SESOC

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JOURNAL

Embodied Carbon Specification Guide page 14

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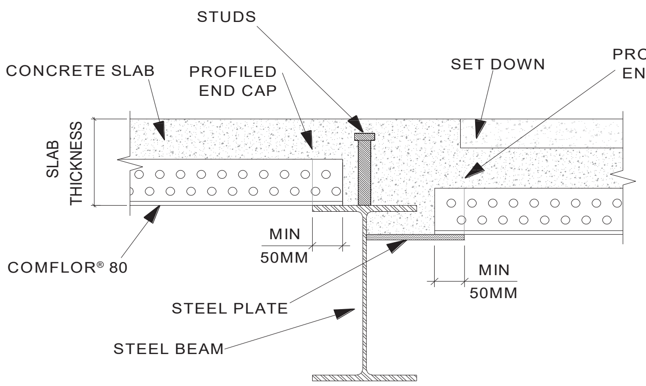

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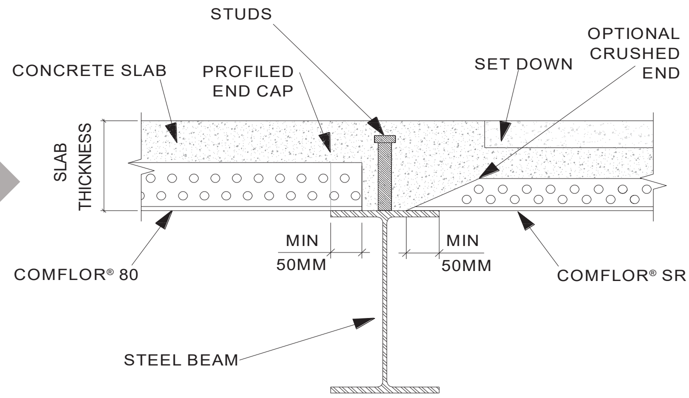

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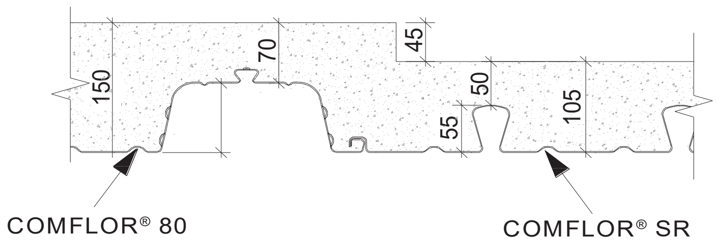

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SESOC

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

A Technical Group of Engineering New Zealand

Vol. 39, No.1

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

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SESOC
PRESIDENT'S REPORT

I'm hoping everyone had a relaxing and enjoyable summer break. We started the year with our annual SESOC election and AGM. Twelve candidates stood for election this year with 446 of our members voting – a great turnout. I would like to congratulate Adam Langsford, Carl Ashby and Laura Whitehurst who are newly elected members of the SESOC Management Committee. Returning elected members include Geoff Bird, Jenni Tipler, Tessa Beetham and Stuart Oliver. Andrew Gaul and Julio Ortiz have been co-opted onto the Management Committee recognising the significant contributions both made to SESOC in 2025, in particular Andrew's contribution to professional practice issues and Julio for his contribution to our training and communications. I would like to thank Andrew, Geoff, Jenni, Julio and Tessa for their ongoing commitment and contribution to SESOC and I look forward to working with them over the coming year.

I would also like to acknowledge the many contributions Paul Campbell and Jason Ingham have made to SESOC. Paul and Jason have been long-serving members of the SESOC Management Committee, including both being past Presidents. Paul and Jason decided not to stand for re-election this year in an effort to enable others to bring fresh perspectives and innovation onto the Management Committee.

At the AGM, held in Te Whanganui-a-Tara Wellington, we had four excellent speakers present information on the Wellington Te Ngākau Civic Precinct which is scheduled to re-open in March. Many buildings in the precinct were damaged during the 2013 Seddon and 2016 Kaikoura earthquakes and the seismic resilience issues in the precinct are challenging. The presentations consisted of:

- Claire Sharpe, Principal at Warren and Mahoney, presented *Te Ngākau Introduction*.
- Laura Whitehurst, Project Director at Holmes, presented *Wellington Town Hall Seismic Strengthening and Uplift: Design and Construction.*
- Chris Speed, Project Director at Dunning Thornton, presented *Built to Rock: The innovative engineering and adaptive re-use of the Wellington Town Hall Annex.*
- Tony Holden, Technical Director at Aurecon, presented *Te Matapihi – Wellington Central Library: Preserving the Past, Securing the Future.*

SESOC membership remains steady and at last report stood at 3515. Given the difficult economic times the Aotearoa New Zealand construction industry has been facing over recent years, it is gratifying to see our membership in such good shape.

The SESOC Sustainable Design Task Force has made good progress over the past quarter. The Task Force has been working with Masterspec developing technical specification clauses to make it easier for designers to specify materials with low embodied carbon. Once completed the specification clauses will also be available for members to use in their own specification templates. The Task Force has also developed an Embodied Carbon Specification guide to complement the new specification clauses. The guide provides technical guidance on how to specify commonly used structural materials to reduce embodied carbon emissions in construction projects. A copy of the new guide has been included in the journal, and we are seeking feedback from our membership on the document.

In collaboration with Engineering NZ, NZSEE, NZGS, TDS and CNZLS, SESOC provided feedback on the December 2025 Earthquake-prone Buildings Amendment Bill. The proposed changes to the earthquake-prone buildings system associated with this Bill are significant. While SESOC generally supported the Government's intention to simplify the process of identification and seismic retrofit, we proposed the following areas of improvements to the Bill in our submission:

- URM buildings in low seismic zones (e.g. Tāmaki Makaurau Auckland and Te Tai Tokerau Northland) in urban centres with high pedestrian traffic should be included within the scope of buildings requiring remediation under the EPB framework.
- All URM buildings (that, is, including those in low seismic zones and outside urban centres) should be subject to mandatory seismic strengthening or façade securing when undergoing a change-of-use.
- Modifying the proposed changes to ' "at any time" pathway' to ensure territorial authorities have necessary mechanisms they need to manage buildings that are severely deficient.
- Modifying the deadline extensions such that they are limited to five years at a time, up to an overall maximum of 15 years, to enable a more progressive approach to seismic risk reduction.

SESOC made an oral submission to the Transport and Infrastructure Select Committee on the 2nd of March to support our written submission. A copy of our written submission has been included in the journal.

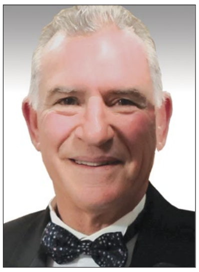
We have been making good progress with our software. Updates to the 2024 DZ TS 1170.5 webtool so that it aligns with TS 1170.5:2025 have been completed and we expect to release the new tool in the coming weeks. Validation of the new SESOC Concrete Retaining Wall app is also progressing well with support from NZGS. We expect to release this tool later in the year.

Looking ahead, Engineering NZ is undertaking a project to review and update CPEng practice fields. SESOC was approached by Engineering NZ in January to see if we would be interested in leading the review of the structural engineering practice field. We have agreed to lead this review and will collaborate with the Bridge Engineering Technical Society (BETS) to ensure the new practice field is also relevant to civil structures (such as bridges, reservoirs, and wharfs). We anticipate completing a draft of the new structural engineering practice field in April 2026 and are planning to seek feedback from our members on the draft in early May.

Lastly, the Canterbury Structural Group (CSG) will mark its 50-year anniversary in 2026. This is a significant milestone, and we understand CSG is planning to host an event later in the year to celebrate the many activities and successes of the structural group since its inception. SESOC thanks CSG for their ongoing support of structural engineers in Waitaha Canterbury, and I encourage you to join us for the CSG 50-year anniversary event.

