefore be required to return the tank to its initial position. The residual deflection averaged approximately 32% of the peak hold-down displacement; per-case values across the R = 4, 5, 6 designs are given in Table 6, reflecting the necessity of anchorage inspection and likely replacement following a design-level event.

Table 6: Residual / peak displacement ratio for buckling-restrained connections.

Case	R = 4	R = 5	R = 6
1 (H/D = 0.5)	30%	39%	46%
2 (H/D = 3.0)	19%	24%	31%
3 (H/D = 5.0)	21%	26%	35%

FIELD IMPLEMENTATIONS

Four Aotearoa New Zealand projects have implemented the RSFD anchorage system across the wine and dairy sectors, spanning the practical range of capacity, aspect ratio, and operating environment encountered in industry. Project-specific design parameters and analytical performance metrics are summarised in Table 7.

Table 7: Summary of k_f reduction across the four implemented tanks.

Project	k_f (RSFD)	k_f (Rod)	H/D	Reduction
125 kL wine tank	0.52	0.77	1.9	33%
175 kL wine tank	0.54	0.77	2.2	29%
310 kL wine tank	0.50	0.77	1.7	35%
250 kL dairy silo	0.46	0.86	3.6	52%

Across the implementations, k_f reductions of 29%–52% were obtained relative to a necked-rod baseline, with the largest gain in the slender 250 kL dairy silo (k_f reduced from 0.86 to 0.46). The 310 kL wine tank implementation is representative: twelve RSFDs of 140 kN ULS capacity (12 mm at ULS, 18 mm ultimate, Ω = 1.20) replaced the original necked-rod design and reduced k_f from 0.77 to 0.50. The reduced moment demand allowed the tank shell to be specified with thinner plate over the lower courses and reduced the foundation moment, both of which produced material savings. The over-strength factor of Ω = 1.20 quoted for the 310 kL implementation reflects the activation of a secondary fuse mechanism beyond the ULS displacement, in which controlled rod elongation provides additional deflection capacity prior to reaching the device ultimate state (Sahami et al., 2021b).

The installation sequence adopted across the four implementations consisted of a fillet weld between the RSFD and the tank skirt, followed by a base-plate connection to the foundation anchor (typically a chemical anchor stud). The device itself is supplied as a sealed assembly with a pre-welded base block, with no field assembly of internal components required.



Figure 15: Installed 175 kL stainless-steel wine tank with RSFD hold-downs around the skirt perimeter, Marlborough.

SLIP FORCE TUNING

Parametric study on the slip-force ratio F_{slip} / F_{ult} was carried out for Case Study 2 across the range 30%–90%. Below 50%, displacement demand grew rapidly without proportionate gain in k_f reduction ($k_f \approx 0.49$ at 30%, top-cone displacement 43 mm). Above 60%, displacements stabilised at around 28 mm but k_f climbed steadily to 0.73 at 90%. The 50%–60% window is recommended as the practical design range; 50% offers the largest force reduction with adequate displacement control, while 60% may be preferable where service or wind slip threshold governs.

CONCLUSIONS

Post-earthquake damage data from the 2013 Seddon and Lake Grassmere and 2016 Kaikōura events demonstrate that conventional ductile rod-based anchorage of cylindrical steel storage tanks is not consistent with low-damage design objectives. The retrofit of hold-downs between 2013 and 2016 succeeded in reducing anchor damage from 47% to 23% of tanks, but barrel and cone damage rose from 28% to 54% and from 14% to 43% respectively — the load path simply migrated up the tank.



Figure 16: Close-up of installed RSFD hold-downs welded to the tank skirt and bolted to the foundation.

This paper has presented a self-centring, tension-only anchorage system based on the Resilient Slip Friction Damper. The system has been verified through component testing on 40 kN and 150 kN devices, evaluated through nonlinear time-history analysis on three case-study tanks of aspect ratio 0.5, 3 and 5, and applied to four Aotearoa New Zealand wine and dairy projects. Specific findings:

- Tanks anchored with the tension-only system returned an overturning-moment demand on average 30–40% lower than the other conventional anchorage systems analysed, with the relative reduction increasing with tank aspect ratio.

- A slip-force ratio in the range 50–60% of the ultimate device capacity is recommended on the basis of the parametric study, with 50% giving the largest force reduction within stable displacement bounds.
- The analysed over-strength factors for the necked-rod and buckling-restrained designs were observed to exceed the ASCE 7-16 (2017) over-strength value of 2.0 by a considerable margin, particularly where higher design ductility values were adopted, reaching 2.46–3.00 for the necked-rod designs and up to 4.2 for the buckling-restrained R = 6 design.
- The influence of compressive demand at the gripping point on elephant-foot and diamond buckling under stiffer anchorage systems warrants further study under realistic tank-anchorage boundary conditions. The tension-only configuration of the RSFD avoids this load path by design, and presents a useful point of comparison in such future work.

ACKNOWLEDGEMENTS

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TALL HYBRID BUILDING WITH OUTRIGGER - BELT TRUSS STRUCTURE

Iqbal, A.[1], Sadeghi, P.[2]

ABSTRACT

Tall wood buildings are emerging as a practical option with increasing demand to find solutions for sustainable development of built environments. Despite significant progress in technology with engineered wood products, it often remains challenging to meet drift requirements with inherent material properties of wood. Hybrid structures combining wood with other materials can be a practical option in those cases for technical and economic reasons. In this paper a 20-storey building structure is analysed and designed with steel core and timber-steel hybrid outriggers and belt trusses for a high seismic and wind load region. The exercise demonstrates feasibility of adopting this concept in practical structures subject to comparable demands.

INTRODUCTION

Tall building construction introduces considerations such as high demands on energy and resources to urban environments, in addition to complexity in design and implementation. Efforts have been underway in recent years to find ways to meet sustainable design principles for tall buildings with optimised shape, size and type of structure, materials, and efficiency to provide safety, serviceability and economy. One of the structural configurations used in modern tall buildings is the outrigger and belt-truss system, where exterior and interior frames are combined by horizontal links called outriggers at specific levels and the perimeter columns of the exterior structural system are tied together by belt trusses at the same levels (Coull and Lau, 1989). The central core of the building often comprises vertical trusses. It leads to an efficient arrangement by utilising axial strength and stiffness of exterior columns to resist a portion of the overturning moment due to lateral forces. The outrigger and belt truss system is also effective in reducing drifts due to lateral loads. Such properties are useful for high-rise buildings, particularly those in high seismic or wind regions.



Figure 1: 140 Williams Street, Melbourne (left) and AON Centre, Wellington (right)

The outrigger-belt truss system was developed in the late 1960s by Skidmore, Owings and Merrill (SOM) of Chicago for 41-storied 140 William Street (formerly known as BHP House) in Melbourne, Australia (Figure 1), with three belt trusses at the bottom, middle and top of the building. The exposed belt trusses serve both aesthetic and structural purposes. It was subsequently used by SOM in the 42-storied First Wisconsin Center in Milwaukee, since renamed U.S. Bank Center. The 30-storied AON Centre (Figure 1, formerly BNZ Centre then State Insurance Building) in Wellington includes the system, as is the case for many other tall buildings.

Modern engineered wood products have emerged as a practical sustainable construction material with applications in a growing number of tall buildings. However because wood has a relatively low modulus of elasticity, code-specified drift limits may govern the structural design, and it is therefore often challenging to satisfy the requirements for tall wood buildings. Structures employing the outrigger and belt truss system offer an option that provides reduced drifts at the top, increased lateral load capacity and overall stability.

To evaluate the feasibility of adopting wood in tall buildings employing the outrigger and belt truss system, the design of a 20-storied virtual building is attempted here. The hybrid structure consists of a steel core comprising a

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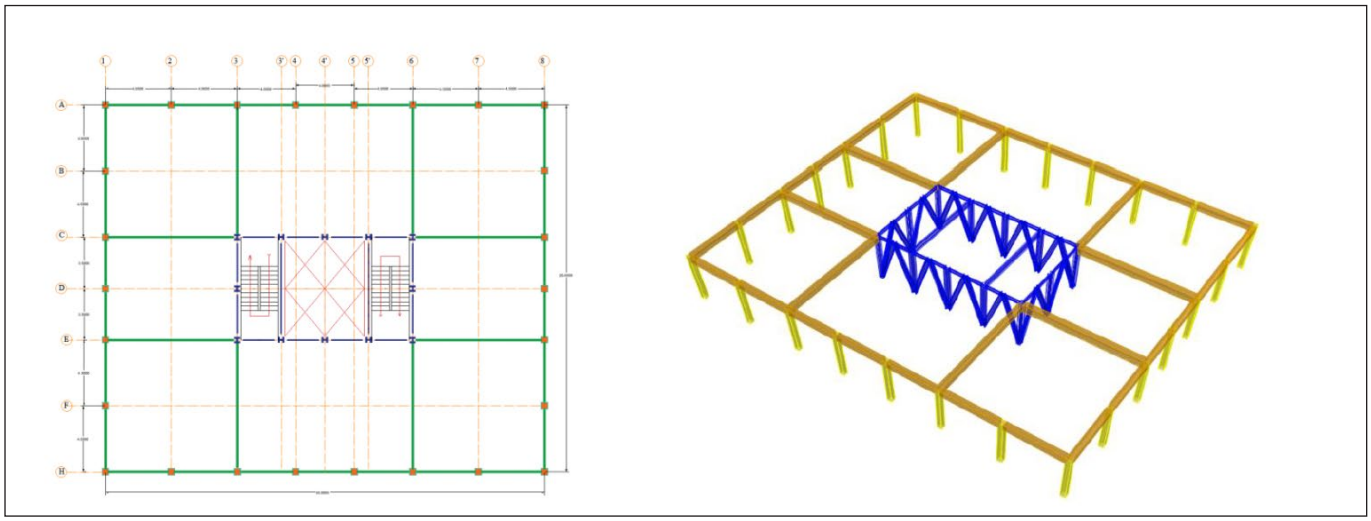


Figure 2: Building plan and view of typical floor mode in ETABS.

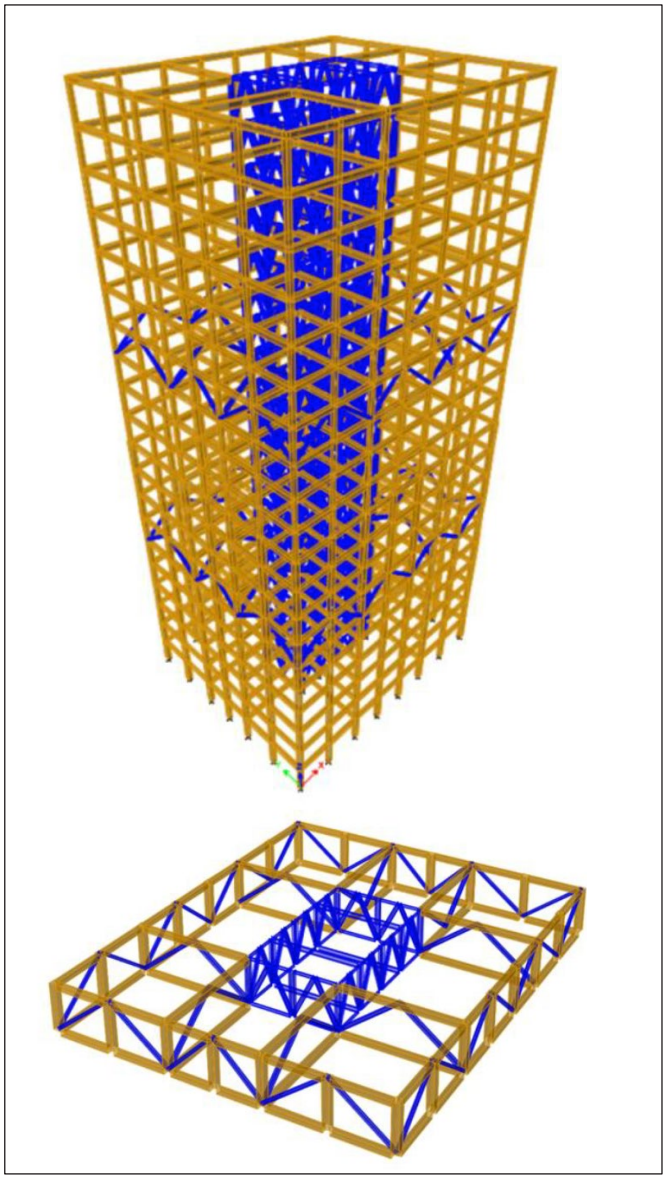


Figure 3: Model of complete structure and floor with outriggers and belt trusses in ETABS.

chevron brace frame and glulam exterior frames, connected by two sets of hybrid outriggers and belt trusses. The building is designed for Vancouver, British Columbia, and is located in a high seismic zone along with significant wind loads.

Building Description and Modelling

The building is 30 m x 25 m rectangular in floor plan and each storey height is 3.5 m with uniform geometry (Figure 2). Outriggers with belt trusses are placed at the 7th and 14th storey. The building structure has been modelled and analysed using Structural Analysis and Design Software ETABS (CSI Berkeley, 2025). Gravity loads and lateral loads as per Canadian National Building Code (NBCC, NRC, 2025) are applied. Static and dynamic linear analyses are carried out to obtain building drifts, storey shear and overturning moments. A view of the ETABS model of the structure and floor with outriggers and belt trusses are shown in Figure 3. The glulam members are shown in brown while steel sections are shown in blue.

All the members in the core and diagonals in the outriggers and belt trusses are steel sections. Steel used in the model are of ASTM A992 Grade 50 (Fy = 344.74 MPa, Fu = 448.16 MPa). Mass timber members used as primary structural components of the structure are made of Douglas Fir-Larch 24f-Ex for beams and Douglas Fir-Larch 16c-E for columns, with Cross-Laminated Timber (CLT) grade E1 for floors as per Canadian Standard CSA O86 (CSA, 2019). Details are in Table 1 below.

Timber sections are difficult to fix at the ends compared to leaving them with pin-end boundary restraints except where they are continuous through multiple stories. If fixing at ends is required, timber will typically require embedment into concrete or the use of substantial steel fasteners.

Column bases at exterior frames are therefore modelled with pinned connections to foundations. Timber connections in general are considered pinned, to avoid connection failures and maintain integrity of the overall structure. All elements are compatible with system deformations expected during a design event.

The outrigger system comprises relatively stiff horizontal trusses connecting the core and connected to the exterior columns. Ideally the core is centrally located, at equal distance from the exterior frames in each direction. The outriggers join the column with the core to resist rotation when the structure is subjected to lateral loads, causing the external moment to be resisted by combined action for the bending of the core and the axial tension and compression of the exterior columns connected to the outriggers. Overall moment resisting of the structure is increased with the core acting as a vertical cantilever, and through tension in the windward columns and compression in the leeward columns. The columns at the periphery also help in distributing the moment and reducing rotation of outriggers, by combining the exterior columns with trusses in orthogonal directions forming the belt truss around the periphery of the building. The belt trusses are vertically at the same level as the outriggers. The combined belt trusses and outriggers stiffens the building and reduces rotation and drift of the structure.

For optimum performance outriggers are placed at 1/ (n+1), 2/ (n+1) up to the n/ (n+1) height locations, with n as the number of outriggers (Kian and Siahaan, 2001). The two outriggers for the building are placed at 1/3rd and 2/3rd of the total height (at 7th and 14th Storey), respectively.

Loads and Analysis

In addition to dead load, the virtual building had live load, snow load, super-imposed dead load and live load (according to NBCC 2025) applied to the structure as detailed in Table 2. The climatic information for snow loads according to NBCC 2025, clause 4.1.6 are:

Location: West Vancouver: $S_s = 2.4\ kPa$, $S_r = 0.2\ kPa$

Importance category: Normal, $I_s = 1$, $C_b = 0.8$, $C_w = 1$, $C_s = 1$, $C_a = 1$

The spectral response acceleration values, Sa(T), for the reference ground conditions (Site Class C) are determined in accordance with NBCC 2025- Appendix C and are based on a 2% probability of exceedance in 50 years. The site coefficient (F) for spectral acceleration S(T) is determined in accordance with Clause 4.1.8.4.(4) NBCC (2025). The design spectral response acceleration S(T) is plotted in Figure 4.

Table 1: Details of structural components

OBJECT TYPE	MATERIAL	SECTION
Beam	Douglas Fir-Larch 24f-Ex	365X418 mm
Beam	Douglas Fir-Larch 24f-Ex	415X494 mm
Column	Douglas Fir-Larch 16c-E	515X496 mm
Column	Douglas Fir-Larch 16c-E	365X342 mm
Brace	ASTM A992 Grade 50	2C15x50
Belt Truss	ASTM A992 Grade 50	HSS 8x8x3/8
Belt Truss	ASTM A992 Grade 50	HSS 8x8x1/2
Outrigger	ASTM A992 Grade 50	HSS 10x10x1/2
Beam	ASTM A992 Grade 50	W200X22
Beam	ASTM A992 Grade 50	W360X51
Column	ASTM A992 Grade 50	W310X226
Column	ASTM A992 Grade 50	W310X129
Floor	CLT grade E1	Thickness=315mm

Table 2: Gravity loads on structure

Dead and super-imposed dead loads (SIDL)	
SIDL	Finishes and roof insulation = 0. 5 kPa M & E Services= 0.25 kPa
SIDL Floor 1-19	D_{Roof} = 0.75 kPa Finishes and sound insulation = 0.35 kPa M & E Services= 0.25 kPa Partition load=1kPa
SIDL Cladding (all levels)	$D_{flr\ 1-19}$ = 1.6 kPa D_{Clad} = 2 kN/m (perimeter)
Live and snow loads (NBCC 2025)-Table 4.1.5.3	
Roof	S_{Roof} = 2.12 kPa
Floor 1-19	$L_{flr\ 1-19}$ = 1.9 kPa

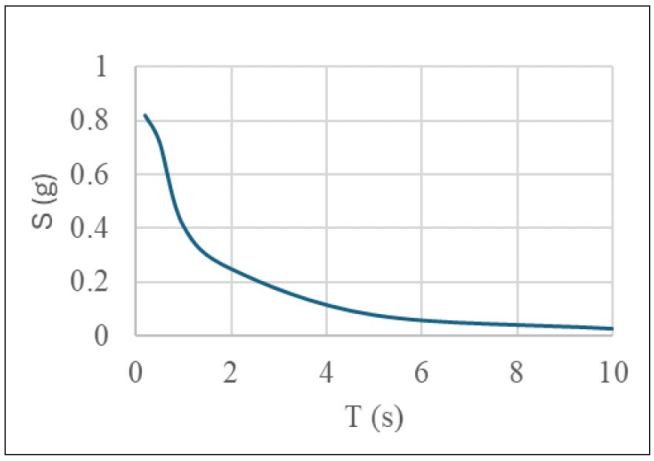


Figure 4: Design spectral response accelerations

Tall mass timber buildings are generally flexible and experience dynamic oscillations with significantly large amplitudes under winds loads. The building under consideration has a frequency of vibration greater than 1 Hz and dynamic wind load is required to be applied as per the NBCC 2025 requirement. It also permits use of the static wind analysis procedure to design the lateral load resisting system of buildings. Equivalent dynamic wind load was also estimated for static wind load analysis.

Seismic Design Considerations

Seismic weights of the various known superimposed loads and expected component self-weights are tallied in accordance with Clause 4.1.8.2 of NBCC (2025), including 60 percent of the storage load, 25 percent of the snow load, and partition loads of 0.5 kPa (half the gravity design value). The weights are typically lumped at floor levels. The lowest storey requires some additional consideration because if the lower half of the lowest storey is lumped at the ground level, some weight and resulting base shear is not captured. For a timber structure the lowest half storey of shear may more appropriately be neglected due to its relatively low weight and increased flexibility. Ductility Factor (Rd) of 3.0 and Overstrength Factor (Ro) of 1.3 were applied to the structure according to NBCC 2025. The stiffness provided by the belt truss and outrigger are neglected in determining Rd and Ro.

The structure with outriggers and belt trusses at intervals possesses irregularities in geometry and vertical stiffness. Analysis results confirm the stories with outrigger and belt truss are stiffer than the others and differences in stiffnesses between stories need to be taken into consideration in analysis. Although torsion is not expected to be significant for predominantly symmetric structures, the structure is checked for torsional response as required by the code. P-delta effects need to be accounted for in the structure. Application of the stability coefficient up to the maximum limit of 0.4 prescribed by the NBCC (2025) is intended to ensure the force demand does not exceed that of the resistance capacity, considering P-delta effects. The relevant NBCC load combination for determining P-delta effects for seismic is given as

$$1.0D + 1.0E + 0.5L + 0.25S + (1.0\ C\ if\ Crane\ exists)$$

Analysis methods considered for Hybrid concrete-timber and hybrid steel-timber SFRS are those allowed according to Technical Guide for the Design and Construction of Tall Wood Buildings in Canada (FPInnovations, 2022).

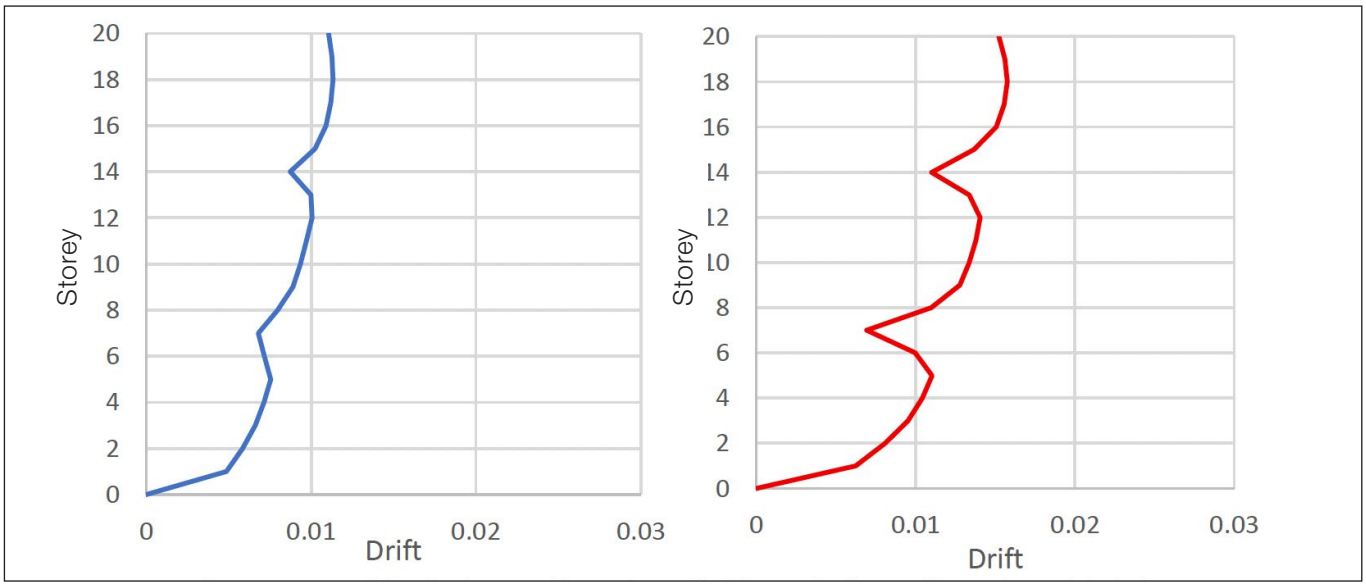


Figure 5: Drifts in long direction (left) and short direction (right)

Seismic Response

Results obtained from response spectrum analysis are observed and graphs are plotted in Figures 5, 6 and 7. Storey drifts, storey shears and overturning moments have been studied.

Figure 5 shows the variation of storey drift of response spectrum analyses in long and short directions. It is evident the drifts are well within acceptable limits. The variation in drift due to the presence of the outriggers and belt trusses are noticeable. Compared to the short direction, the long direction has less variation in storey drift above and below the outrigger-belt truss level.

Variation of storey shears of dynamic response spectrum analysis in the long and short directions are shown in Figure 6. Transverse shear deformation has been neglected. The long direction exhibits higher shear resistance, as expected. The values increase gradually without any significant variation in trend at the outrigger and belt truss levels.

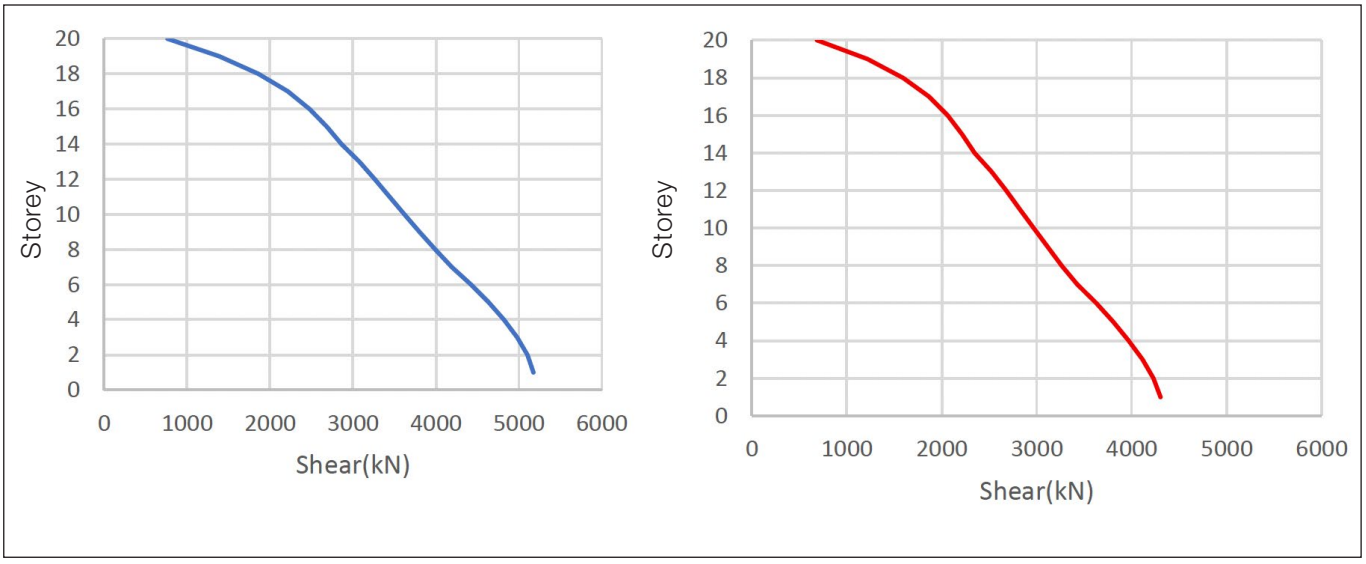


Figure 6: Storey shear in long direction (left) and short direction (right)

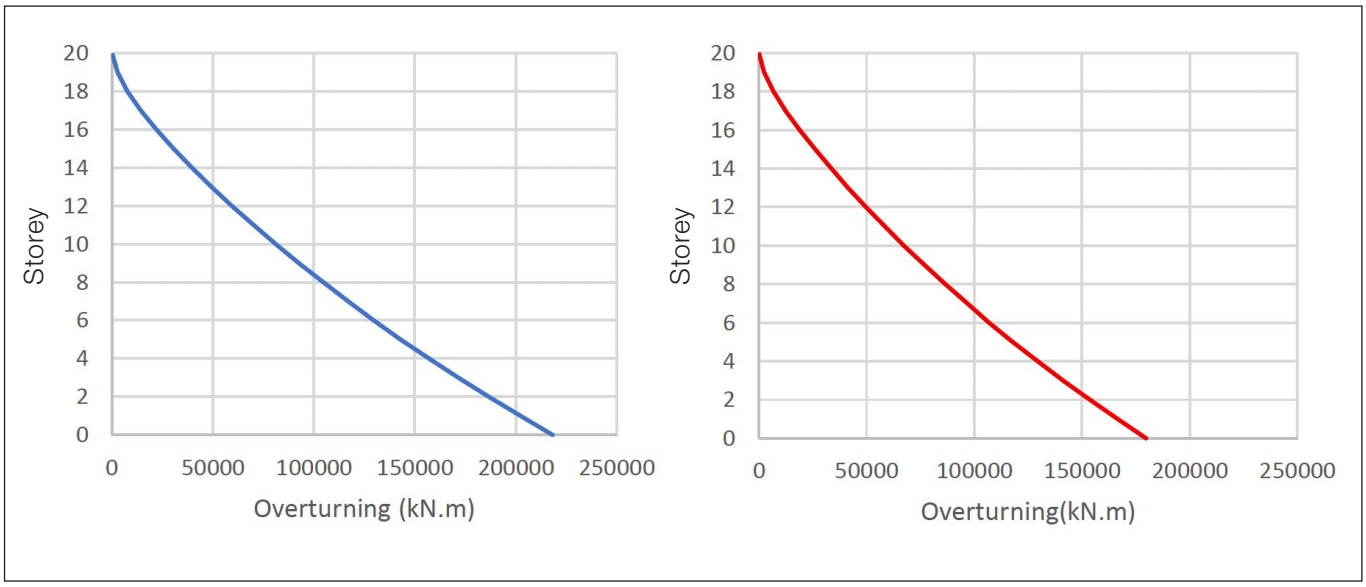


Figure 7: Moment in long direction (left) and short direction (right)

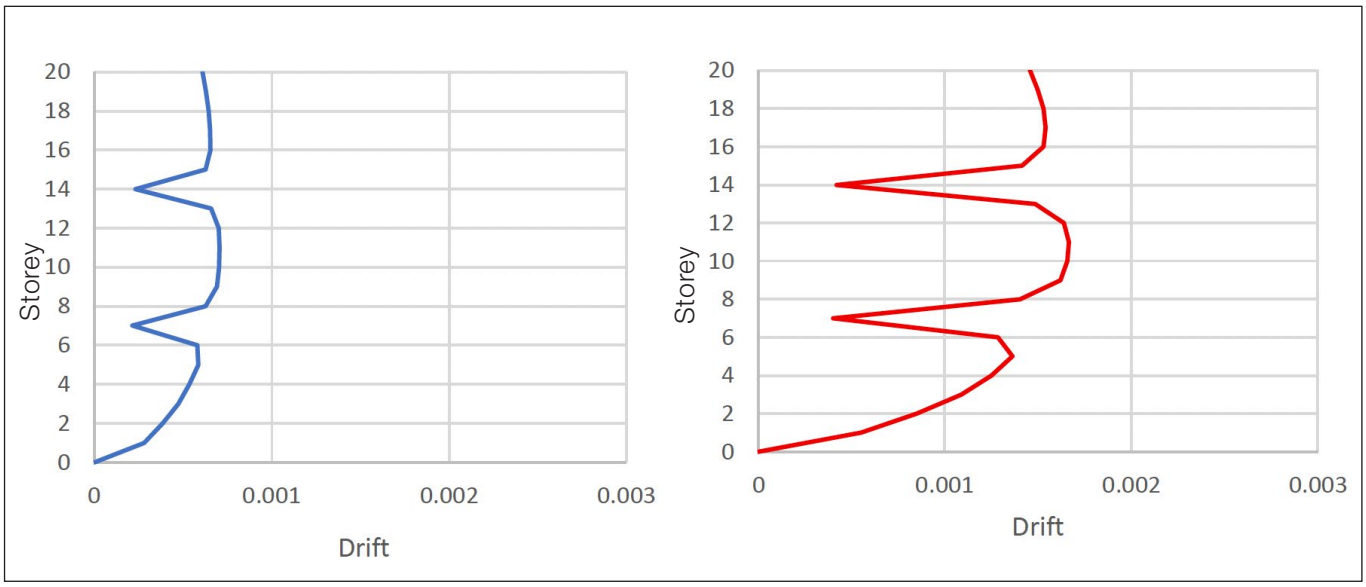


Figure 8: Drifts in long direction (left) and short direction (right)

Figure 7 shows a gradual increase of overturning moment throughout the structure. Similar to the case of shear, there is no noticeable change in trend between the moment at levels with outriggers and belt trusses and that at the levels above and below.