Stuart Oliver
SESOC President
March 2026

NOTE FROM THE EDITOR



As we move into the second quarter of 2026, the structural engineering profession in Aotearoa New Zealand finds itself at a pivotal crossroads relating to consenting and compliance. For years, our industry has operated within a framework where the Building Consent Authorities (BCAs) - primarily local councils - acted as a primary gatekeeper for quality and compliance. However, recent legislative shifts—most notably the Building and Construction Amendment Act 2025 and the subsequent rollout of consent-free secondary dwellings—signal what I see as a clear government mandate to "cut the red tape".

While the implicit objective is to improve housing delivery and sector efficiency, for the structural engineer these reforms represent a profound shift in the locus of responsibility. The removal of BCA oversight for "low-risk" structures does not remove the requirement for Building Code compliance; rather, it places the full weight of that compliance directly onto the shoulders of the Licensed Building Practitioners (LBPs) and the engineers who support them.

With the Ministry of Business, Innovation and Employment (MBIE) progressing new liability rules and increased disciplinary penalties for professionals, the "survive until '26, thrive in '27" thinking within the construction industry takes on a different meaning. We find ourselves stepping into a future of self-certification where our internal quality assurance processes must be more robust than the council reviews they replace, and where our professional liability may be tested more than ever. This is particularly critical as we see an influx of overseas building products and standards now more widely accepted under the new International Standards recognition schemes, while the the industry's and the public's expectation for sustainable practices and materials grows higher

This era of deregulation is, paradoxically, an era that demands higher professional standards. As evidenced by the winners of 2025 SESOC Structural Engineering Excellence Awards, the ability to innovate while maintaining rigorous safety and sustainability targets remains our core value. Whether it is managing the nuances of the National Seismic Hazard Model (NSHM) or designing for "repairability" rather than just survival, our role as the guardians of the built environment is expanding.

As we prepare for this new regulatory environment, we must ask ourselves: "Are our practices robust enough for the autonomy the new system grants us?" The "red tape" may be thinning, but the blueprint for safety and professional integrity that we operate under as stringent professionals is likely to place more pressure on our society.

Stewart Hobbs
Editor

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**
September 2026	Up to end of June 2026	20 July 2026
April 2027	Up to end of January 2027	20 February 2027
Advertising	**Booking Deadline**	**Material Deadline**
September 2026	30 June 2026	20 August 2026
April 202t	30 January 2027	20 March 2027

Advertising rates: As part of our ongoing commitment to structural engineering we are always trying to improve the status and quality of the SESOC journal. The following rates apply:

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

Readership: *There are currently approximately 3500 subscribers to the SESOC Journal.*
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Advertising copy: *You need to supply a high definition PDF of your advertisement. If there are any problems with the copy we will revert to you with comments, but note responsibility for the correctness of advertisements rests with the advertiser.*
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.

SESOC EPB BILL SUBMISSION

Submitted by Stuart Oliver - President of the Structural Engineering Society New Zealand (SESOC)

February 13 2026 (As submitted)

Building (Earthquake-prone Buildings) Amendment Bill

Tēnā koe

We thank you for the opportunity to comment on the proposed Building (Earthquake-prone Buildings) Amendment Bill. SESOC would welcome the opportunity to appear before the Select Committee to make an oral submission, or to be contacted to make any further clarifications on the matters raised in our submission.

The Structural Engineering Society New Zealand (SESOC) is a non-profit organisation representing approximately 3500 practicing structural engineers. Our objectives are:

- To promote the science, art and practice of structural engineering;
- To ensure the advancement and dissemination of knowledge relating to structural engineering; and
- To provide a forum for structural engineering practitioners to communicate amongst themselves and to the public at large.

This submission has been developed in collaboration with the New Zealand Geotechnical Society, New Zealand Society for Earthquake Engineering, Timber Design Society, the Concrete New Zealand Learned Society, HERA, and Engineering New Zealand. Together we represent engineers working across different aspects of the earthquake prone building system.

GENERAL COMMENTS

SESOC supports the Government's intention to simplify the process of identification and seismic retrofit ; and a focus shift toward the highest-risk building typologies while seeking to improve compliance through a simpler, more targeted regulatory system.

We are generally of the view that a framework that is practical and consistently implemented is more likely to deliver meaningful life-safety outcomes than a comprehensive and complex system that is difficult to comply with or enforce. However, as a professional body representing structural engineers, our focus is on ensuring that structural safety is not compromised as a result of proposed amendments.

In particular we support:

- **Risk Based Focus**
  The intent to prioritise buildings that present the highest risk to life, rather than applying blanket requirements across all building types.
- **System simplification**
  The intent to simplify the earthquake prone building (EPB) identification system and required retrofits as it is likely to improve consistency of application and compliance across territorial authorities and building owners.
- **Exclusion of 1–2 storey unreinforced masonry buildings in small towns**
  The decision to exclude low-rise unreinforced masonry (URM) buildings in small towns from mandatory strengthening requirements as they typically reflect lower exposure risk.
- **Separation of seismic strengthening from means of escape from fire and disability access and facilities**
  Allowing mandatory seismic strengthening works to proceed without having to comply with 'as nearly as is reasonably practicable' (ANARP) building code provisions relating to means of escape from fire and disability access and facilities, provided the strengthening works do not negatively impact existing accessibility, fire protection systems or fire performance.
- **Targeted retrofit solutions**
  cost-effective strengthening outcomes. We hope standardised solutions can be developed where appropriate to further reduce cost.
- **Removal of %NBS as an EPB identification tool**
  Removing %NBS as the primary EPB identification tool, provided MBIE continue to support %NBS (or a future replacement) to ensure the assessments outside the EPB framework can continue to be implemented.

AREAS OF RISK AND CONCERN

Whilst we generally support the intent of the proposed amendments. SESOC would like to raise the following risks that we have identified within the proposed EPB framework.

Exclusion of higher-risk buildings in low seismic zones
Some buildings with high seismic risk will be excluded under the proposed framework, with no clear mechanism to ensure that these risks are identified or appropriately managed. In particular we are concerned about unsecured URM buildings in urban centres within low seismic zones (e.g. Auckland and Northland) which have high pedestrian traffic. Experience from past earthquakes has demonstrated falling masonry from unsecured facades represents a high life safety risk to members of the public.

Façade and parapet securing is a cost-effective means for reducing risk to the population. SESOC recommends the highest risk buildings in low seismic zones (e.g. Auckland and Northland) be reconsidered for targeted strengthening such as URM buildings in urban centres with high pedestrian traffic.

Building change of use and ANARP provisions
SESOC is concerned that Section 115 as currently drafted, carves out all buildings from needing to undergo a review for seismic performance, rather than being limited to EPBs only. This would enable building changes of use to occur without seismic assessment or retrofit, including for buildings with known or potential structural vulnerabilities. SESOC recommends that the proposed amendments to section 115 of

the Building Act be amended so that they apply only to earthquake-prone buildings, rather than to all buildings.

In principle, SESOC supports the proposed ANARP carve-out for lower-risk buildings, recognising that building change-of-use (e.g. converting the upper levels of a URM building into apartments) can act as a financial mechanism to increase a buildings value, which can in turn enable building owners increased access to capital to fund seismic strengthening.

SESOC considers that a change of use should trigger targeted seismic retrofit requirements to all URM buildings (i.e. including URM buildings in low seismic zones within urban centres; and URM buildings in medium and high seismic zones outside of urban centres) , given their well-established life-safety risks. A change of use usually signals a long-term re-purposing of a building, which is an appropriate trigger for a more comprehensive seismic upgrade.

Accordingly, SESOC recommends retaining the current change-of-use ANARP requirements, with limited and clearly defined exceptions. Any exceptions to ANARP requirements should be tightly scoped (i.e. EPBs and all URM buildings) to ensure that change-of-use decisions continue to appropriately manage life-safety risk, particularly where increases in occupant numbers or changes in use intensity are proposed.

Deadline extensions
Long extensions of compliance timeframes, of up to 15 years, risk undermining the intent of the Bill. While extensions may be appropriate in limited and well-justified circumstances, blanket extended deferral periods delay the mitigation of known seismic risks and prolong exposure to avoidable life-safety hazards.

SESOC suggests limiting extensions to a maximum of five years at a time, up to an overall maximum of 15 years. This would retain flexibility where justified, while better supporting ongoing and demonstrable seismic risk reduction and reinforcing the intent of timely remediation.