Response Under Wind Load

Analysis was also performed to calculate response of the structure under wind loads. Figure 8 shows maximum storey drifts for dynamic wind load in long and short directions. There are noticeable variations in drifts between levels with outriggers and belt trusses and those above and below. However, maximum storey drifts are found to stay well within the limits in both directions.

CONCLUSIONS

Performance of a virtual tall wood-steel hybrid building with timber frame and steel braced core is analysed for seismic and wind loads. Overall stiffness of the structure is increased by using an outrigger structural system against lateral loads acting such as wind and earthquakes. There is a reduction in maximum storey drift due to wind and earthquake loads in both directions. Results indicate that the outrigger and braced frame is an effective system for tall hybrid buildings, particularly for those with tight drift limits and where there are architectural restrictions in extending the core in plan. Further research is ongoing for additional information such as steel-to-timber connection details.

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Changes to the “Granny Flats” Building Consent Exemption

From 8 October 2026, changes to the "granny flats" building consent exemption will allow additional building products to be used and make it easier to use existing water supply, wastewater and stormwater services.

What's changing?

Certified composite structural building products can be used.

Granny flats can be built using certified composite structural building products. This includes some Structural Insulated Panel (SIP) systems and solid timber building systems.

The building product must be used:

- within the scope and limitations of its registered product certificate (CodeMark)
- in accordance with the manufacturer’s instructions, to ensure it complies with the Building Code.

This change supports modern construction methods, including prefabricated and off-site manufactured homes, and gives people more options when designing and building a granny flat.

Using existing water and drainage services

You may be able to use existing water supply, wastewater and stormwater services for a granny flat, rather than installing new systems.

Using existing council or utility connections

If your property is already connected to a council or other utility provider, you may be able to use or modify those connections for a granny flat.

The utility provider must agree that the existing connection is suitable. You do not need a building consent to make these changes, but you must get any approvals required by the utility provider.

Using existing on-site water and drainage systems

If your property uses on-site systems, such as rainwater, wastewater treatment or stormwater disposal systems, you may be able to continue using them or upgrade them for your granny flat.

The systems must have enough capacity and comply with the New Zealand Building Code.

Installing a new on-site system

If council or other utility services are not available, or cannot support an additional connection, you may be able to install a new on-site system for the granny flat.

Updated guidance and resources

From 8 October 2026, updated granny flats guidance, quick guides and other supporting resources will be available on the Building Performance website.

Building a granny flat: resources | Building Performance

New foundations guidance available

Guidance is now available to help homeowners and building professionals understand foundation options for granny flats built under the building consent exemption.

The guidance provides practical information to support the design and construction of granny flats.

Granny flats foundations guidance

Contact us at building@mbie.govt.nz if you have any questions.

Ngā mihi,
The Building Performance team
Ministry of Business, Innovation and Employment

UNIVERSITY PHD THESES

The following synopses have been prepared by PhD engineering students in relation to their theses studies.

IMPROVING THE THERMAL AND STRUCTURAL PERFORMANCE OF COLD-FORMED STEEL FRAMING

Dr Yazeed Al-Radhi (University of Auckland)
CPEng (NZ), CPEng (Aust), PhD, MEng, BEng (Hons)

Cold-formed steel (CFS) framing offers significant advantages for residential and light commercial construction, including high strength-to-weight ratio, dimensional accuracy, durability and suitability for prefabricated construction. However, steel's high thermal conductivity creates thermal bridges through insulated building envelopes, reducing wall thermal resistance and making energy-efficiency requirements more challenging to achieve.

This PhD research at the University of Auckland investigated the problem from both thermal and structural engineering perspectives, with the aim of improving the thermal efficiency of CFS framing without compromising structural performance.

The first part of the research examined heat transfer through complete CFS wall systems using validated two- and three-dimensional finite element models. The results showed that thermal performance is influenced not only by cavity insulation, but also by the geometry and distribution of the steel framing. Studs, tracks, nogs and concentrated framing at wall junctions can create significant heat-flow paths that simplified calculation methods may not adequately capture.

An improved method for predicting CFS wall thermal transmittance was then developed using Response Surface Methodology (RSM). Based on a large database of numerical analyses, the method provides an efficient alternative to detailed finite element modelling for every wall configuration. A complementary Framing Lapping Zone (FLZ) approach was also developed to account for areas of concentrated framing and increased heat flow. The work contributed to the BRANZ/NASH research programme on CFS framing and energy efficiency in Aotearoa New Zealand.

The research also investigated whether the steel framing itself could be redesigned to reduce thermal bridging. Slitted-web studs, staggered framing and a new built-up thermal stud were assessed. The built-up system comprises two conventional lipped-channel sections arranged toe-to-toe and connected intermittently by batten plates, interrupting the direct conductive path through the wall.

Importantly, the concept was also structurally verified. Full-scale specimens were tested under axial compression and bending, followed by nonlinear finite element modelling validated against the experimental results. Parametric studies then examined the influence of stud geometry and batten-plate configuration on strength, stiffness, stability and buckling behaviour.

The research integrates thermal modelling, improved thermal prediction, structural testing and CFS member development, demonstrating that thermal bridging can be reduced partly through intelligent design of the steel framing itself.

SEISMIC PERFORMANCE AND SHEAR STRENGTHENING OF PRECAST HOLLOW-CORE FLOORS

Mohamed Mostafa (University of Auckland)

Advised by: Assoc. Prof. Rick Henry and Prof. Ken Elwood

The precast concrete hollow-core flooring system is widely used in Aotearoa New Zealand's existing building stock. These floor units are vulnerable to web cracking during earthquakes due to their lack of transverse shear reinforcement. The research presented in this thesis comprises three main research streams.

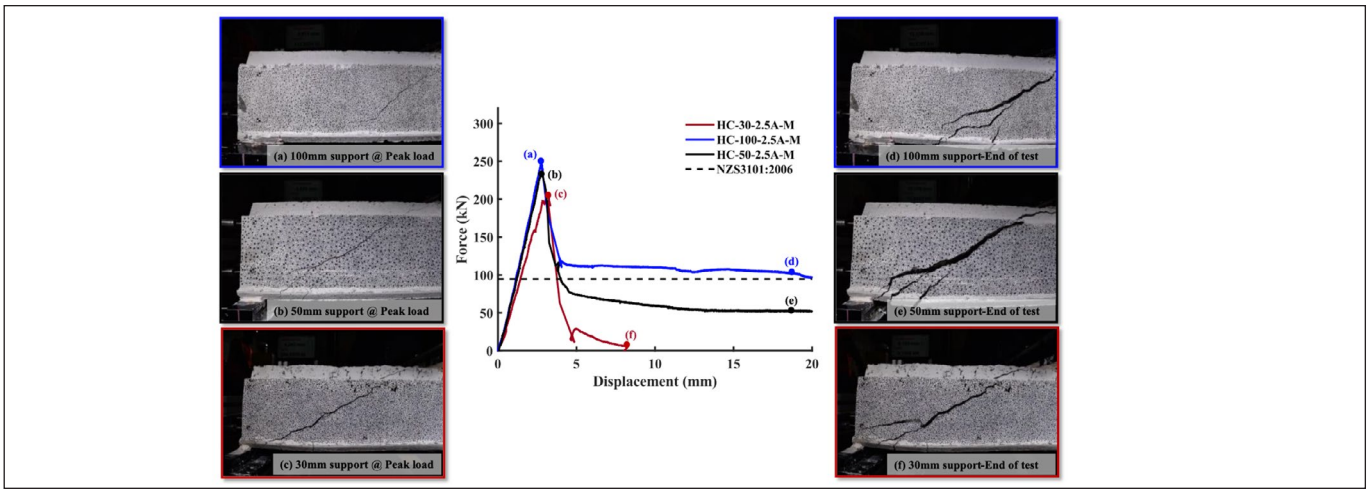
The first was a detailed damage investigation of an instrumented building incorporating precast hollow-core floors with modern detailing that experienced ULS spectral demands during the 2016 Kaikoura earthquake. The damage investigation revealed the deficiency of modern seating details specified in the current Concrete Structures Standard and showed that these units can sustain cracking in their unreinforced webs at inter-storey drift demands well below typical design drifts (e.g. 0.6% inter-storey drift).

Such observations raise concerns that existing buildings with precast hollow-core flooring may already have cracked webs due to prior earthquakes causing inter-story drifts that exceeded the 0.6% threshold and highlighted the need to understand if units with cracked webs still meet life safety requirements following an earthquake.

Cracking of the unreinforced webs of the hollow-core floor units is of particular concern as these floors were designed to rely on the uncracked section capacity to resist gravity shear demands. Therefore, the second research stream consisted of an experimental programme that examined the post-cracking residual shear capacity of twelve full-scale hollow-core units. Results showed that web cracking reduced load-bearing capacity by at least 60% of the units' ultimate strength, with instances where the units lost all design capacity upon the webs being cracked.

The findings from this experimental research emphasised the necessity for a practical and experimentally validated retrofit solution. For this reason, the third research stream evaluated the efficacy of post-installed anchors as transverse shear reinforcement for strengthening these floors, through testing six full-scale floor units.

The retrofitted units maintained a residual strength of approximately three times the design gravity demands, demonstrating the effectiveness of this approach as a retrofit and repair solution. The findings from this research advanced the understanding of the seismic performance of precast hollow-core floors and provided an experimentally validated retrofit method that can be used to strengthen existing hollow-core floors.



Example of some of the post-cracking measured responses compared to the calculated section capacity

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SUSTAINABLE USE OF EXCESS CONCRETE AND WASTE MATERIALS IN RETAINING STRUCTURES: PERFORMANCE, CARBON UPTAKE, AND LIFE CYCLE ASSESSMENT

Osama Mughrabi (University of Auckland)

Supervisors: Dr Enrique del Rey Castillo & Dr Charlotte Toma | *Funded by Callaghan Innovation / Envirocon Ltd*

PhD Research | Department of Civil and Environmental Engineering | The University of Auckland

BACKGROUND & MOTIVATION

Excess concrete waste — the leftover fresh concrete returned to batch plants after construction — is a significant challenge in Aotearoa New Zealand and globally. Up to 9% of total concrete production is returned unused, while cement manufacturing accounts for approximately 7% of global CO_2 emissions.

Envirocon Ltd, the industry partner of this research, has developed a business model that redirects excess concrete trucks to nearby collection hubs and casts the returned material into modular gravity retaining wall blocks. While this model effectively eliminates the disposal problem, the volume that can be productively upcycled remains constrained by the mix proportions of the returned concrete. This research investigates whether waste-derived alternatives — sourced from construction and demolition waste (CDW) streams with no established recycling pathway in Aotearoa New Zealand — can be systematically added to increase the productive volume and reduce the overall carbon footprint.

RESEARCH APPROACH

Unlike conventional supplementary cementitious material (SCM) research that replaces existing constituents, this study adds a new volume to the excess concrete — keeping the original mix composition intact, since it cannot be altered once batched. Each constituent in the added volume was replaced with a locally available waste material: cement with glass-based SCMs (window glass powder and glass wool powder, both CDW with no recycling path in Aotearoa New Zealand, with bottle glass as a literature benchmark), tap water with concrete slurry wash water collected from mixer cleaning, coarse aggregates with subbase recycled concrete aggregates (RCA), and fine aggregates with crushed window glass sand.

KEY FINDINGS

SCM Reactivity

Glass-based SCMs showed no pozzolanic reactivity at room temperature — behaving as inert fillers — but became reactive when cured at 40°C, with portlandite consumption of 47–69% and C-S-H formation exceeding the control. This implies that high-temperature reactivity tests such as the R3 test are unreliable for predicting real-world performance under ambient factory conditions. A novel slope-based TGA method was developed, defining phase boundaries by zero-slope points on the DTG curve rather than fixed literature temperature ranges, eliminating systematic error from phase shifting.

Structural Performance

Despite strength reductions attributed to the non-reactive SCMs acting as fillers at room temperature, both combined waste material mixes exceeded the minimum 28-day compressive strength required for demoulding, confirming their suitability for use in gravity retaining wall blocks. The 15% added volume was identified as the optimal addition ratio.

Carbon Uptake Quantification

A novel methodology combining wavelength-dispersive X-ray fluorescence (WDXRF) with atmospheric carbonation depth measurements was developed to quantify CO_2 uptake potential, quantifying the $CaCO_2$ content of fully carbonated binder paste samples and yielding a CO_2 sequestration capacity of 35% of binder weight (89.5 $kgCO_2/m^3$) for the control mix. Despite lower sequestration capacity, SCM and combined mixes achieved comparable or higher uptake than the control during service life, driven by deeper carbonation from increased porosity.

Cradle-to-Cradle LCA

The cradle-to-cradle LCA showed that using waste materials substantially reduced upstream production emissions (modules A1–A3) compared with conventional C20 concrete. Because the remaining footprint is so much smaller, later life cycle stages become proportionally more important: transport accounted for 35–37% of total emissions in the waste material mixes, against 11% for C20 concrete. The same effect applies to carbon uptake, which offset 21–23% of total emissions in the waste material mixes against 3.3% for C20 concrete. Most significantly, one year of atmospheric carbonation of the crushed blocks at end of life offset the entire cradle-to-gate emissions of the waste material mixes, unlike conventional concrete.

SIGNIFICANCE FOR PRACTICE

This research provides the first systematic evidence that excess concrete can be upcycled at scale in Aotearoa New Zealand using locally available CDW materials, with no need for virgin materials in the added volume. The findings support the existing business model and provide a pathway for expanding it. The work also demonstrates that establishing nearby collection hubs is the most impactful intervention for reducing the carbon footprint; that end-of-life carbonation should be included in LCA boundary conditions, as omitting it overstates the environmental impact; and that glass-based SCMs from CDW streams are a viable alternative to imported supplementary materials where industrial by-products are scarce.

SEISMIC RESILIENCE OF BUILDINGS IN NEPAL COMBINING TRADITIONAL PRACTICES AND MODERN ENGINEERING PRINCIPLES

Jitendra Bothara (University of Auckland)

Supervisors: Professor Jason Ingham and Dr Dmytro Dizhur

Department of Civil and Environmental Engineering | The University of Auckland

ABSTRACT

Earthquake fatalities in developing countries remain strongly associated with the collapse of vulnerable buildings, particularly low-strength masonry constructed using locally available stone, brick and earth. Wholesale replacement of traditional construction with modern reinforced concrete or engineered masonry is neither economically nor practically feasible in many communities. Traditional buildings remain affordable, accessible, thermally appropriate and culturally accepted, while locally sourced materials can also have lower environmental impacts. Conventional building codes and engineering practice, however, frequently give limited recognition to these construction systems. The focus of the PhD was therefore on how culturally acceptable and affordable traditional construction can be made safer, and why convergence of traditional knowledge and modern science is necessary for seismic safety.

The work drew on field observations of earthquake-resilient vernacular masonry along the Himalayan arc, numerical analysis and experimental testing. Traditional construction contains seismic-resilient features including structural integrity, redundancy, deformability and energy dissipation. Modern engineering principles were used to understand these features, identify weaknesses and develop practical improvements.

A major component focused on reconstruction of schools in remote mountainous areas of Nepal following the 2015 Gorkha earthquake. An attribute-based methodology was developed to select building systems considering seismic performance together with locally available materials and skills, accessibility, economy, sustainability and cultural compatibility. This led to building solutions that retained familiar materials and construction practices while introducing engineered strengthening measures.

Static and dynamic laboratory tests showed that wire-mesh containment substantially improved strength, deformation capacity and energy dissipation. Shaking-table tests on 2/3rd and 1/3rd scale stone masonry school building models showed that gabion mesh bands and containment could maintain box-type behaviour and structural integrity under severe excitation. A response modification factor of R = 3.5 with 10% modal damping was proposed for design of the strengthened system.

The findings show that traditional knowledge and modern engineering are not competing approaches. Seismic safety can be improved by understanding, validating and strengthening construction that communities can access, afford and build, rather than assuming that safety requires its replacement.



Figure 1. Experimental validation of a strengthened stone masonry school building system combining traditional construction with modern engineering.

IMPROVED FATIGUE-LIFE CHARACTERISATION VIA HIGH-THROUGHPUT FATIGUE TESTING

Amir Mousavi [1], Ahmed Bilal[1], Mo Balfour-Lochner[1], Grant West[2] and Derek Warner[2,1]

Fatigue is one of the most common failure mechanisms, where repeated loading can cause failure even when stresses remain well below the static strength of a material. Predicting fatigue failures is difficult because nominally identical specimens tested under the same conditions can fail after very different numbers of loading cycles. Fatigue failures must therefore be treated statistically, with particular attention to the lower tail of the fatigue-life distribution, where rare early failures are critical to design, reliability and maintenance decisions. Conventional fatigue datasets often contain too few results to directly characterise these low-probability failures, requiring substantial statistical extrapolation. Larger datasets can reduce this uncertainty, but conventional fatigue testing is slow and expensive because specimens are normally tested one at a time, often for millions of cycles. This motivates the development of high-throughput fatigue testing that can generate large datasets efficiently while maintaining controlled, mechanically meaningful loading and relevance to established standards such as ASTM E466 and ISO 1099.

To address this challenge, a stacked-cylinder fatigue testing method has been developed, as illustrated in Figure 1. Metallic ring specimens are placed around compliant polymer cylinders. Axial compression causes the cylinders to expand radially through Poisson's effect, while the rings restrain this expansion and develop tensile hoop stress. A locally reduced section is designed to provide a predominantly uniaxial fatigue test region comparable with a conventional specimen. Multiple ring–cylinder units can be stacked and loaded simultaneously using a single actuator, increasing throughput through parallel testing, without resorting to testing at artificially high frequencies or the use of miniaturised specimens. An initial proof-of-concept tested 39 aluminium 6061-T6511 rings to approximately two million cycles; 33 specimens failed and the resulting fatigue-life distribution showed promising agreement with comparable conventional fatigue data. The demonstration also confirmed that a predominantly uniaxial and uniform ring stress state and a mechanically stable multi-specimen stack were achievable.

The proof-of-concept also identified three important limitations that now guide development toward a standards-relevant fatigue methodology. First, the original reduced test section was only 8 mm^2, below the approximately 19.4 mm^2 minimum associated with the ASTM E466 geometry considered here. The ring has therefore been redesigned with a 20 mm^2 test section and standards-based geometry. Second, failure of neighbouring rings caused measurable changes in the loading of surviving specimens. Finite-element modelling and supporting experiments have therefore been used to quantify this interaction and select a 56 mm cylinder height that substantially reduces stress redistribution while retaining a compact stack. Third, the original proof-of-concept exhibited time-dependent evolution of ring loading due to changes in the polymer-cylinder response. To address this, a dedicated high-throughput fatigue testing machine has been developed at the University of Canterbury with extensive strain monitoring and ring-strain feedback, allowing the applied actuator load to adapt as the cylinder response evolves.

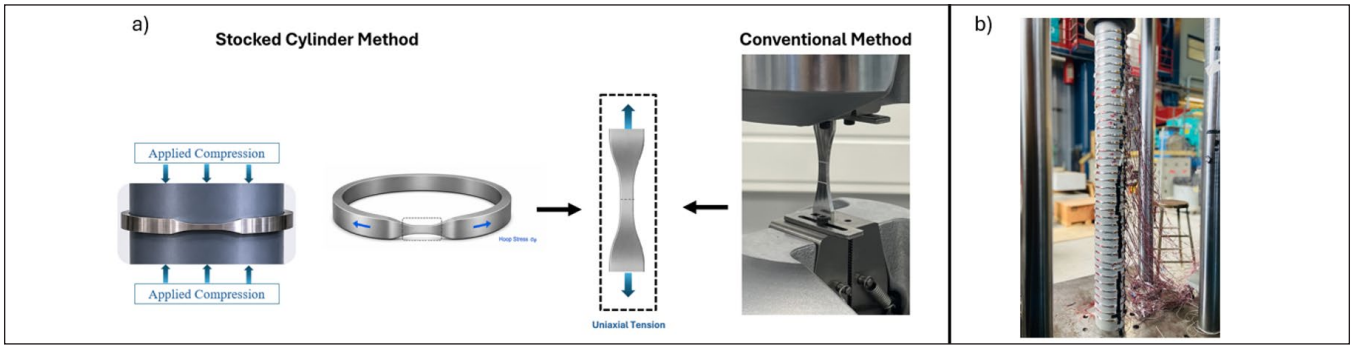


Figure 1. (a) operating concept compared with conventional uniaxial fatigue testing; (b) proof-of-concept stacked-cylinder test setup.

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2 Cornell Fracture Group, School of Civil and Environmental Engineering, Cornell University, Ithaca, NY, USA

MACRO-ELEMENT MODELLING OF REINFORCED CONCRETE STRUCTURES CONSIDERING PLASTIC HINGE ELONGATION AND SHEAR–AXIAL–FLEXURAL INTERACTION

Zakariya Waezi[1]

Supervisor: Rajesh Dhakal[2] *(University of Canterbury)*

The seismic performance of reinforced concrete (RC) structures is strongly influenced by the interaction between shear, axial, and flexural actions within plastic hinge regions. These interaction effects govern not only strength and stiffness degradation but also key response mechanisms such as plastic hinge elongation, depth growth, energy dissipation, and the transition from ductile flexural behaviour to brittle shear-dominated failure. Although modern seismic design philosophy relies on ductile plastic hinging to ensure satisfactory structural performance, most conventional lumped plasticity models neglect plastic hinge elongation and fail to capture shear–axial–flexural interaction (SAFI) [1]. Conversely, distributed plasticity and fibre-based models can represent several of these phenomena but are computationally expensive and often impractical for large-scale nonlinear dynamic analyses.

This research introduces WDHinge [2-4], a novel macro-element developed within the OpenSees framework to explicitly model plastic hinge elongation and SAFI while maintaining computational efficiency suitable for performance-based earthquake engineering applications. The proposed element extends the multi-spring hinge concept by incorporating mechanisms associated with longitudinal reinforcement, confined and unconfined concrete, diagonal concrete struts, and transverse reinforcement (Figure 1). Unlike previous lumped plasticity formulations, WDHinge directly accounts for the contribution of shear reinforcement deformation, enabling realistic simulation of elongation, shear deformation, and depth growth within plastic hinge regions.

The proposed element was verified against an extensive database of experimental studies, including cyclic cantilever beam tests, frame sub assemblage experiments, shaking-table tests, and full-frame structural systems. Comparisons demonstrated that WDHinge accurately predicts load–displacement hysteresis, plastic hinge elongation, shear deformation, and overall cyclic response.

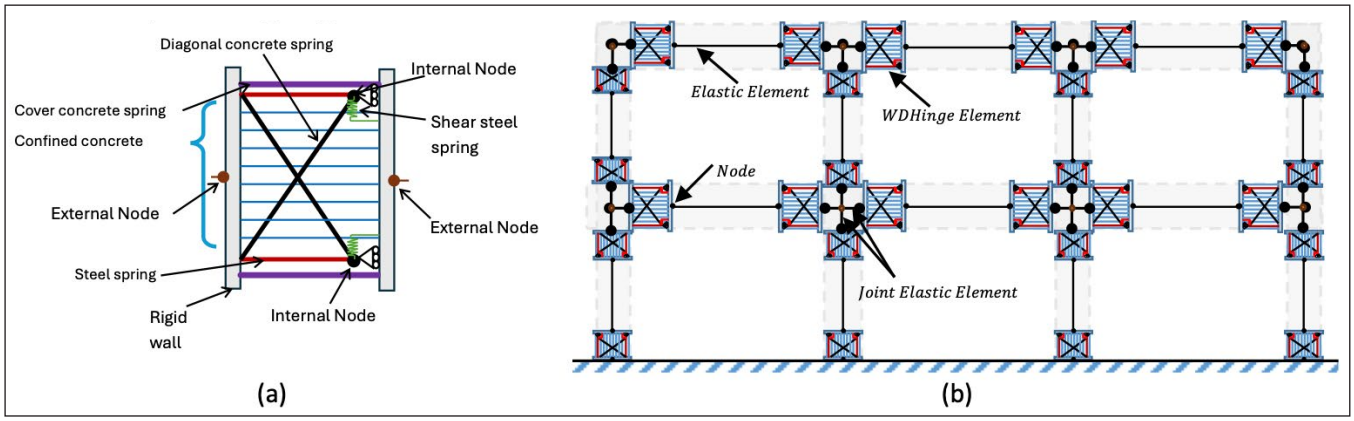


Figure 1. WDHinge macro-element: (a) element components and (b) application to a 2D RC frame model.

1 Structural and earthquake engineering researcher at the University of Canterbury
2 Head of the Civil Engineering Department at University of Canterbury

The results showed significant improvements over existing lumped plasticity models, particularly in cases where axial load effects, unequal reinforcement arrangements, and interaction-governed behaviour influenced the response (Figure 2). Accounting for the deformation of shear reinforcement enabled more realistic predictions of internal shear demands and member deformation mechanisms.

The macro-element was subsequently applied to nonlinear static and dynamic analyses of existing and modern RC buildings. Comparisons with conventional lumped plasticity models showed that neglecting plastic hinge elongation and SAFI can significantly underestimate internal force redistribution and structural deformation demands. Incremental dynamic analyses of Aotearoa New Zealand code-compliant RC moment frames demonstrated that interaction-controlled behaviour influences drift demand, collapse capacity and the distribution of damage throughout the structure. The research also identified important interactions between damping models and nonlinear deformation mechanisms, highlighting that inappropriate damping formulations may introduce artificial energy dissipation and unconservative seismic response predictions.

This research provides structural engineers with a practical analysis tool that bridges the gap between simplified lumped plasticity and computationally intensive distributed plasticity models. By explicitly accounting for plastic hinge elongation and SAFI, WDHinge enables more reliable prediction of earthquake damage, collapse mechanisms and seismic performance.

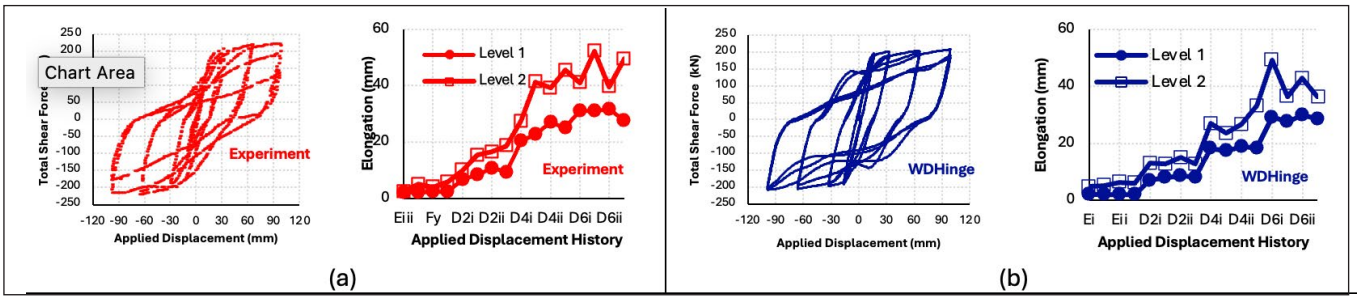


Figure 2. Results of 2D tested by Wuu [5] (a) experimental load displacement and elongation history; (b) numerical load-displacement and elongation history incorporating WDHinge;

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STOP-DRILLING AS AN IN SITU REPAIR FOR FATIGUE CRACKS

Hooman Pakdaman[1], Derek Warner[1] and K. T. Ramesh[2] (University of Canterbury)

ABSTRACT

Fatigue cracking is a common failure mode in structures subjected to cyclic loading. When a fatigue crack is detected, engineers need a practical repair method to limit further crack growth and reduce the risk of failure while minimising disruption to service. Among the available fatigue crack repair methods, stop-drilling is a simple, rapid and relatively inexpensive option.

In this method, a hole is drilled at the crack tip. From a mechanical perspective, the hole replaces the sharp tip with a rounded boundary, reducing the local stress and the force driving further crack growth. Conventional repair procedures, however, often involve taking the structure out of service, unloading it or, in some cases, disassembling a component. These preparatory steps can make an otherwise simple, local repair time-consuming and costly, providing an incentive to repair the structure while it remains under service load. This PhD research at the University of Canterbury examines whether maintaining the service load during stop-drilling changes the mechanical response of the repaired crack.

We compared drilling under a maintained tensile load with unloading before drilling followed by reloading. A two-dimensional elastic-plastic finite element model of an edge-cracked Al 6061-T6 plate was used under both plane stress and plane strain conditions. Before repair, the crack was advanced under cyclic loading to develop a wake of residual stress and plastic deformation. Hole radii ranged from 0.1 to 2 times the pre-repair monotonic plastic-zone radius. Both repair sequences were assessed at the same final applied load.