Narrowing of the "at any time" pathway
Territorial authorities use the existing 'at any time' pathway to manage buildings that are severely deficient and/or have a very high seismic risk. The proposed narrowing of the 'at any time' pathway will reduce the ability of territorial authorities to effectively manage such buildings. In particular, territorial authorities will have no ability to require assessment or remediation of buildings that fall outside the scope of what is defined in the Act as an EPB, leaving these buildings without a clear mechanism for intervention.

This may result in known or emerging (newly identified as knowledge evolves) risks remaining unaddressed where a building does not meet the specific criteria or timeframes set out in the framework, despite presenting a disproportionate risk to occupants or the public. A recent example of this occurred in the UK where it was discovered that Reinforced Autoclaved Aerated Concrete panels were prone to sudden collapse after an extended period of time performing satisfactorily, with this issue only becoming apparent near the end of their usable life.

Furthermore, the 'at any time' pathway drafted in the Bill excludes buildings that were designed before 1st January 1976. This limitation will prevent territorial authorities from managing earthquake prone buildings that were missed during the identification period. This would include, for example, URM buildings that were miscategorised as non-URM buildings, or URM buildings whose existence was only discovered by a territorial authority after the identification period.

Accordingly, SESOC recommends the proposed changes to the 'at any time' pathway" detailed in the Bill be amended to ensure territorial authorities have the necessary mechanisms they need to manage buildings that are severely deficient. However, we agree the use of the 'at any time" pathway' should be limited to address buildings that represent a very high risk to public safety and as such, SESOC agree that this pathway should include agreement from the MBIE chief executive.

An alternative approach would be to expand the dangerous building provisions within the Act to better address seismic risk. The dangerous building pathway could be broadened to include clearly defined seismic life-safety triggers, where risks are demonstrably high but are not otherwise captured by the proposed earthquake-prone buildings framework.

Priority buildings
As drafted, the Bill would, in effect, provide extended remediation timeframes for hospitals, schools, and emergency facilities. SESOC does not support this change and is not aware of any clear rationale for removing these buildings from priority status. These facilities frequently accommodate large numbers of people, including individuals who may have limited ability to evacuate quickly, and play a critical role in community response and recovery following an emergency.

For these reasons, SESOC considers it essential that they continue to be remediated within the shortest practicable timeframes and, where feasible, to a higher standard than the minimum required under the earthquake-prone buildings regime, reflecting their importance to society.

Further, SESOC is concerned that many buildings supporting these functions are made of concrete or other heavy materials and are two-storeys or less and as such will be excluded from the EPB framework. We recommend the EPB methodology include a height limitation in addition to the storey definition, to ensure that larger buildings, particularly those accommodating large numbers of occupants, are appropriately identified and reviewed.

OTHER CONSIDERATIONS

Ongoing Role of %NBS
The proposed EPB framework removes the %NBS as the earthquake-prone building identification tool. While SESOC supports this change, %NBS (or a future replacement) is expected to continue to be used in several other regulatory and commercial contexts including:

- Demonstrating ANARP for non-EPBs undergoing a change of use or alterations
- Informing targeted versus full retrofit decisions
- Market assessments
- Retrofit design and engineering decision-making
- Insurance and finance purposes

This change creates a risk of ongoing confusion and inconsistency across the industry, particularly where %NBS continues to influence decisions without a clear or consistent role within the revised EPB framework.

SESOC recommends MBIE continues to support %NBS (or a future replacement) to ensure the assessment procedure is maintained and can continue to be used outside the EPB framework.

Clarity in EPB methodology and terminology
The Bill relies fundamentally on the application of the EPB methodology that has not yet been developed or consulted on. Consequently, it is difficult to accurately comment on all aspects of the EPB framework without understanding the technical detail of the EPB methodology. This is because the effectiveness of the proposed framework is highly dependent on the methodology being clearly defined, robust, and accompanied by explicit guidance on intent and interpretation.

Given the central role the EPB methodology plays in determining building identification, risk thresholds, and compliance pathways, clarity and consistency in its development will be essential to achieving the Bill's stated objectives and ensuring confidence in its implementation.

SESOC requests a copy of the EPB methodology in draft form is provided for review as soon as it becomes available.

Removal of building parts provisions
SESOC considers that additional clarity is required within the framework to manage non-typical building configurations and boundary cases. In the past, these have been managed through provisions relating to 'parts' of a building. In particular, clearer guidance is needed on how buildings with shared or common party walls are to be treated, including rows of attached buildings where a change of use may apply to a single tenancy or unit rather than the entire structure. Clarification of the definition of a 'building' in these circumstances is important to ensure consistent and equitable application of the provisions.

Further consideration is also required for buildings with mixed or staged construction, including additions or alterations constructed using different structural materials or systems (an example of this is a building consisting of URM in the lower levels with a historic extension constructed using reinforced concrete). It is unclear how such buildings would be assessed under the proposed typology-based approach, and whether individual 'parts' of a building may continue to be referenced where this is necessary to appropriately reflect differing seismic performance and risk profiles.

Supporting implementation of the new EPB framework
Proposed changes to the EPB framework detailed in the Bill are significant. Successful implementation of the new framework will require support from central government, including funding for the following:

- Establishment of a panel of contracted engineering experts to support the implementation (as a minimum) of the system. This panel would provide technical advisory input to MBIE and territorial authorities. This would include provision of technical advice during the transition to the new framework to support activities such as removing the earthquake prone status from more complex buildings.
- Training for contractors and building inspectors so they have the necessary competence to implement the new framework, in particular the appropriate use of standardised retrofit solutions for URM buildings.
- Training for engineers recognising they will have a key role conveying the changes to the EPB framework to building owners and users.

CONCLUSION

SESOC supports the overall intent of the Building (Earthquake-prone Buildings) Amendment Bill to simplify the earthquake-prone buildings framework and to better target the highest seismic risks.

As currently drafted, SESOC considers that many of the highest-risk building typologies have been appropriately identified in the Bill and that the proposed framework removes a number of low-risk buildings that do not warrant regulatory intervention. However, SESOC remains concerned that some key high-risk building typologies in low seismic regions (e.g. Auckland and Northland) are excluded from both the earthquake-prone buildings framework and the change-of-use provisions. In these cases, high occupant densities combined with known structural vulnerabilities may result in disproportionate life-safety risks that are not adequately addressed, leaving residual risk that warrants further consideration.

Taken together, these issues indicate that further refinement is required to ensure the framework remains genuinely risk-based, equitable, and focused on achieving consistent life-safety outcomes across all regions. In particular SESOC believes the Bill requires the following changes:

- URM buildings in low seismic zones (e.g. Auckland and Northland) in urban centres with high pedestrian traffic should be included within scope the scope of buildings requiring remediation under the EPB framework.
- All URM buildings (i.e. including those in low seismic zones and outside urban centres) should be subject to mandatory seismic strengthening or façade securing when undergoing a change-of-use.
- Modifying the proposed changes to 'at any time" pathway' to ensure territorial authorities have necessary mechanisms they need to manage buildings that are severely deficient.
- Modifying the deadline extensions such that they are limited to five years at a time, up to an overall maximum of 15 years to enable a more progressive approach to seismic risk reduction.

SESOC believes the matters raised in this submission would strengthen the Bill's ability to deliver timely, proportionate, and effective seismic risk reduction across the building stock.

Thank you for the opportunity to provide feedback on this important initiative.

Ngā mihi nui,

Stuart Oliver
President of the Structural Engineering Society New Zealand (SESOC)

JOINT TECHNICAL SOCIETIES POSITION STATEMENT ON REPRESENTATION DIVERSITY EQUITY AND INCLUSION

Bridge Engineering Technical Society

NEW ZEALAND GEOTECHNICAL SOCIETY INC

SESOC

A Collaborating Technical Society

As technical societies, we recognise that diversity, equity, and inclusion (DEI) are critical to the growth, innovation, and success of the engineering profession. We strive to excel for our members and in our contributions to society. These aims require that we create a culture of inclusion in which all members feel valued and can contribute to our capability as societies.