The two repair sequences produced essentially the same final crack opening response. Accumulated plastic work outside the drilled region differed by less than 1%. Differences in peak accumulated equivalent plastic strain ahead of the hole remained below 10% and depended on the constraint conditions, so neither sequence consistently produced the lower peak strain.

Hole radius had a much greater influence on peak plastic strain than the drilling sequence. Increasing the radius reduced the peak strain, but the reduction levelled off once the hole radius became comparable to the monotonic plastic-zone radius of the original crack. Increasing the radius beyond this characteristic size increased crack opening and accumulated plastic work.

Within the scope of the model, these findings support the mechanical plausibility of stop-drilling while a structure remains under load.

Related outputs: Presented at the 14th International Fatigue Congress (IFC14), Funchal, Portugal, 2026. A full manuscript has been submitted to Engineering Fracture Mechanics.

The full manuscript is under review at Engineering Fracture Mechanics and is available as a preprint: Pakdaman, H., Warner, D. H. and Ramesh, K. T. (2026). The mechanics of stop-drilling fatigue cracks under load. SSRN. https://ssrn.com/abstract=7440382

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CROSS AUS & UK

Please note that Cross UK has prepared a special edition Bridges Special Newsletter dated July 2026. Some of the topics include:

- Concrete spalling onto a public area from the soffit of a nationally important flyover
- Corrosion of partially grouted steel tendons in viaduct raises concern
- Concern over the quality of precast bridge beams
- Weld defects in steel girders for major infrastructure assets
- Incorrectly installed tension control bolts could have led to serious consequences
- Temporary stability risk due to construction changes on bridge deck
- Concern over the experience of a highway authority appointee
- Concerns about highway infrastructure due to maintenance funding shortfalls

ALTERATIONS TO SPINE WALL AND JOISTS RESULT IN STRUCTURAL ISSUES

(CROSS-UK 16 March 2026)

A Reporter shares details of structural damage in a listed building after alterations that involved moving a spine wall and notching timber joists.

The incident highlights the risks of modifying structural elements without thorough design checks and quality control.

Reporter's Submission

The Reporter, an experienced structural engineer, explains that they were asked to advise on structural movement in a multiple storey listed historic building that had been converted into residential apartments.

The building had a traditional arrangement with long-span continuous timber joists spanning from front to back at each floor and a central spine wall providing support to the floors and the wall above.

The issue arose after alterations were made to a first floor apartment. According to the Reporter, the architect's design involved moving the load-bearing spine wall by around 300mm and notching timber floor joists by approximately 75 mm to accommodate a soil pipe for a new bathroom (see Figure 1).

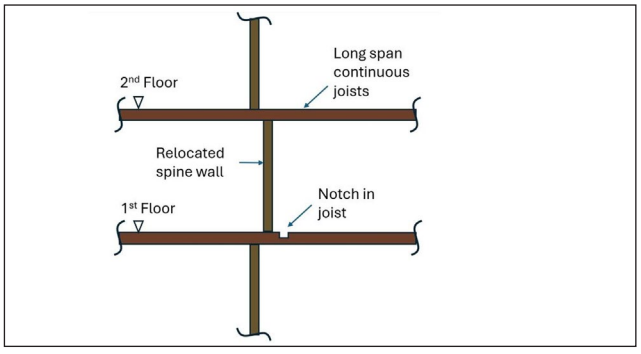


Figure 1: Indicative cross-section showing relocated spine wall and notched joist

The Reporter highlights that the project's structural engineer designed steel plates to reinforce the joists at the notches. However, the engineer did not carry out an overall check of the joists for the increased bending moments, nor did they calculate deflections or verify the adequacy of the screws connecting the plates to the joists.

The floor was opened up in several places, the Reporter continues, and it was found that the steel plates were missing at those locations (see Figure 2).



Figure 2: Soil pipe passing through notched and unreinforced floor joist

Following the works, the Reporter states the first and second floor joists deflected. This caused doors in the upper ground floor flat to jam and cracks to appear at the second, third, and fourth floor levels. The Reporter's calculations indicated that the joists could be overstressed locally at the notches by factors of up to two.

As far as the Reporter is aware, the situation remains unresolved.

Expert Panel Comments

The works described by the Reporter are major structural interventions to the building. Moving a loadbearing spine wall and notching continuous timber joists are significant changes that require thorough design, verification, and implementation, and the observed structural movement indicates clear shortcomings in these areas.

Relocating the spine wall alters load paths and increases both bending and shear demands on the joists. Capacity should be established not only at the notch but also at the supports and across the span. There should be an overall understanding of how the structure carries load to ground before substantial alterations proceed.

Where joists are notched, any strengthening must be properly designed and verified, including the fixings. Plates that do not have adequately designed screw connections, or that are not installed, provide no effective reinforcement. In aged timber, variability in knots, shakes, and twists demands conservative design and site checking, and any plates should extend well beyond the notch to be effective. Fixings within clearance holes will not carry load until the joist has deflected.

Primary structural interventions in existing buildings should be accompanied by regular inspections by the structural engineer. Deviations on site can compromise performance and the Panel stresses the importance of verifying that proposals are implemented as designed, although it notes the inadequacy of the original design in this case described by the Reporter.

Short term stability or minor movement is not a guarantee of long-term performance in timber. Allowable timber stresses over short durations can be much higher than long term values, so early indications of movement should be regarded as warning signs of potentially worse outcomes if left unresolved. Additional steelwork may be required to restore adequate capacity.

The design of alterations to existing buildings should be collaborative, with all parties working within the constraints of the structure. The Panel emphasises the need for suitably qualified and experienced structural engineers and contractors, and for the structural engineer to challenge proposals that do not align with the behaviour of the existing fabric.

A recurring theme of CROSS Safety Reports is the need for a clear understanding of the overall structural behaviour of a building, rather than focusing on isolated elements. The Panel notes there is lack of clarity over who is responsible for ensuring incremental renovation over time does not lead to unsafe outcomes.

TEMPORARY WORKS FAILED TO CONSIDER POSSIBILITY OF EXISTING REINFORCED CONCRETE (RC) SLAB AND BEAM FAILING

(CROSS-UK 14 May 2026)

The Reporter was asked to advise on the design of temporary works to allow the removal of a defective reinforced concrete column to be replaced by a new steel column. Whilst the proposed props were capable of safely taking the required forces, no evidence was found to show that the prop designer had checked the existing structure.

Reporter's Submission

A Reporter was asked to advise on the design of temporary works to allow the removal of a defective reinforced concrete (RC) column to be replaced by a new steel column. The column is at the base of a multi-storey block of flats and takes an estimated characteristic load of over 1,165kN and an estimated design load of approximately 1,590kN.

Upon arriving on site, the Reporter found that heavy duty props had already been installed and the prop designer had left on site a copy of their drawing which showed pairs of heavy duty props propping the beams and slab on either side of the column.

The status of the propping drawing was 'For Approval' and the drawing included the following note. 'The client must be assured that the scheme's technical details meet their requirements, that it is both safe and suitable for its intended use, and, if the system creates additional loads on an existing structure, that the structure can safely support those loads.'

Whilst the proposed props were capable of safely taking the required forces, no evidence was found to show that the temporary prop designer had checked the existing structure. The Reporter therefore asked the contractor to carry out opening up works to determine the dimensions of the propped beam, the thickness of the slab and the details of the reinforcement in the beam.

Once this had been done it was apparent that the existing beams and slabs were not strong enough to

transfer loads from the temporary props to the existing concrete column above and would fail in bending and shear under the applied loads.

The Reporter therefore designed a more complex system of temporary works and prepared a written method statement describing the procedure which would allow the column to be safely replaced.

The contractor could have removed the defective column without providing these additional temporary works in which case it is likely that a significant part of the building would have collapsed.

It is unacceptable, says the Reporter, for the propping designer to issue what is effectively a temporary works drawing without checking the existing structure irrespective of the caveats noted above. The propping designer should have advised the client at the outset to appoint an independent engineer who could not only design the props but also check the structure to take the applied forces and design reinforcement works to the structure are necessary.

Expert Panel Comments

This is a worrying report, and it looks to have been a failure of the temporary works procedure that should have been in place with a Temporary Works Coordinator (TWC) and checking engineer appointed on what sounds like a significant structure. The Expert Panel notes that when temporary works are introduced to support an existing structure, it is critical to ensure a viable load path is understood and verified from the point where loads are applied through to a suitable foundation.

Members highlight that designing individual elements in isolation - such as props - can lead to unsafe assumptions if the capacity of the surrounding structure has not been assessed. In the case described, the proposed propping arrangement appears to have considered the capacity of the props themselves but did not demonstrate that the existing beams and slab could safely transfer the additional loads. The concern here is that the column above continues to load the system and that the load now goes through the beams that were taking one floor of load into the column and now have to carry the shear from more floors.

CROSS has published similar reports, such as (CROSS Safety Report 1459 – Inadequate propping of an existing timber beam in a Victorian building), where failure of load redistribution was not considered in the temporary works design, so possibly this is not an uncommon misconception and reinforces the need for competence and adequate checking.

Management of temporary works procedure

Members also observe that the case illustrates a breakdown in the temporary works management process. Clear responsibilities are required so that the implications of temporary support arrangements are fully assessed. The Panel notes that it is not uncommon for designers to address only the specific task they have been asked to undertake; however, competent engineers should recognise when a brief is incomplete and advise accordingly. A coordinated approach involving the TWC and appropriate checking procedures is therefore essential.

The Panel further notes that refurbishment and repair projects are becoming more common as the industry seeks to extend the life of existing buildings. This increases the likelihood of complex temporary conditions arising during construction. The transition from permanent, through temporary, back to permanent can involve some challenging circumstances. Each stage must be carefully considered and planned. It is the responsibility of the TWC to ensure this happens, but also the responsibility of all designers to think about the overall load path, not just the local support.

CONCERNS WITH TEMPORARY BRACING OF MASONRY WALLS DURING CONSTRUCTION

(CROSS-UK 13 May 2026)

A Reporter is concerned about the continued use on sites of ad hoc timber bracing systems that support masonry walls without proper temporary works design, calculations, or oversight, leaving structures unstable and workers exposed to serious risk (Figure 1).



Figure 1: Completely unsuitable timber bracing to a masonry wall

Reporter's Submission

This safety issue is deeply important to the Reporter who personally suffered a life-changing injury as a result of a preventable incident on site when a masonry wall collapsed during construction.

Experiencing the consequences first-hand has driven the Reporter's commitment to improving safety standards, not for compliance alone but to protect lives, prevent others from suffering similar injuries, and to promote adequately designed, reliable temporary works across the industry.

Some makeshift systems are not fit for purpose, are often poorly fixed, and provide a false sense of security. Without a designed solution, there is no assurance that the structure can withstand loads, movement, or changing site conditions.

The underlying cause of some safety issues can be traced back to a systemic failure in communication, planning, and accountability around temporary works. Some Principal Contractors (PCs) are not effectively communicating with Temporary Works Coordinators (TWCs) and are sometimes failing to identify, manage, or request Temporary Works Designs (TWDs) within bricklayers' work packages. As a result, temporary works could be installed informally on site without design, approval, or control.

This lack of engagement leaves significant gaps in compliance with BS 5975-1:2024 (Temporary works – Management procedure for the control of temporary works, Code of Practice), the Construction (Design and Management) Regulations 2015 (CDM 2015), and the Health and Safety at Work Act 1974. Temporary works might be treated as a site "means and methods" issue rather than a designed and coordinated safety-critical activity.

This would expose workers to serious risk and leave PCs and duty holders open to enforcement action, prosecution, and uninsured losses. It can reflect a cultural issue within the industry where temporary works are underestimated, poorly understood, and not given the same level of scrutiny as permanent works, says the Reporter.

The key learning outcome is that temporary works must always be treated as safety-critical activities, not informal site practices. Ad hoc solutions, particularly within bricklaying and façade packages, consistently lead to uncontrolled risk when they are not identified, designed, and managed in accordance with BS 5975.

A good solution is earlier and stronger engagement between PCs, TWCs, designers, and trade contractors. Temporary works must be clearly defined within work packages, supported by engineered designs, calculations, and formal approval before installation. Defined responsibility, competence, and inspection regimes are essential, says the Reporter.

The Reporter goes on to say that the industry needs to recognise that unmanaged temporary works expose workers to life-changing harm and businesses to prosecution, invalid insurance, and reputational damage. By improving communication, enforcing compliance, and adopting professionally designed systems, the industry can prevent repeat incidents, protect lives, and raise safety standards.

Expert Panel Comments

This is a good report that highlights the often ad-hoc approach to temporary works design which particularly impacts low rise and masonry construction. The Expert Panel notes that the temporary stability of masonry walls during construction is a well recognised safety concern because partially completed walls can be vulnerable to overturning from relatively modest loads such as wind, impact, or construction activities. Collapses of masonry walls during construction have historically resulted in serious injuries and fatalities, and therefore the provision of adequate temporary support should be treated as an important construction safety issue.

Panel members observe that ad hoc or improvised bracing arrangements are sometimes used on construction sites without formal design, calculation, or review. While such measures may represent an attempt by site teams to address a recognised risk, they may not provide a reliable or consistent level of restraint. In some circumstances poorly conceived temporary works can introduce additional hazards because the behaviour of the system has not been assessed. The Panel therefore stresses that temporary works should be properly identified and managed in accordance with established procedures such as those set out in BS 5975.

The Panel also notes that masonry construction packages, particularly in low rise projects, can sometimes be less controlled than other structural activities. This may result in gaps in responsibility for both temporary and permanent stability. The report also highlights a wider issue of craft skills changing over time. The awareness of the importance of temporary support of masonry walls has fortunately been growing. The temporary restraint shown in the photo above is a response to a requirement but obviously not in a structured way. There is an obvious need for simple

guidance advising constructors and Temporary Works Coordinators of simple and effective solutions that are cost-efffective and reliable during construction.

Simple best-practice solutions for temporary bracing of masonry walls could assist contractors and TWCs in implementing safe and effective restraint systems without the need for bespoke design in every case. The Panel suggests that this may be an area where further guidance from organisations such as the Temporary Works forum could support improved industry practice.

CONCERN OVER POSSIBLE OVERLOADING OF CAR PARKING STRUCTURES

(CROSS_AUS 17 April 2026)

Reporter's Submission

This report raises concerns relating to the structural design of multi-storey car parking structures, i.e.:

- superimposed loads that should be used for structural design
- superimposed loads that should be used when considering behaviour in fire conditions

The reporter is concerned that the increase in vehicle weights in recent years may have resulted in significantly reduced factors of safety in car parks.

Definition of Light Vehicle Loading

The reporter is a forensic specialist who has seen many car park designs for apartment buildings and other public areas, and almost all have been designed to carry a superimposed load of 2.5kPa. Since 2002, AS1170.1 permits car parks to be designed for 2.5kPa provided they can only be accessed by 'light vehicles', the gross vehicle mass of which does not exceed 2500kg.

In general, there are three different masses published for motor vehicles:

- **Tare mass** – mass of unoccupied and unladen vehicle (with only 10 litres of fuel and no accessories)
- **Kerb mass** - as above but with a full tank of fuel
- **Gross vehicle mass (GVM)** - Maximum allowable mass of a vehicle when fully loaded.