Engineering remains a sector that lacks diversity. It is upon all of us—leaders, members, and partners – to foster a more inclusive and diverse industry. Change comes from the top. As technical societies, we have an important role to play in leading this transformation.

Commitment to Representation

We commit to ensuring diverse representation across all aspects of our society, including:

- Targeting a diverse and inclusive Management Committee, so that we can better lead our industry towards solutions that are inclusive, accessible, and sustainable.
- Ensuring equity in selection of conference speakers and panellists, and that equitable opportunities to contribute and lead are available to all, targeting 40% women/40% men/20% any gender representation by 2027.
- Requiring technical subgroups, working groups, webinars, seminars and publications to promote inclusive participation and recognise the value of varied perspectives in engineering solutions.

Creating a Safe and Empowering Community

We strive to create a professional environment where all individuals—regardless of gender, race, ethnicity, disability, sexual orientation, or background—feel safe, valued, and empowered to succeed. This includes:

- Raising awareness of systemic barriers and unconscious biases that impact career progression and workplace culture, so that we can design fairer systems, leading to stronger collaboration and engineering solutions for communities.
- Providing support networks that foster mentorship, sponsorship, and professional development opportunities.

Changing the Public Perception of Engineering

The way engineering is perceived plays a significant role in attracting diverse talent. We commit to actively reshaping outdated stereotypes by:

- Acknowledging the importance of authentic, diverse role models in engineering as a source of inspiration for future engineers.
- Showcasing engineering as a field that values and is driven by collaboration, creativity, and inclusivity—not just technical skills.
- Engaging in outreach programmes and partnerships to encourage participation from under-represented groups.

Guiding Principles for a More Inclusive Industry

To drive lasting change, we commit to embedding the following principles in all society activities:

1. Authentic Engagement – Upholding Te Tiriti ō Waitangi through the recognition of Matauranga Māori and the principle of partnership. Valuing cultural competency as a core skill for Aotearoa New Zealand's engineers so that as a profession we can work with Māori and Iwi to deliver outcomes that benefit all society.
2. Equitable Opportunity – Understand that opportunities within and aligned with the technical societies have been defined by the people who have held them to date. Advocate for strong candidates in all roles and positions within societies, while making space for diverse voices to contribute.
3. Active Inclusion – Foster an environment where all voices are heard, valued, and respected. Ensure this is a guiding principle when defining the working environment, particularly in technical working groups, standards committees, and society sub-committees.
4. Accountability from Leadership – Ensure that DEI commitments are upheld at the highest levels of our society.
5. Transparency and Measurement – Ensure that all societies report on diversity metrics and progress, not only relating to membership statistics but also across all mediums of engagement, inclusive of conferences, publications, webinars and public events. Ensure that reporting of data is honest and representative of the issue at hand.
6. Commitment – We commit to use our influence to promote these guiding principles beyond the immediate purview of the society.

SESOC EMBODIED CARBON SPECIFICATION GUIDE

CONTENTS

Foreword from the SESOC Sustainable Design Task Force

The SESOC Embodied Carbon Specification Guide is the result of nine months of development, drawing on material from previous SESOC journal articles authored by members of the Task Force and informed by consultation with representatives across the materials industry.

Purpose:
The guide is designed to help structural engineers specify concrete, steel, and timber with progressively lower embodied carbon in everyday projects. It aims to serve as a practical reference for engineers building capability in this emerging area of professional practice.

Industry Opportunities:
New Zealand's materials sector is well placed to deliver significant embodied-carbon reductions over the next five years—for example, with the opening of NZ Steel's Electric Arc Furnace at Glenbrook later this year. Structural engineers have an important role to play as we alert our clients to these opportunities and help create mainstream demand for lower-carbon products through design specifications.

Continuous Improvement:
As decarbonisation advances, new data, technologies, and supply-chain capabilities will continue to emerge. This guide is intended as a living document that will be updated to reflect—and help drive—industry progress.

Example specification Clauses:
Example specification clauses will be added over the coming months, with a planned release in mid-2026. Work is also underway with Masterspec to make these clauses available through its online specification platform within a similar timeframe.

Feedback invited:
Version 1.0 of the guide is released here for public comment. SESOC members and other interested parties are invited to submit feedback to the SESOC Executive. Officer, John Snook (executiveofficer@sesoc.org.nz), by Friday 15 May 2026. An updated version, including example specification clauses, will be published on the SESOC website in mid-2026.

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Disclaimer

The information in this guide is drawn from a wide range of resources as well as the experience of the authorship team, and the research focus groups. Following this guide does not guarantee a level of embodied carbon performance on an individual project. This guidance is intended for informational and educational purposes only; it is not intended to be used as a means of compliance with regulatory requirements or performance-based frameworks for embodied carbon.

It is important to note that this document is issued as guidance and has no official status. However, designers are advised to consider the issues raised and the possible solutions offered when preparing designs, and to exercise their engineering judgement in determining a suitable course of action in this regard.

Where errors or omissions are noted in the document, it is requested that users notify SESOC through exec@sesoc.org.nz.
The authors would like to express their thanks to Masterspec for partnering with SESOC to make embodied carbon specification clauses accessible through their platform.

1. INTRODUCTION

1.1 Background

An overview of embodied carbon for buildings in an Aotearoa New Zealand context is provided in the MBIE Whole-of-Life Embodied Carbon Framework [1]. A high level methodology for assessing the embodied carbon on buildings in New Zealand is provided in the 2022 MBIE Whole-of-Life Embodied Carbon Assessment: Technical Methodology [2] and in the MBIE + Engineering NZ-endorsed New Zealand Green Building Council (NZGBC) Methodology from 2024 [3]

1.2 Scope of guidance

This document provides technical **guidance** on specifying commonly used structural materials to reduce embodied carbon emissions in construction projects, including recommendations for weighted average A1-A3 GWP limits to be included in material specifications.

Example specification clauses are provided in Appendices E, F & G for consideration and adaptation by specifiers to align with specific project requirements. Masterspec subscribers can access these specification clauses through Masterspec Libraries online.

1.3 Scope limitations

Material specification is a practical step to reducing embodied carbon in construction, although it will not be sufficient on its own to meet New Zealand's 2050 emission reduction targets.

Broader design decisions about, for example, project scope, scale, adaptive re-use, and site selection will create opportunities to deliver better embodied carbon outcomes in the most cost-effective way. This specification guide should be used in conjunction with SESOC's Top Tips for Low Carbon Design [4]. This guidance document relates to Tip numbers 18-21 (of 21) only. We recommend that project teams consider Tips 1-17 for decarbonising their design before making interventions in materials specifications.

This document is focused on embodied carbon only and does not provide guidance on operational carbon emissions or wider environmental impacts such as biodiversity or toxic substances. Section 7 provides introductory guidance on the specification of structural materials for broader sustainability outcomes.

1.4 Life-cycle scope

The scope of this guide is "cradle to gate" embodied carbon – emissions from modules A1–A3 of the Life Cycle Assessment (LCA) framework, which cover the product stage, including material production. Although a full embodied carbon assessment should include all life-cycle stages to inform holistic decision-making during design, modules A1–A3 are most directly affected by structural specifications.

General information on the LCA system can be found in the MBIE Whole-of-Life Embodied Carbon Framework [1] and Whole-of-Life Embodied Carbon Assessment Technical Methodology [2].

1.5 Reducing embodied carbon over time

To meet emissions reduction targets, embodied carbon will need to be reduced over time. This guide recommends A1-A3 GWP limits for structural materials that are considered appropriate at the time of writing. These limits are intended to be achievable for everyday projects, and to stimulate increased demand for lower-carbon products that are available in the market.

For some materials, like concrete, guidance is provided on how A1-A3 GWP limits might be reduced over the next few years. We anticipate that this guidance will need to be updated over coming years as decarbonisation gains momentum in the construction sector, and as material suppliers make progress on their various commitments towards net zero carbon [5] [6].

1.6 Fabrication emissions

For the purpose of this guide, emissions from domestic pre-fabrication workshops may be excluded from the specified A1-A3 GWP limits. This exclusion has been suggested as an interim measure due to a current lack of emissions data, and would apply to precast concrete fabrication yards, steel fabrication workshops and pre-nailed timber fabrication workshops.

The materials specifications in this guide are focussed on quantifying the embodied carbon from manufacturing and offshore fabrication processes, which usually represent the majority of A1-A3 emissions. Although domestic pre-fabrication emissions are excluded from the suggested GWP limits, they must still be accounted for in any LCA that is undertaken.
International standards include off-site fabrication emissions within lifecycle stage A3. In New Zealand, there is currently insufficient data to quantify these emissions reliably. The limited global data available is not representative of New Zealand's pre-fabrication practices and energy grid characteristics. In the current environment, one of the following pragmatic approaches tend to be adopted when completing an LCA:

i. *Assume pre-fabrication emissions are small, and consider them implicitly covered by the $kgCO_2e/m^2$ value that is adopted for A5 emissions, or*
ii. *Include a conservative uplift (for example +10–15%) added to the known A1–A3 value for the relevant product/material to account for pre-fabrication emissions.*

It is anticipated that the fabrication sector will, in future, provide New Zealand-specific data to close this information gap. Once available, this data will allow pre-fabrication emissions to be incorporated directly into the specified A1–A3 values for relevant products and materials.

1.7 Transport emissions

For the purposes of this guide, transport emissions from the materials factory to the domestic pre-fabrication workshop may be excluded from the specified A1-A3 GWP limits. However, all transport emissions must still be accounted for in any LCA that is undertaken.

If competitive tenders are expected from both domestic and offshore fabricators, and if A4 emissions for imported products are significant enough to affect decisions about product selection, the specifier will need to modify the standard scope requirements to allow meaningful comparison (e.g. specify A1-A4 GWP limits or A1-A5 GWP limits for that material).

Guidance on A2 or A4 transportation emissions for imported products can be found on the website of the National Embodied Carbon Data Repository [7]. Transportation emissions factors are taken from the Ministry for the Environment's 2023 emissions factor database, with assumptions about transport modes and distances by BRANZ.

Table 1.1: Examples of approximate A4 transportation emissions for materials [7]

Origin – Destination	*Transport modes*	*A4 GWP (conservative)*
Auckland – Tauranga / Rotorua	*Road freight*	*0.03 kgCO_2e/kg*
Auckland – Wellington	*Road freight*	*0.09 kgCO_2e/kg*
Auckland – Christchurch	*Road freight + ferry*	*0.15 kgCO_2e/kg*
Australia (East Coast) – New Zealand	*Container ship + road freight from NZ port*	*0.1 kgCO_2e/kg*
East & Southeast Asia – New Zealand	*Container ship + road freight from NZ port*	*0.3 kgCO_2e/kg*
UK & European Union – New Zealand	*Container ship + road freight from NZ port*	*0.6 kgCO_2e/kg*
USA (West Coast) – New Zealand	*Container ship + road freight from NZ port*	*0.3 kgCO_2e/kg*

As GWP footprints decrease, the relative significance of A4 Transport emissions will increase.

For example:

Table 1.2: Examples illustrating the relative importance of (A4) transport emissions to GWP

Example Product	*A1-A3 GWPf*	*Indicative A4 GWP*	*A4 contribution*
Reinforcing steel (blast furnace, ex NZ)	*4.0 kgCO_2e/kg*	*0.1 kgCO_2e/kg*	*2.5 %*
Reinforcing steel (EAF, ex NZ from 2026)	*1.9 kgCO_2e/kg*	*0.1 kgCO_2e/kg*	*5 %*
Reinforcing steel (EAF, ex Korea)	*0.5 kgCO_2e/kg*	*0.3 kgCO_2e/kg*	*38 %*
Cross-laminated (CLT) panel, untreated (timber ex-New Zealand)	*70 kgCO_2e/m^3*	*24 kgCO_2e/m^3*	*26 %*
CLT panel, untreated (ex-Australia)	*250 kgCO_2e/m^3*	*50 kgCO_2e/m^3*	*17 %*
CLT panel, untreated (ex-Sweden)	*53 kgCO_2e/m^3*	*282 kgCO_2e/m^3*	*84 %*

1.8 Project communication

We recommend that the proposed A1-A3 GWP limits for key structural materials are communicated to the client and the wider project team at the preliminary design stage, for example in a Design Features Report. This allows time for review and discussion of any procurement implications.

> ***The SESOC Commercial Design Features Report Template is being updated to include sustainability features including GWP limits for structural materials – expected to be available from mid-2026 [8].***

1.9 Beyond carbon

Although the focus of this guide is lowering embodied carbon, the pillars of sustainable construction are much wider. The holistic view of sustainability links the protection of the planet's natural resources and ecosystems, with the centring of social equity and economic growth through financial responsibility, providing a framework to guide decision making.

Guidance on specification to achieve broader sustainability outcomes is provided in Section 7.

1.10 References

Additional resources are available via the SESOC Sustainable Design Resources page [9].

2. GLOSSARY

- **Biogenic carbon**
  Biogenic carbon, reported as GWPb in an EPD, is carbon absorbed by plants via photosynthesis and stored in bio-based materials like timber. It remains sequestered from the atmosphere while in buildings and is released only if the timber is incinerated or allowed to decompose. In contrast, fossil carbon – reported as GWPf – is released from long-term underground storage when fossil fuels are burned.
- **Carbon Footprint of a Product (CFP)**
  A third-party verified document which quantifies the direct and indirect greenhouse gas emissions generated throughout a product's life cycle. A CFP focuses exclusively on global warming potential, so it does not need to address the other environmental impacts covered by an EPD.
- **Embodied carbon**
  The total greenhouse gas emissions associated with a product's entire lifecycle—from raw material extraction through to disposal. The embodied carbon of a building excludes operational emissions during the life of the building. 'Upfront Embodied Carbon' is a subset of this, comprising lifecycle modules A1-A5.
- **Environmental Product Declaration (EPD)**
  A third-party verified document outlining a product's environmental impact throughout its lifecycle, including embodied carbon. EPD Australasia registers EPDs for building and construction products in Aotearoa New Zealand and Australia [10].
- **Global Warming Potential (GWP)**
  A measure of a product's embodied carbon per unit of material, typically expressed in terms of equivalent carbon dioxide per unit, such as $kgCO_2e/kg$ or $kgCO_2e/m^3$. GWP accounts for all greenhouse gas emissions and is stated on an EPD.

  In order to reduce the embodied carbon footprint of a structure, the project team can reduce the quantity of new materials and/or reduce the GWP of the materials that are used.
- **Global Warming Potential limit (A1-A3 GWP limit)**
  Where GWP limits are mentioned in this guidance document, they refer to a **weighted average limit** for each material specification category across the whole project. For example, all 30 MPa concrete mixes used on a project may fit within one specification category and share one weighted-average GWP limit, while 45 MPa concrete mixes may be specified in a separate category with its own limit. A similar approach can be used for other materials – for instance, applying one weighted-average limit for all hot-rolled steel and a different limit for all cold-formed steel. Individual elements may exceed the specified GWP limit as long as their higher values are balanced by corresponding reductions in other elements within the same specification category.

  GWP limits in this guidance document apply to life cycle modules A1-A3 only. This is known as the "product stage" or "cradle-to-gate" and encompasses raw materials extraction & processing (A1), transport to the manufacturer and fabricator (A2), and manufacturing processes (A3).

  Refer to the 'Life cycle scope' in Section 1 for clarification of fabrication emissions in the NZ context.
- **Life-Cycle Assessment (LCA)**
  A methodology defined by ISO 14040/14044 for evaluating environmental impacts across a product's life stages, from raw material extraction to disposal or recycling. A LCA approach is used to assess the environmental impacts from embodied carbon in the built environment but can also be used to consider other environmental impacts.

- **Masterspec**
  A digital specification tool for building designers in New Zealand, created by Construction Information Ltd, which is jointly owned by the New Zealand Institute of Architects (NZIA) and the Master Builders Association (MBA). GWP limits can be embedded into specifications for structural materials using Masterspec, or via bespoke specification documents. Guidance has been cross-referenced to the relevant Masterspec materials specifications.
- **SESOC**
  Structural Engineering Society New Zealand.