The obvious conundrum for car park designers is how to ensure that the GVM of any vehicle entering a car park does not exceed 2500kg. There appears to be a widely accepted understanding that this is achieved by ensuring the maximum height of any vehicle is limited to 2.2m. This maximum height limit is also equal to the minimum clear headroom specified in AS/NZS 2890.1 (Parking facilities Part 1: Off-street car parking) and the National Construction Code (NCC).

However, a recent search of available data by the reporter suggests that 40% of new cars being sold in Australia **exceed** the GVM of 2500kg and appears to contradict the correlation of height limit with 2.5kPa design load. The reporter has summarised this in the following table (note that table has been replaced by Editor) showing the GVM of the most popular vehicles sold in Australia.

Table 1: GVM of popular vehicles in Australia

Rank	Vehicle	Vehicle Type	Approximate GVM (kg)
1	Ford Ranger	Utility / Pickup	~3,230–3,350 kg (varies by variant)
2	Toyota RAV4	Medium SUV	~2,100–2,200 kg
3	Toyota HiLux	Utility / Pickup	~2,950–3,050 kg
4	Isuzu D-Max	Utility / Pickup	~3,000–3,100 kg
5	Ford Everest	Large SUV	~3,100–3,150 kg
6	Toyota LandCruiser Prado	Large SUV	~2,990–3,150 kg
7	Hyundai Kona	Small SUV	~1,850–1,980 kg
8	Mazda CX-5	Medium SUV	~2,020–2,140 kg
9	Mitsubishi Outlander	Medium SUV	~2,100–2,400 kg (higher for plug-in hybrid)
10	Tesla Model Y	Medium SUV	~2,405 kg

The ratio of tare mass to GVM is around 0.75, so it appears that in 2002 the revised standard effectively changed the definition of a 'light vehicle' with a 25% reduction and ended the relationship between vehicle height limit of 2.2m and the 'light' classification defined in the standard.

It is worth considering how the average kerb weight of vehicles has changed in Australia from 2002 to 2025. Preliminary research by the reporter suggests a 50% increase in weight over that period which (if true) is quite remarkable. However, AS/NZS1170.1 appears to account for this by its definition of a light vehicle, and it could be argued that a well informed designer should have been ignoring the 2.5kPa design load for at least the last 10 years. Monitoring of vehicle weights would have highlighted that the definition in the standard had become outdated if vehicle height was being used as a guide for weight.

In summary, the reporter considers that the standard should reconsider the definition of a light vehicle to being more representative of the heaviest motor vehicles commonly available under 2.2m in height. The reporter also believes that it should be considered whether the design loads on car parks should be increased to reflect the increasing weight of cars over the last 20 years or so.

In the meantime, the reporter notes that to comply with AS1170.1 designers are given two options in table 3.1 of the Standard, viz. F Light vehicle traffic (not exceeding 2500kg GVM) or G Medium vehicle traffic (exceeding 2500kg and not exceeding 10,000kg GVM). Thus, for vehicles exceeding the 2500kg GVM limit, this requires designers to use a design load of 5kPa.

Superimposed loads to be combined with fire in multilevel car parks

Another related point is that AS/NZS1170.0 indicates that the load to be considered in a fire scenario (clause 4.2.4) is G+ψlQ. In this equation ψl is defined as the "factor for determining quasi-permanent values (long term) of actions". This value is 0.4 for a car park, which, according to the commentary, is meant to represent an "arbitrary point-in-time value" to be considered with the fire, which is itself an 'extreme action'.

Many inner-city car parks are either 98% full (during the day) or completely empty (at night) and the reverse is often true of apartment car parks. One car park reviewed by the reporter is a hospital car park which has numerous floors dedicated to staff and this is 98% full 24 hours per day.

Whilst the long-term coefficient is suitable when considering, say, concrete creep in an inner-city car park, it may be much less suitable when considering fire in the same situation. If there is a fire in an inner-city car park during the day, all bays are very likely to be full. At night there may be no cars and therefore much less fuel to burn in the unlikely event of a fire. There will be transition periods each morning and night, but it seems less likely that the 'design event' will occur during a transition period. A further notable point about the morning transition is that the car park will often fill from the bottom upwards so the loading on beams or slabs in the lower floors is likely to be either full or empty.

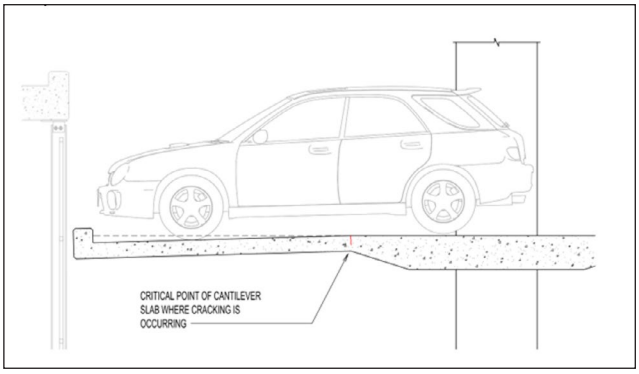


Figure 1: Car park cantilever slab

A specific case examined by the reporter was a hospital car park with four vehicles parked between narrow columns loading a cantilevered slab. The cantilever was the critical element being examined. AS1170.1 suggests the design load for fire should be 1kPa (0.4 x 2.5kPa) but 4 'average weight' modern vehicles parked nose-in represent at least 1.5kPa (see diagram below). The application of the standard in these circumstances is clearly a significant underestimate of the potential actual load.

The reporter suggests that car parks not be designed for 2.5kPa as the standard only permits this in circumstances where the GVM can be limited to 2500kg. The reporter knows of no circumstances where this can be assured without weighing each vehicle prior to entry.

In addition, the reporter suggests the use of the "long-term serviceability coefficient in fire" situation is not reflective of the loading likely to be present, especially on cantilevered slabs supporting parked vehicles.

Expert Panel Comments

The reporter has raised an issue driven by the reported increase of the Gross Vehicle Masses (GVMs) of "light" vehicles over recent decades. In Australia utility vehicles (utes) accounted for over 20% of new car sales in 2025, and the GVM of many of these exceeds 3200kg. The Reporter is concerned that the design load of 2.5kPa as nominated in AS/NZS1170.1, and which is based on a maximum vehicle mass of 2500kg, is no longer adequate.

This is not just an Australasian issue. CROSS Report ID 1257 Overloading of older multi-storey and underground car parks published in May 2024 addresses similar perceived problems in the UK. Further, the Institution of Structural Engineers Car Park Design published in June 2023 "reflects the significant changes to vehicular size, weight, and manoeuvrability in recent years to offer modern guidance on parking bay sizes and deck loadings" and recommends an increased design load from 2.5kPa to 3.0kPa.

Designers are reminded that it is their duty to address the risk of exposure to any hazard appropriately. This is a core requirement of WHS legislation that sits above Standards and Codes of Practice. If the requirements of a current local Standard are identified as lacking, the designer and their employer may wish to use their engineering judgement to make their own assessments of loading, noting that, in the case of design loads, AS/NZS1170.1 only gives minimum values.

Alternatively, designers should consider which other International Standard, specialist literature, statistical evaluation, testing, improved design or construction, in-service use or other means would be most appropriate to address the shortcoming. These should be communicated to and agreed in writing with the client. Best practice would be to ensure it is reflected in the relevant agreement.

As the reporter notes, compliance with AS1170.1 would require an increase from 2.5kPa to 5kPa for vehicles exceeding the 2500kg GVM limit. This is considered to be significantly conservative, and will lead to over-designed standard car park decks. The industry needs to consider embodied carbon implications in its design decisions, especially when it comes to code revisions, and therefore any increases should be made so as to avoid using excess material that also affects the commercial viability of projects.

In assessing any increase in design load relative to code values for a particular structure, designers will have to consider many factors, including, but not limited to:

- the probable distribution of vehicle types and masses
- the probability of full GVM being achieved across large volumes of vehicles on a suspended level or part thereof, noting that the difference in kerb weight and GVM can be in the order of a 40% to 50% increase
- sizes of parking bays
- distribution of vehicles and vehicle types in aisles (generally not applicable to cantilevers)
- design load safety factors for live loading

The effect on the impact resistance of walls and barriers at the ends of aisles and the bottom of down-ramps, which is based on a 1500 kg vehicle travelling at 2 m/s (7 kph), should also be considered.

The reporter's additional argument in support of a reconsideration of the live load reduction factor for fire in car parks appears reasonable, particularly given the observation that some car parks (for example, hospitals) can have 24 hour occupancy at close to 100%.

The increased fire load from the growing number of electric vehicles (and significant difficulty in putting out fires in lithium batteries) appears to be of additional concern. Designers should take the expected occupancy rates and distribution of such rates into account in choosing to adopt the code value, or a greater value of up to 1.0 as appropriate.

These matters should be drawn to the attention of the loading committee for AS 1170.1, noting any legacy issues that an upgrade to design loadings may create with regulators.

IStructE 2026 Structural Awards

The 2026 Structural Awards shortlist is announced.

The judging panel considered these key attributes when making their selection:

Planet (environmental impact),

People (social value),

Process (technical innovation)

Profession (professional advocacy).

Winners will be revealed on 6 November

Special Course

Dept of Civil and Environmental Engineering

ENEQ640*: Displacement-based Design of Low Damage Buildings



Alternative design philosophies and solutions for the seismic design of low-damage structures. Paradigm shift in performance-based seismic design: from Life Safety to Damage Control. Targeting safety, functional recovery and loss reduction. Introduction to PRESSS (concrete) and Pres-Lam (Timber) Technology and the hybrid (rocking-dissipative) system concept. Introduction to and application of Displacement Based Design (DBD). Simplified modelling techniques using lumped plasticity approach. Connection between floor-diaphragm and lateral resisting systems. Low-damage solutions for non-structural elements. Examples of on-site applications worldwide in low-, medium-, or high-seismic areas. Constructability aspects, sequence and detailing. Retrofit strategies using low-damage concept and technology.

Schedule

Course Code	Course Name	Duration	Time
ENEQ640	Displacement-based Design of Low Damage Buildings	2 Nov 2026 – 26 Nov 2026	Mon, Tue and Thu 2,3,5,9,10,12 Nov Mon, Tue 16,17 Nov 15.00 – 18.00
			Exam 26 Nov

Course Instructor



Adjunct Professor

Stefano Pampanin

Stefano Pampanin is Full Professor of Structural Engineering at Sapienza University of Rome and Adjunct Professor at University of Canterbury in Christchurch. He has been Professor of Structural Design and Earthquake Engineering and Chair of the Structural and Geotechnical Cluster at UC, where he joined in 2002 for 16 years.

He has been President of NZSEE (2012-2014). Fellow of IPENZ and NZSEE, Laurea (UniPavia), Masters (UCSD), PhD (Politecnico Milano). Fulbright Visiting Scholar (UCSD). In the past 30 years, he has dedicated a significant effort in R&D, actual implementation and dissemination of low-damage solutions in concrete and timber, as well as seismic assessment and retrofit strategies for existing buildings

**This course can be credited towards a Master of Civil Engineering (MCivilEng) With an endorsement in Structural Engineering. Contact Dr Giuseppe Loporcaro for any enquiries: Giuseppe.Loporcaro@canterbury.ac.nz*

STANDARDS UPDATE

The table below provides an overview of activities relating to updates of structural Standards. Two Standards that were out for public consultation at the time of writing are NZS 3404 and NZS 3604. These consultations have also been promoted to SESOC members to encourage them to provide feedback.

Standard	SESOC Rep	Update/Comments
Structural design actions - AS/NZS 1170.0 General principles; & AS/NZS1170.1 Permanent, imposed and other actions	Michael Robinson	A proposal has been developed to revise AS/NZS 1170.0, and consideration is also being given to develop a proposal to revise AS/NZS 1170.1. SESOC, via Engineering NZ, are represented on the committee by Michael Robinson.
AS/NZS 1170.2 Wind Actions		Amendment 2 of AS/NZS 1170.2:2021 was published in June 2024. This is still to be referenced in the Aotearoa New Zealand building Code System.
NZS 1170.5/ TS1170.5 Earthquake Actions	Tim Sullivan	TS 1170.5:2025 was published in November 2025. NZS 1170.5:2004 remains the referenced standard under the New Zealand Building Code system. TS 1170.5 is an alternative solution for those wanting designs to incorporate the latest hazard data and modern engineering principles.
NZS 3101 Concrete Structures		The building regulator is looking to establish a Concrete Technical Coordination Group (TCG) that will have oversight of the development and maintenance of concrete industry Standards and recommend which Standards should be prioritized for review. This is expected to include NZS 3101.
NZS 3404 Steel Structures	Andy Thompson	The NZS 3404 public comment draft was released on the 29 July 2026. The public consultation will close on 22 September 2026.
NZS 3404.3 Steel structures - Durability requirements		NZS 3404.3 was published at the end of July 2026. This document supersedes SNZ TS 3404:018.
AS/NZS 2327 Composite Structures		The proposed revision to AS/NZS 2327 that was consulted on from November 2025 to January 2026 has been finalized – at the time of writing it is still to be published
AS NZS 1720.1 Timber Structures	Jaimie Whitehead	Jaimie has been nominated to be the SESOC representative on the Standards Australia TM-010 Timber Structures and Framing Committee. It is understood the TM-010 Committee is underway with developing a joint AS/NZS 1720.1 Timber Structures design standard. This is expected to supersede the NZS AS 1720.1. NZS 3603:1993 remains the referenced standard in the building code system.
NZS 3604 Timber-framed buildings	Graeme Beattie	The PC Draft was released at the end of June 2026. The period to provide feedback was extended to 8 September 2026.
NZS 4229 Concrete masonry buildings not requiring specific engineering design		Concrete NZ, are looking to initiate a revision of NZS 4229. A project proposal to confirm the scope of the revision has been submitted to Standards NZ and a response is awaited.

SESOC Emerging Structural Engineers (ESE) Group

Chair: Henry Rowden

Contact: ese-chair@sesoc.org.nz

RECENT NEWS / UPDATE

- We currently have 747 members subscribed to the SESOC ESE mailing list.
- We have a new public LinkedIn page – search 'SESOC Emerging Structural Engineers' on LinkedIn and follow the page to stay up to date with our latest events and content.
- Lily Danby from Silvester Clark has recently joined the Wellington ESE Chapter Committee.
- We are currently looking for ESE Committee representatives in Nelson and Otago/Queenstown. If you are interested, please contact Henry Rowden at ese-chair@sesoc.org.nz.

RECENT PRESENTATIONS / ACTIVITIES

Poneke Wellington Central Library Site Visit - April 2026

Our SESOC ESE Pōneke chapter recently conducted a site visit to get an exclusive look inside the newly opened Wellington Central Library.

After closing in 2019 following the 2016 Kaikōura earthquake, it has since been strengthened and reopened as Te Matapihi ki te Ao Nui, "the window to the wider world", representing a major transformation into a strengthened, reimagined space for Wellington. And it's safe to say Wellington got a real treasure back.

The Aurecon project team walked the group through the design journey, sharing the challenges they faced and how they overcame them. One standout learning was the value of early contractor involvement, which was a great reminder of how good collaboration right from the start can make or break a project.