3. CONCRETE

(Referenced Masterspec Sections: 3105, 3106)

For background and context, refer to the SESOC Journal article: Specifying concrete with lower up-front carbon [11].

3.1. Reinforcing steel *[Masterspec 3112]*

3.1.1 To reduce embodied carbon

- Reduce steel weight via scope reduction, design optimisation, or re-use of existing structures. Using high strength reinforcing steel (e.g. Grade 500E) allows reinforcement quantities to be reduced compared with lower-grade steel.
- Specify an A1-A3 GWP limit, which is likely to require a portion of the steel on the project to have a significant recycled steel content, manufactured using an Electric Arc Furnace (EAF)

Reinforcing steel makes up a substantial share of the global warming potential (GWP) of reinforced concrete. For instance, if a 30 MPa concrete element contains 1% reinforcing steel with an A1–A3 GWP of 4.0 kg CO_2e/kg, the steel alone can account for roughly 50% of the element's embodied carbon [11].

Reinforcing steel is discussed first in this section because it is often the easiest and most common starting point for reducing embodied carbon in reinforced concrete structures – it is relatively straightforward to specify and supply.

3.1.2 Recommended A1-A3 GWP limit

- In 2026-2027: 2.0 kgCO_2e/kg for reinforcing bars and mesh.
- In 2028-2029: 1.9 kgCO_2e/kg for reinforcing bars and mesh.

3.1.3 Availability from certified suppliers

See Table 3.1 for products manufactured using significant (>20%) recycled steel content with ACRS certification to demonstrate compliance with NZ standards.

Table 3.1: Global Warming Potential for reinforcing steel products with recycled content

Supplier	Recycled content (1)	A1-A3 GWP (2)	Comments
Pacific Steel (NZ)	*44%*	*~1.9 kgCO_2e/kg*	*Standard reo coil & mesh feed [12]*
Pacific Steel (NZ)	*82%*	*~0.5 kgCO_2e/kg*	*DCRB product, available 2026 [12]*
Infrabuild (Australia)	*Not reported*	*~2.1 kgCO_2e/kg*	*Average mesh, EPD expired '25 [10]*
Infrabuild (Australia)	*Not reported*	*~1.7 kgCO_2e/kg*	*Average bars, EPD expired 2025 [10]*
Infrabuild (Australia)	*94%*	*~1.0 kgCO_2e/kg*	*G500E bar ex Laverton Mill [10]*
Generic supply (Asia)	*30%*	*~1.8 kgCO_2e/kg*	*NZ compliance to be checked for specific suppliers [13]*
NatSteel (Singapore)	*22%*	*~1.8 kgCO_2e/kg*	*Ex Nauhria reinforcing, HUB-4165*
NatSteel (Singapore)	*92%*	*~0.5 kgCO_2e/kg*	*Ex Nauhria reinforcing, HUB-3525*
Tung Ho (Vietnam)	*Not reported*	*~1.0 kgCO_2e/kg*	*EPD-IES-0004681:001 [14]*
Hyundai Steel (Korea)	*>93%*	*~0.5 kgCO_2e/kg*	*EPD-IES-0020903:002 [14]*
Generic supply (Europe)	*>40%*	*~2.4 kgCO_2e/kg*	*NZ compliance to be checked for specific suppliers [15]*
Celsa (Spain)	*90%*	*~0.4 kgCO_2e/kg*	*EPD-IES-0011656:001 [14]*

(1) Refer to Section 4.1 for information about the benefits and limitations of steel recycling.
(2) GWP values are as at the time of publication and are subject to change with future EPD updates.

Reinforcing steel suppliers may be able to offer project-specific emissions reduction options e.g. lower carbon products, or coupler systems to reduce reinforcing steel quantities and congestion.

3.2. In-situ concrete

Masterspec 3101, 3111, 3121, 3129]

3.2.1 To reduce embodied carbon

- Reduce concrete volume via design optimisation, scope reduction, substitution with low-carbon alternatives where appropriate, or through adaptive re-use of existing structures.
- Specify the lowest strength of concrete that meets the structural requirements for strength and durability. Replacement of concrete with concrete block masonry can help to achieve this.
- Use low embodied carbon binders to replace regular cement in the concrete mix. This is best achieved by setting a maximum average A1-A3 GWP limit for each concrete specification category that is defined in the project specifications (e.g. by concrete grade and/or application as appropriate). This allows the concrete mix designer to select the right blend of low-carbon binders to meet the project requirements in the most cost-effective way, taking account of factors like availability, concrete placement and curing requirements, regional aggregate properties and curing conditions, production techniques and transport distances.

> *Specifiers should refrain from overspecification of concrete with GWP limits. 28-day (or 56-day) strength must be specified, but slump and binder content should generally be selected by the mix designer to suit the proposed application and placement methodology.*
>
> *Low embodied carbon binders could include low carbon cement, blended cement, supplementary cementitious materials (SCMs) and natural pozzolans.*

- The mix designer can also reduce the GWP using other techniques, such as optimising mix designs, selecting high quality well-graded aggregates, or designing for 56-day strength.

3.2.2 Benchmarking GWP for concrete

- To provide a consistent reference for relative rating of mix designs, some concrete suppliers have benchmarked their products against the ISC 2020 Baseline published for modules A1-A3 by the Infrastructure Sustainability Council in their Materials Calculator NZ 2.0 [16] and which was largely drawn from Australian market data.
- For Green Star building projects, baseline GWP values are provided by NZGBC Embodied Carbon Methodology 2.0 (Table 9, [3]). The baseline values cover upfront carbon (A1-A3), construction waste (A5) and use stage (B2-B5).

NZGBC baseline values for concrete are lower than ISC 2020 (A1-A3), as illustrated in Figure A.2 (Appendix A). The NZGBC data set was based on a local data set built from BRANZ data, available EPDs and consultation with concrete suppliers, so is considered to give a relatively accurate representation of industry performance for NZ in 2020.

- Benchmarks and baseline values represent average industry performance and are intended as a starting point for measurement of emissions reductions when moving towards net zero.

3.2.3 Recommended A1-A3 GWP limits

- **'SESOC On Track' A1-A3 GWP limits:** SESOC have established A1-A3 GWP limits for concrete grades between 10MPa to 60MPa as shown in Appendix A, which are intended to align with Concrete NZ's roadmap to net zero carbon by 2050 [5]. It is anticipated that these GWP limits will incur little or no additional cost for ready-mix concrete supplied in urban locations (assuming there are no special mix design considerations beyond 28-day strength).
- **'SESOC LC' A1-A3 GWP limits:** SESOC has established a "Lower carbon" A1-A3 GWP limit for concrete as shown in Appendix B. This represents a higher level of performance in concrete production for reduced emissions and is expected to be available from most urban ready-mix suppliers subject to appropriate lead times and cost premium (a suggested rule of thumb is 2% cost premium for every 10% reduction in GWP).

> *The SESOC LC specification is not recommended for large-area slabs-on-grade that require a highly worked surface finish (such as warehouse floors), particularly where rapid drying or early thermal contraction may occur, because the use of SCMs can make placement and curing more challenging. The specifier will need to liaise with their local concrete supplier(s) to develop an appropriate GWP specification for large slabs, suitable for the aggregate types and seasonal curing conditions in their region. Refer to guidance in Appendix C.*

Further reductions in GWP are possible, beyond those defined by the SESOC LC limits. To achieve best practice in emissions reduction, project-specific optimisation and discussion with suppliers is recommended.

3.2.4 SCM impact on placement and curing

Impacts on concrete placement vary for different SCMs, but the overall tendency is to reduce the amount of free water rising to the surface of the concrete. Increasing SCM dosage will tend to slow down the bleed time, set time, and early strength gain. It is important for designers and contractors to understand that the pour size, placement/finishing methods, and curing requirements may need to be adjusted to suit the mix design with higher SCM content. Refer to Appendix C for further guidance.

> *SCMs are sometimes selected to achieve benefits other than emissions reductions. For example, fly ash can be used to reduce heat of hydration, long-term drying shrinkage, permeability and chloride penetration.*

3.2.5 56-day strength

Supplementary cementitious materials (SCMs) undergo secondary hydration reactions that continue beyond initial cement hydration. As a result, low-carbon concrete typically gains strength more slowly than Portland cement–only mixes. NZS 3104 permits the specification of 56-day concrete strength, which may be used to limit heat of hydration or reduce embodied carbon.

Specifying 56-day strength allows the mix designer to reduce cement content, lowering GWP by approximately 5–10 $kgCO_2e/m^3$ compared with a 28-day strength specification [17]. Designers should carefully consider where 56-day strength concrete is used, taking account of potential performance impacts associated with high SCM content (refer to guidance in Appendix C).

> *In the current market, specifying 56-day strength may achieve GWP reductions in the order of 2–5% for structural concrete. This modest benefit may not justify widespread use due to extended curing and formwork stripping times, and the need for non-standard strength testing timeframes. However, designers and suppliers should remain alert to suitable applications, such as large-volume pours where any construction programme impacts can be managed.*

3.2.6 Fibre-reinforced slabs

Fibre-reinforced slabs require more cement, which increases the GWP of the concrete mix. Industry advice indicates that SCM replacement should not exceed 15% of the cement content.

3.3. Precast concrete

[Masterspec 3130, 3131, 3141]

3.3.1 To reduce embodied carbon

Use low embodied carbon binders to replace regular cement in the concrete mix, for example:

- **GP cement:** low carbon cements can be produced by decarbonising the manufacturing process, and high early strengths can be achieved by milling the cement for longer to create a finer product.
- **SCM binders:** additives can be used to increase early strength if SCMs are used in significant quantities, although this may attract a cost premium. Additional water curing after accelerated curing may be required to achieve the desired long-term strength and/or durability performance

> *International studies have found that precast concrete can be produced with lower embodied carbon than in-situ concrete, due to the benefits of optimising concrete mixes and reducing construction waste [18]. This is yet to be demonstrated in the NZ context.*

3.3.2 Recommended A1-A3 GWP limits

Refer to the A1-A3 GWP limits in Appendix A or B.

3.3.3 Fabrication-related emissions

It is anticipated that EPD information will not be readily available for fabrication processes at the precast yard (e.g., heating/curing, formwork, materials transport to the workshop). For the purpose of this guide, operational emissions from domestic fabrication workshops may be excluded from the specified A1-A3 GWP limits. Although excluded from the proposed materials specifications, these emissions must still be accounted for in any LCA.

3.3.4 Strength considerations

Historically, precast concrete has been specified with relatively high strength (e.g., 40 MPa+) and cured using heated enclosures to achieve the high early strength needed for demoulding and handling (e.g., ~25MPa within 1 day) [19]. The use of high strength concrete will tend to increase GWP, which can be offset through the use of low carbon cements and/or SCMs. (SCM concrete may require additional moist curing after the accelerated curing is complete, and special additives to increase early strength).

Further reductions in GWP may be achieved by specifying lower concrete strengths. In this case, longer curing times and wet curing methods would be required. This approach could be considered for tilt-up construction if the programme can accommodate the time needed to achieve sufficient strength for lifting.

3.3.5 Durability considerations

- Normal concrete: common historical practices within the precast industry – developed through experience rather than strict adherence to NZS 3109 or NZS 3101 – often differ from the prescriptive curing requirements in these standards. Although accelerated curing generally has a detrimental effect on durability, collective views from the industry are that curing methods should be selected on a fit-for-purpose basis. In many situations, the use of relatively high-strength concrete placed under controlled factory conditions will achieve the required performance. However, additional curing may be necessary depending on the drying environment after casting and the exposure classification of the final application [19].

- SCM concrete: the detrimental durability effects of accelerated curing are typically more pronounced for concretes incorporating supplementary cementitious materials (SCMs) than for conventional Portland cement concrete [20, C3.6]. These risks should be assessed by the concrete mix designer and may be mitigated by providing further moist curing after the initial accelerated curing process [19].

Until further New Zealand-specific guidance is developed, relevant international standards may be consulted for recommended curing timeframes. Increasing the specified concrete cover may also be considered as a method to enhance durability.

3.3.6 Precast prestressed flooring systems

[Masterspec 3151]

- **Recommended A1-A3 GWP limits:** determine concrete strength from product literature (typically 40-45MPa) then refer to the A1-A3 GWP limits in Appendix A or B.

> ***Some suppliers offer specific precast flooring products with reduced embodied carbon (e.g., Stresscrete / Formstress offer concrete with 30% less GWP than the ISC 2020 benchmark for no additional cost from their plants in Tāmaki Makaurau Auckland (Waiuku) and Te Whanganui-a-Tara Wellington (Ōtaki) [21]). This corresponds to roughly 10% less GWP than the NZGBC baseline for 2020.***

- **Fabrication emissions:** it is anticipated that EPD information will not be readily available for fabrication processes at the precast yard in the short term, so it is recommended that fabrication-related emissions are excluded from A1-A3 GWP limits as an interim measure.
- **Strand:** there are several pre-stressing strand manufacturers who use recycled steel content in the international market. Designers would need to consult with precast suppliers to determine the feasibility of sourcing these products in New Zealand.

3.4. Concrete block masonry

[Masterspec 3320, 3321]

3.4.1 Recommended A1-A3 GWP limits

- Specify an A1-A3 GWP limit of 120 kgCO2e/m3 for block masonry units and mortar.
- Specify A1-A3 GWP limits for grout infill as for in-situ concrete (refer to Appendix A).
- Specify A1-A3 GWP limits for reinforcing steel as per Section 3.1.

3.4.2 Concrete block masonry

- Tends to use lower volumes of cement than in-situ concrete of the same density because a low-cement mix is used, relying on vibration to produce a well compacted block.
- Table 3.2 shows data from BRANZ LCA Quick for block masonry products, which provides the basis for the A1-A3 GWP limit recommended above. Note that a reinforced block masonry wall would have roughly 80% of the embodied carbon of a similar reinforced concrete wall across LCA stages A1-A5 (the difference between the wall types is greater across lifecycle stages A1-A3, but in this case the A4-A5 component allows a more holistic comparison for decision-making purposes).

Table 3.2: Comparison of Global Warming Potential for block masonry and in-situ concrete [22]

Material description	A1-A3 GWP $kgCO_2e/m^3$	A4-A5 GWP $kgCO_2e/m^3$	A1-A5 GWP $kgCO_2e/m^3$
Comparison of component materials			
Concrete blocks, 17.5MPa	111	39	150
Grout masonry infill, 22 MPa	265	243	508
Concrete, 17.5 MPa, in-situ, no reinforcement, (OPC)	256	19	275
Comparison of reinforced wall build-ups			
Masonry wall, incl. concrete block 20 series (17.5MPa OPC), grouted 22MPa (OPC), inc. steel reinforcing	277	151	428
Reinforced concrete, 25 MPa, in-situ, inc. 50 kg/m^3 steel reinforcing, (OPC)	488	31	519

***Supplier commentary:** at the time of publication, only one domestic supplier (Firth) has EPDs for masonry block products in New Zealand, although this covers more than half of the market.*

3.4.3 Grout fill

- **Water/cement demand:** blockfill mixes have higher water demand than in-situ concrete, partly to achieve workability for pumping, and partly because the smaller aggregates require more cement paste to fill voids and coat particles. Higher water demand will often lead to higher cement content and GWP. Designers should consider specifying larger aggregate sizes where possible.

Supplier commentary: *7mm aggregate is typically made using quarrying by-product, which makes it a cost-effective option, but requires more cement/binder than larger aggregate sizes, increasing GWP. 10mm or 13mm blockfill will reduce GWP but requires a larger pump (availability varies with region).*

- **SCMs:** a high level of cement replacement is possible within block masonry grout (e.g. up to 40% fly ash is accepted in ASTM C476, and higher values are possible [23]). This is partly because the rate of strength gain is less critical than it is for in-situ concrete. It is not unusual for moderate quantities of fly ash to be used in NZ blockfill mixes, provided the supplier has a silo available for storage and metered dispensing of fly ash.