Lightning Talks/Networking Event, Otautahi Christchurch – July 2026

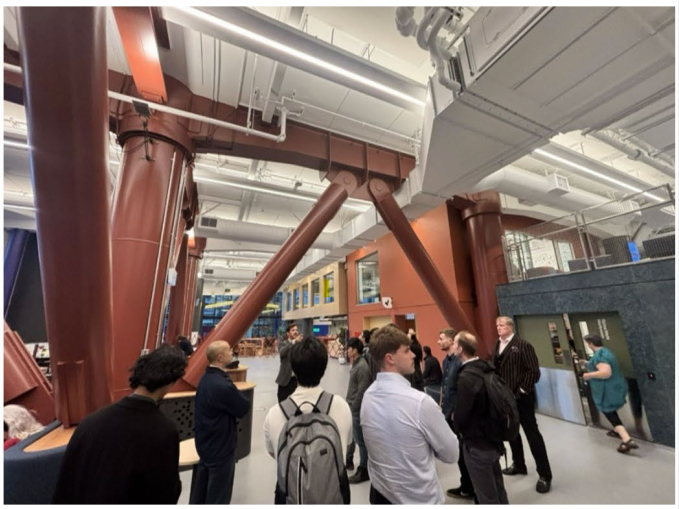
The Christchurch Chapter of the SESOC ESE Committee recently held a Lightning Talks/Networking event at the WSP office in Christchurch. The talented speakers gave some special insights and advice, drawing from their career experience and projects thus far.

We were thrilled to hear from:

- Kavindith Pulahinge (WSP) – Improving Design Efficiency Through Better Tool Integration.
- Ryan Bosselman (Beca) – Designing for Vibration on Puahuru Bridge.
- Nick Williams (WSP) – O'Sullivan's Bridge – Assessment and Strengthening of a Callender-Hamilton Truss Bridge.

Thank you to each of the above speakers for sharing their experience, insights and energy.





NEWS FROM THE REGIONAL STRUCTURAL GROUPS

Tāmaki Makaurau Auckland Structural Group

ASG SESOC Representative: Henry Rowden
Contact: henry.rowden@holmesgroup.com

LOCAL UPDATE

- We currently have 675 people on our mailing list with a 55% average open rate.
- ASG events hosted in 2026 so far: 6 technical presentations & 1 site visit.
- If you have any events or projects you would like to showcase to fellow SESOC members in the Auckland region, please contact any ASG committee member via the details on the ASG website.

RECENT PRESENTATIONS

- **16th June 2026** – ***Various building responses recorded during the 2011 Tohoku Earthquake*** – by Dr Kazuhiko Kasai, Professor Emeritus, Institute of Science Tokyo.
- **9th July 2026** – ***Site visit to National Storage Facility construction site in Mount Albert, Auckland***, hosted by Steel & Tube.
- **27th July 2026 –** ***Large-Scale Experimental Validation of an Innovative Seismic Isolation System for Modular Mass Timber Buildings*** – by Dr Ashkan Hashemi, Senior Lecturer, University of Auckland.
- **11th August 2026 –** ***Study of Seismic Performance of Reinforced Concrete Frame Structures*** – by Dr Xiang-Lin Gu, Distinguished Professor, Tongji University.
- **25th August 2026 -** ***Research and Developments in Structural Steel Systems: Low-Damage Friction-Sliding Systems and Welded and Bolted Connections*** – by Dr Shahab Ramhormozian, Associate Professor, Auckland University of Technology.



Presentation on Study of Seismic Performance of Reinforced Concrete Frame Structures – by Dr Xiang-Lin Gu, Distinguished Professor, Tongji University



Site visit to National Storage Facility

Te Matau-a-Maui Hawkes Bay Structural Group

Chair: Aaron Kaijser
Contact: akaijser@kotahistudio.co.nz (Kotahi Engineering Studio)

LOCAL UPDATE - 30/8/2026

- We are trying to provide events that are in demand and build awareness of events by local companies so they are well-attended. Without buy-in there is a risk of events not being available in the region..
- We welcome opportunities for webinars and hosting recorded presentations. Frequency of and speakers is less in the regions so if there is anything that may be of interest, please let me know.

RECENT EVENTS

June 23rd – Site Visit: Heretaunga House





A joint SESOC/TDS event, design engineers Dunning Thornton (Chris Speed) and contractor LT McGuinness led a site visit of Heretaunga House - a four level 5000m2 mass timber building in Hastings.
The project has been developed on the site of the old council offices and is shallowly founded on reinforced concrete ground beams. The hybrid steel and mass timber superstructure uses each material to its greatest strengths. Steel EBF frames expertly fabricated by Red Steel (Napier) provide proven seismic resilience and repairability in this high seismic zone. CLT floor panels manufactured by Red Stag (Rotorua) are light weight and fast to erect, while limiting wet trades. The LVL gravity post and beam frame updates historic woodwork-ing traditions with the speed and capabilities of CNC manufacturing by Timberspan (Hastings). A stand-alone utility building comprising of precast units by Lattey's (Hastings) compliments the hybrid structure by provid-ing a hub for plant and 'wet' utility spaces, this limits this moisture, acoustic and weight risks that can often drive costs into timber construction.
Well received by the 15 people in attendance.

August 14th and 21st – Site Visit: Hawkes Bay Prison ACP project



A joint SESOC/EngNZ event, hosted by Naylor Love and RBG, held on consecutive Friday's. The Accelerated Capacity Project (ACP) is a Department of Corrections initiative to rapidly increase high-security prison ca-pacity at Hawke's Bay Regional Prison through the delivery of two new 80-cell High Security Units, associated support facilities, exercise yards, and extensive security infrastructure. The project is being delivered under an accelerated Design & Build mode. Event attendence capped at 15 people for each event, with 10 and 15 people attending, respectively.

UPCOMING EVENTS

September 8th – Seminar Series: Residential Design

First of the nationwide series. Currently 18 people have signed up with some active encouragement/ persua-sion occurring in the background to ensure our sole practitioners, in particular, are well represented for this particularly relevant topic.

September 8th – Seminar Series: Residential Design

November TBC – Site Visit: Napier Civic Buildings

Waitaha Canterbury Structural Group

Chair: Matt Stewart
Contact: matt@h3structural.co.nz

LOCAL UPDATE - 10/09/2026

Canterbury Structural Group continues to deliver well-attended technical and networking events for the local structural engineering community.

The CSG mailing list remains strong, with over 500 members subscribed. The mailing list continues to be the primary way CSG notifies members of upcoming technical presentations, seminars, site visits, and collaboration opportunities. Site visits remain in high demand and are generally limited by site capacity, with recent tours requiring ticketed registration and booking out quickly.

2026 RECAP OF CSG EVENTS:

- **(1) Technical presentation:**
  - CCC on Construction Monitoring – 70 attendees.
- **(4) Site tours:**
  - Forté Health Hospital Expansion – 25 attendees;
  - Adult Acute Facility at Hillmorton Hospital – 15 attendees;
  - 79 Hereford Street seismic strengthening and retrofit – 35 attendees;
  - South Library & Customer Services – 30 attendees.
- **(1) CSG event:**
  - CSG 50 Year Anniversary Celebration at the Arts Centre Great Hall – 150 tickets sold;
- **(1) SESOC seminar:**
  - Steel Portal Frames.

.RECENT EVENTS

6 March 2026 – Site Tour of South Library and Customer Services in Beckenham

Chris Hurst (Quoin), Paul Anselmi (Ignite), and Rob Armstrong (Cook Brothers) led CSG on a site tour of Ōmōkihi, the new South Library and Customer Services in Beckenham. The site was formerly the earthquake-damaged South Library. The new design retained the ground-floor slab and foundations, with the new foundation and superstructure constructed over the existing foundation footprint.

The superstructure is a single-storey Importance Level 3 building with an elective SLS2 performance threshold adopted to improve operational continuity in moderate earthquake events. The structural system combines steel moment frames and steel braced frames. Hearing the architectural drivers, structural challenges, and builder input made for an interesting site tour and a valuable learning opportunity for attendees.





20 March 2026 – Site Tour of 79 Hereford Street

New life to a "Dirty Thirty" – CSG visited 79 Hereford Street, Otautahi Christchurch, with the site tour led by Chris Hurst and Brad Lindbom (Quoin), together with Shane LeCompte (Developer). The building is an early 1970's six-storey office structure over a basement, with a

structural system comprising in situ concrete and concrete masonry walls, and in situ concrete floors spanning over precast ribbed slabs and concrete beams.

The project involved seismic strengthening to 80% NBS (IL2) and the addition of a new lightweight timber and steel framed Level 7. Strengthening measures included basement raft improvements, in situ skin walls, new concrete shear walls, eccentrically braced frames with reinforced concrete links, tension-only X brace frames, diaphragm enhancements, FRP and steel plating to selected columns, and targeted beam shear strengthening associated with the new lateral systems.





28 April 2026 – Site Tour of Adult Acute Facility at Hillmorton Hospital

Chris Ravn (Naylor Love), Hamish McCormick (Beca), and Sam Paterson (Klein) led CSG on a site tour of the Adult Acute Facility at Hillmorton Hospital in Christchurch. The facility is part of the relocation of mental health services from Princess Margaret's Hospital to Hillmorton Hospital.

The new 9,850 m^2 facility includes single-storey timber-framed patient accommodation, communal and treatment areas, and a two-storey office/administration building constructed with steel framing, a ComFlor floor system, and eccentrically braced frames. A key highlight was the use of timber construction and off-site manufacturing, including LVL roof trusses and HyJoist floor cassettes, which created generous space for services within the attic zone. A major takeaway was the importance of early contractor involvement in off-site manufacturing.





4 June 2026 – Christchurch City Council Construction Monitoring Presentation

Christchurch City Council presented their interpretation of construction monitoring requirements to meet Building Act obligations, detailed their expectations for site reports, and answered questions from members. The focus was on residential construction.

A key learning was that any variation to the approved building consent documentation, no matter how minor, should be clearly noted in the site report with a note that the “builder/project manager is to obtain owner approval and notify CCC”. This generated a lively discussion from the audience, including questions about how practical this would be for minor on-site clashes and constructability items.



Building Consent officials from Waimakariri District Council and Selwyn District Council were also in attendance. A future presentation from CCC on construction monitoring requirements for commercial projects was requested.

19 June 2026 – Canterbury Structural Group 50 Year Anniversary Celebration

CSG celebrated its 50-year anniversary with an evening event at the Arts Centre Great Hall. The event was an opportunity to reflect on the people, projects, and progress that have shaped structural engineering in Canterbury over the past five decades, and to recognise the strength of the local professional community.

The event summary highlighted the University of Canterbury's structural engineering legacy, major changes in structural engineering over the past five decades, the evolution of engineering education and the young engineer experience, engineering developments in Christchurch through the 1970's, 80's and 90's, and reflections on the Canterbury Earthquake Sequence. Holding the event at the Arts Centre Great Hall was fitting given its cultural and architectural significance, and its own story of earthquake strengthening, restoration, and resilience.

Thank you to SESOC and CEAS for their support, to the presenters and session chairs, and to John Snook for contributing to a very successful anniversary event. It was a meaningful night for many.





13 August 2026 – Site Tour of Forté Health Hospital Expansion

CSG visited the Forté Health Hospital expansion on Kilmore Street. The project enlarges the hospital by approximately 6,000 m², increasing operating theatre capacity from five to nine theatres and inpatient capacity from 22 to 61 beds. It is designed as an Importance Level 4 healthcare facility, supporting continued operation after major seismic events, and is targeting a 5 Green Star sustainability rating.

A technical highlight was the building's resilient seismic design. The structure uses a dual lateral load-resisting system with Tectonus DMAX Braces working in parallel with steel moment-resisting frames. This provides enhanced damping, redundancy, and low-damage performance during earthquakes. The DMAX braces dissipate seismic energy through controlled friction. The high damping and low overstrength of this system resulted in lower base shear and capacity design forces compared to conventional yielding brace systems.

Thank you to Naylor Love and Lewis Bradford Consulting Engineers for hosting the group and sharing insights into the design and construction challenges of this significant healthcare project.





UPCOMING PRESENTATIONS

- Canterbury Museum Site Visit (tentatively 22 October)
- SESOC Seminar – Residential Design (4 November)
- University of Canterbury – Light Timber Frame Bracing Wall Testing (November/December)

Ōtepoti, Otago Structural Group

Report for SESOC Management Committee Meeting – August 2026

Prepared by Simon Burrough

LOCAL UPDATE

- The QuakeCore annual meeting was held in Ōtepoti Dunedin end of August 2026. the w/c 31st August. Lots of industry leaders in town for the week and a meeting with the local group was held.

RECENT EVENTS

- **EPB changes discussion held 29th July 2026** . A period of uncertainty hangs over the Dunedin market with the upcoming EPB changes and Coastal Otago zoning shifting to a medium seismic zone. The group met to discuss how the market is reacting (many clients are waiting until the changes are confirmed, some market-led strengthening still occurring though). Peter Brookland from the DCC gave an update on how the council is preparing for the changes and their approach in the interim (no new buildings being identified currently but those in the system are progressing). The EPB methodology is eagerly anticipated.

- **OSG meeting – 31st August 2026.** Ken Elwood and Paul Campbell were in town for the QuakeCoRE annual meeting and talked to the group about a range of topics. Paul gave an update on BRiDGE, projects and possible CPEng class changes. There was more discussion on the EPB changes and the recent 3-storey residential design guidance.

BRAIN TEASER ANSWERS

1. WHERE SHOULD THE SUPPORTS GO?

Place each support approximately **2.071 m from its nearest end**, giving a distance between supports of approximately **5.858 m.**

Let the total beam length be (L), the overhang at each end be (b), distance between supports (a), and the uniformly distributed load be (w).

The hogging moment at each support is:

L = total beam length

b = overhang at each end

w = uniformly distributed load per unit length

Support moment = -w*b^2/2/

Centre moment = w*L*(L - 4*b)/8

At the optimum:

w*b2/2 = w*L*(L - 4*b)/8

b = L*(√2 - 1)/2

= 0.2071*L

For L = 10 m:

Overhang at each end = 2.071 m

Distance between supports = 5.858 m

The optimum occurs when the central sagging moment equals the magnitude of the hogging moment at the supports.

2. DOES THE HOLE SHRINK?

The hole becomes larger.

Imagine filling the hole with a snug steel disc made from the same material. When heated uniformly, the disc and plate would expand together. Removing the imaginary disc leaves a larger hole.

The hole's diameter increases in the same proportion as other linear dimensions of the freely expanding plate.

3. SHARING THE LOAD

The steel rod carries 20 kN and the aluminium rod carries 7 kN.

The guided plate makes both rods extend by the same amount. For an elastic rod:

Extension = P*L/(E*A)

P = axial force

L = rod length

E = elastic modulus

A = cross-sectional area

Equal extensions, lengths and areas give:

$P_{steel}/E_{steel} = P_{aluminium}/E_{aluminium}$

$P_{steel}/P_{aluminium} = 200/70 = 20/7$

$P_{steel} = 27*200/(200 + 70) = 20$ kN

$P_{aluminium} = 27*70/(200 + 70) = 7$ kN

With a total load of 27 kN, the steel rod therefore carries 20 kN and the aluminium rod 7 kN.

4. THE PAINTED CUBE

24 cubes.

Cubes with exactly two painted faces lie along the original cube's edges, excluding the corners.

There are 12 edges, with two qualifying cubes on each:

The eight corner cubes each have three painted faces.

5. AN UNUSUAL STOPWATCH

Light **both ends of the first rope** and **one end of the second rope** simultaneously.

The first rope finishes burning after 30 minutes. At that instant, light the other end of the second rope.

The second rope has 30 minutes of burning time remaining if burned from one end. Burning it from both ends consumes that remainder in 15 minutes.

The total elapsed time is therefore:

Total elapsed time = 30 minutes + 15 minutes = 45 minutes.

Uneven burning means that halving a rope's length would not necessarily halve its burning time.

SESOC Membership Report

Report for SESOC Management Committee Meeting – August 2026

Prepared by Jenni Tipler

PURPOSE

To provide visibility to the Management Committee on membership data and trends.

EXECUTIVE SUMMARY

Membership numbers are down from last quarter – which is roughly in line with trends seen in previous years

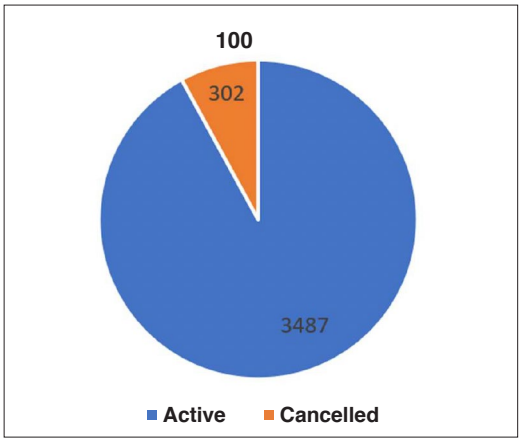
SESOC MEMBERSHIP SUMMARY AS AT 05.08.2026

Member Type	Aug-26	Jun-26	Feb-26	Nov-25	Aug-25
Complimentary Member	9	9	9	10	12
Journal Only	8	8	8	9	11
Journal Only Overseas	0	0	0	0	0
Life Member	20	20	20	20	20
Member	2995	3082	3034	3029	3067
Member Overseas	153	169	163	163	151
Member with IStructE(NZ)	140	156	157	160	161
Student Member	162	161	124	116	110
Total	**3487**	**3605**	**3515**	**3507**	**3532**
Growth over the Previous Quarter	-118	90	8	-25	-66
Growth over the Previous Year	1%	0%	1%	-1%	3%

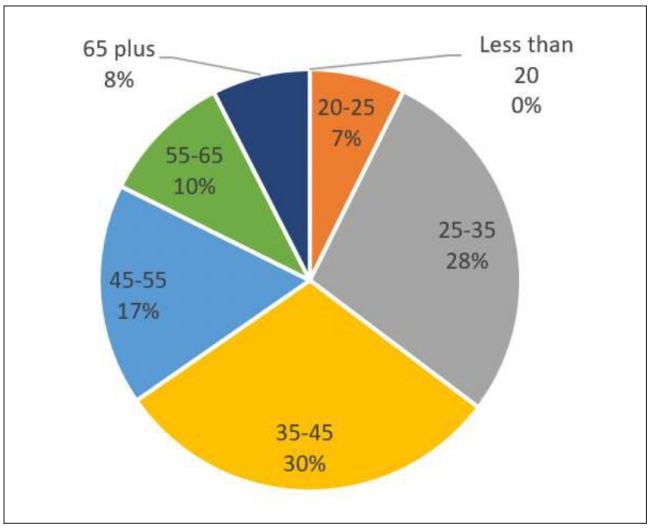
MEMBERSHIP GENDER SPLIT

Male	83.9%
Female	14.1%
Gender Diverse	0.0%
Not disclosed	2.0%

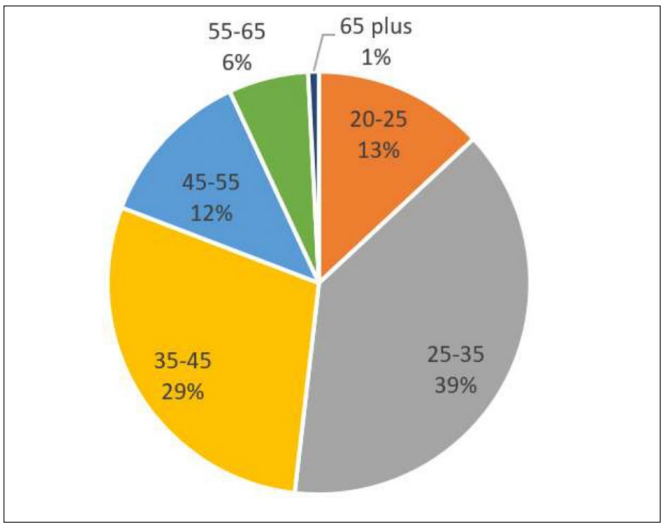
MEMBERSHIP STATUS



TOTAL MEMBERSHIP BY AGE



WOMEN - MEMBERSHIP BY AGE

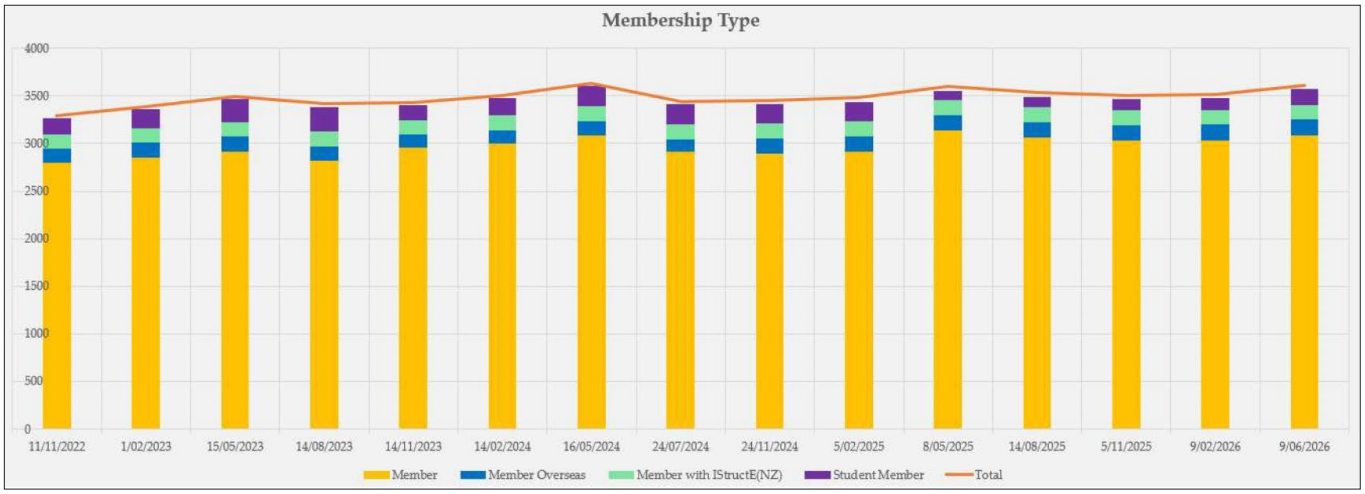


SESOC LIFE MEMBERS

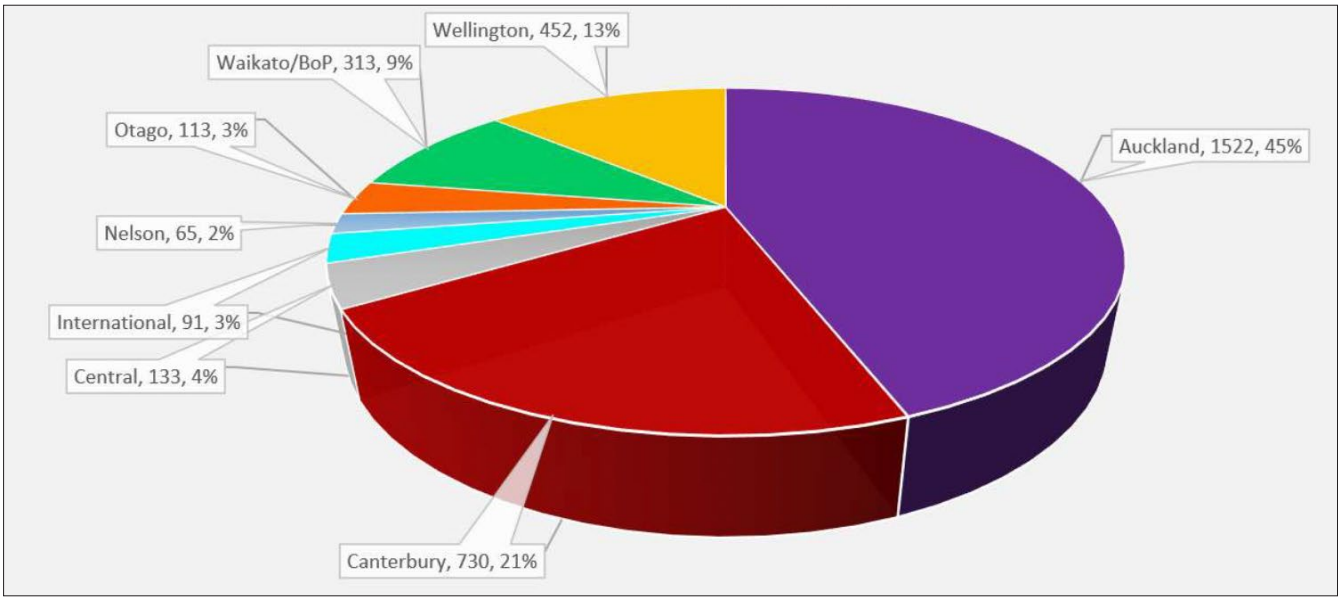
Barry Brown	1996
Ian Billings	2002
Richard Fenwick	2009
Barry Davidson	2009
Trevor Robertson	2010
Ashley Smith	2010
Mark Batchelar	2012
Robert Jury	2013
Gordon Hughes	2016
John Hare	2016

Geoff Bird	2016
David Brunsdon	2016
Des Bull	2017
Charles Clifton	2017
Michael Stannard	2017
Paul Campbell	2018
Jason Ingham	2020
Stewart Hobbs	2023
Hamish McKenzie	2023
Michelle Grant	2025

MEMBERSHIP GROWTH



MEMBERSHIP BY REGION OF AOTEAROA NEW ZEALAND



SESOC Conference 2027

Planning for the 2027 SESOC Conference is progressing well. The conference will be held at Cordis, Auckland from 30 June to 2 July 2027, with venue and conference management agreements now signed. The theme for the conference is Connecting Experience, Building Communities - focused on learning from the past, connecting with peers and building inspiration for the future

The conference website and sponsorship prospectus were launched during September 2026.

SESOC TREASURER'S REPORT

Management Committee Meeting 25th February 2026

Based on Engineering New Zealand Account Reporting to 31 January 2026

Currently tracking ahead of this year's budget. Anticipated end of financial year (September) Budget projected loss of **$111,530.00**. Tracking from end of July for a profit of $102,384.

Commentary

Income from advertising and seminar income has been down on the income side of the ledger. Expenses we have not had as high as projected costs for publications, the website is yet to be overhauled, seminar expenses have been lower in parallel to the related income, design guides line item has not been used.

Current Assets	
Accounts Receivable	$5,950
Westpac Cheque 08 (ENZ)	$141,097
Westpac Term Deposits (ENZ)	$150,000
Westpac Trust Investment (SESOC)	$395,000
Westpac Base 00 Account (SESOC)	$5,189
Westpac Bonus Saver 25 Account (SESOC)	$47,536
ENZ Receivables	$0
Total Current Assets	**$744,772**

Current Liabilities	
Sundry accruals	$0
Accounts payable	$0
GST	($3,313)
Tax payable	$5850
Total Current Liabilities	$8,790
Net Assets over Liabilities	**$734,502**

Comparison	
May 2026	$885,334

Income & Expenditure	
For the Financial Year to 30 September 2026, to date	
Income	$413,165
Expenditure	$310,781
Net Surplus	**$102,384**

Adam Langsford
SESOC Treasurer
www.sesoc.org.nz
C/o Engineering New Zealand, PO Box 12 241, Wellington 6144, New Zealand

Structural Engineering Society New Zealand (Inc.)
Membership Application

A Technical Group of Engineering New Zealand

Send to: Jenni Tipler
Structural Engineering Society New Zealand (Inc.)
Email: membership@sesoc.org.nz

I believe myself to be a proper person to be elected a member of the Structural Engineering Society New Zealand (Inc.) and do hereby promise that, in the event of my election, I will be governed by the Rules of the Society for the time being in force or as they may hereafter be amended and that I will promote the objectives of the Society as far as may be in my power.*

I hereby apply for membership of the Structural Engineering Society (Inc.) and supply the following details:

Title: ..... **First Names:** .....

Surname: ..... **Date of Birth:** .....

Permanent Address (Res): .....

..... **Home Phone:** .....

Postal Address (if different): .....

Qualifications and Experience: .....

Business Name & Address: .....

.....

Phone: ..... **Fax:** .....

Email address: ..... **Alternative Email** .....

Position Held: .....

SESOC rules can be found on the SESOC website

Membership Class Applied For: *(please tick applicable category)*	**Aotearoa New Zealand*** Subscription GST included	**Overseas**** Subscription 0%GST NZ$
SESOC Membership	○ $110.00	○ $130.00
Student Membership	○ Complimentary	Not Available
NZ IStructIE Membership	○ Nil	Not Available

Please state your Engineering New Zealand and/or IStructE membership numbers as applicable EngNZ.....

IStructE.....

* Aotearoa New Zealand Membership Subscriptions are in NZ dollars and include GST

** Overseas membership subscription in NZ dollars (GST not applicable)

Signature of Applicant : _____ **Date:** _____

PLEASE DO NOT SEND FEES WITH THIS APPLICATION
AN ACCOUNT WILL BE SENT ON YOUR ACCEPTANCE INTO THE SOCIETY
An application for membership can also be made on the Society website www.sesoc.org.nz

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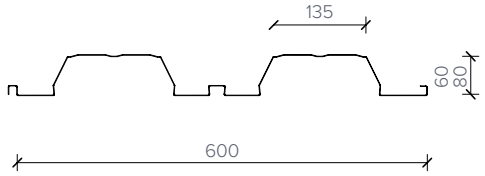
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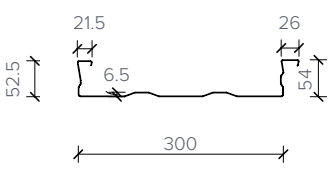
STEEL SUPPLIED BY NEW ZEALAND STEEL

Completed Projects: AIAL WP1 – 14,000m² | Union Square – Buildings A, B & E 8,350m²
Milford Apartments 8,500m² | One Stephens Ave Apartments 7,250m²
Northern Quarter – Tauranga CBD 8,350m²

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Design Tool: STEELSPEC online design software
https://www.metalcraftcompositeflooring.co.nz/

Specification available on:
masterspec

https://www.metalcraftgroup.co.nz/

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