4. STEELWORK

Refer to:

- HERA Design Guide: How to specify low carbon structural steel. [24]
- SESOC journal article: Reducing steel's carbon impact: industrial decarbonization and specification considerations for structural engineers. [25]

4.1. Structural steelwork

[Masterspec 3410, 3411, 3411S, 3419, 3419S]

4.1.1 To reduce embodied carbon:

- **Reduce steel weight:** via design optimisation and substitution of steel products with low-carbon alternatives where appropriate, or through adaptive re-use of existing structures. High strength steel products (e.g., G350) can also reduce overall material quantities by enabling more efficient section sizes.

  The draft "Roadmap to net-zero greenhouse gas emissions for Aotearoa New Zealand's steel industry" (Thinkstep, 2025) relies on these approaches to achieve a 5% per-capita reduction in steel use by 2030, and proposes a 30% reduction by 2050. [6]

Guidance on rationalisation vs optimisation for structural steel design is provided by the IStructE in a paper on lean design [26].

Guidance on low-carbon design for steel and hybrid low rise buildings is available in HERA Report R4-166 [27]. Guidance on circular design and adaptive re-use is available in HERA Report R4-164 [28].

- **Specify A1-A3 GWP limits:** guidance on GWP limits for various structural steel products is provided in Appendix D, which reference the HERA design guide How to specify low carbon structural steel [23] and NZGBC benchmarks [3]. Reductions in embodied carbon are predominantly achieved by including recycled steel in products.

4.1.2 Steel recycling

Recycled steel typically has 40-60% less embodied carbon than new steel (greater reductions are possible with renewable electricity supply and high scrap content). [29]

- **New Zealand context:** the domestic construction and infrastructure sectors achieve an estimated scrap steel recovery rate of approximately 85% (272,000 tonnes recovered from 322,000 tonnes of scrap per year) [30]. From 2026, the commissioning of an electric arc furnace (EAF) at Glenbrook will enable increased domestic recycling of scrap steel, reducing the need for export of our scrap steel and supporting local low-carbon steel production. NZ Steel intends to continue the local manufacture of steel plate, steel coil, wire rod, and reinforcing bar.
- **Limits of recycling:** globally, only about 30% of current steel demand can be met using recycled content, due to limitations in scrap availability. This is projected to increase to approximately 45% by 2050, but will not be enough to meet the projected demand [31]. As a result, specifying high recycled content for individual projects does not necessarily lead to a reduction in global steel production or emissions—it may simply shift the demand for virgin steel elsewhere. Therefore, the most effective strategy for designers is to minimise the total quantity of steel specified in a project [32]. This strategy also applies to other structural materials.
- **ResponsibleSteel:** is a certification scheme that promotes the production of low-emissions steel. It encourages steelmakers to increase recycled content where feasible, but more importantly, it supports systemic decarbonisation of the global steel industry. The scheme aims to avoid a competitive rush for limited scrap resources, which could result in minimal or no net carbon reductions on a global scale. [33]

***ResponsibleSteel** has four Decarbonisation Progress Levels for certification for steel products. At least 50% of global steel production has an emissions intensity level that is certifiable to at least Progress Level 1 [34], although only one mill has been formally certified at the date of publication (Big River Steel, Osceola mill, Arkansas, USA – Progress Level 1) [35]. ResponsibleSteel also operate an ESG certification scheme for steelmaking sites (refer to Section 6 – Beyond Carbon).*

***SteelZero,** led by the Climate Group in partnership with ResponsibleSteel, is a global initiative working to speed up the transition to a net-zero steel industry. While ResponsibleSteel provides a decarbonisation pathway for steelmakers, SteelZero mobilises steel users, specifiers & procurers to amplify the voice of the demand side [36].*

4.1.3 Alternative option: specify 'Lower Emission Steel'

IStructE suggests the following example specification [37]:

> *"A minimum of [X%] by mass of the structural steel used on the project shall be from steel products that meet one or more of the following criteria:*
> - *ResponsibleSteel Certified Steel, or steel meeting an equivalent international standard.*
> - *Steel meeting the ResponsibleSteel Decarbonisation Progress threshold for "Lower Emission Steel", or equivalent*
> - *Steel produced by a steelmaking site where the site's corporate owner has defined and made public both a long-term and a near-term emissions reduction target, validated by the Science-based Targets initiative (SBTi) or another quantitative, scientifically justified target of comparable ambition, quality and coverage."*

SESOC has updated the example specification to match the wording of Steelzero's current Commitment Framework v1.2 (Dec 2025) [38]

Organisations that join SteelZero make a commitment to procuring, specifying or stocking 100% of their steel requirement by 2050 as net zero steel, with an interim commitment to procure, specify or stock Lower Emission Steel for 50% of their steel requirement by 2030 [39]. Lower Emission Steel is defined as steel aligning with ResponsibleSteel Decarbonisation Progress Level 2 [38].

Ambitious NZ project teams may wish to start with specifying 10% "lower emission" structural steel in 2026, with the aim of initiating supply chain pathways that could grow towards the SteelZero target of 50% by 2030.

Alternatively, steel meeting the emissions requirements for ResponsibleSteel Decarbonisation Progress Level 1 could be specified in higher proportions (e.g. 50%).

4.1.4 Steel re-use

Refer to HERA Report R4-164 "Low-carbon design guidance framework". [28]

- **Re-use of buildings:** most re-use of steelwork occurs through refurbishment, retrofit or adaptive re-use of existing buildings where the structural steelwork remains in place. There is currently no centralised database for steelwork that has been disassembled and is available for reuse in New Zealand.
- **Circularity passport:** HERA are developing a steel circularity passport system for tracking steelwork. Once launched, this system could be specified to encourage future reuse of steelwork. [40]

4.1.5 Offsetting

HERA manages two certification schemes for offsetting of carbon emissions for steel, [41]:

- Carbon Conscious Steel certification (for offsetting >30% of embodied carbon)
- Zero Carbon steel certification (for 100% offsetting of embodied carbon).

Note: offsetting cannot be used to meet Green Star certification thresholds for upfront (A1-A3) embodied carbon or lifecycle impacts.

4.2. Light gauge steel

4.2.1 Specification guidance

A1-A3 GWP limits for light gauge steel should be specified using the same methodology as for structural steel (refer to Appendix D). In cases where an EPD is not yet available for the specified cold-formed product, the A1-A3 GWP limit may be applied to the hot-rolled coil manufacturing stage only.

4.2.2 Materials supply

NZ Steel supplies the hot-rolled steel coil used in most light gauge steel products that are manufactured in Aotearoa New Zealand. With the planned commissioning of an electric arc furnace (EAF) at Glenbrook in 2026, designers will have the opportunity to specify lower-GWP steel from local sources. Steel coils are roll-formed by local manufacturers into components such as purlins and girts, stud and track framing, profiled cladding and suspended ceiling systems. Light gauge steel is also imported, often with limited transparency on embodied carbon. Imported products may vary significantly in A1-A3 GWP due to differing production practices.

4.3. Protective coatings

4.3.1 Specification guidance

Exclude protective coatings and galvanising from the A1-A3 GWP limits for steelwork. This does not exempt protective coatings from inclusion in any LCA that is undertaken.

4.3.2 Galvanising

There is a lack of data available for hot dip galvanising processes in New Zealand. An Environmental Product Declaration (EPD) published by the American Galvanizers Association reports that galvanising typically contributes 10–12% of the total GWP for hot-rolled sections and plate, reducing to around 6% for hollow sections [42]. Similarly, a study by the European General Galvanizers Association found that galvanising processes contribute approximately 5% of the total embodied carbon of galvanised steel products [43].

4.3.3 Intumescent coatings

The Steel Construction Institute (SCI P449) notes that intumescent coating for typical beams can be approximately 10% of the steel's embodied carbon (when measured relative on an average Module A1 GWP value of 1.64 kgCO2e/kg for rolled steel sections in the UK) [44]. This percentage can vary widely.

5. TIMBER

Refer to:

- NZ Wood Design Guides: Chapter 2.1 Timber, Carbon, and the Environment [45]
- Refer to SESOC journal article: Understanding carbon emissions of structural timber products: the full story [46]

5.1. Timber framing

[Masterspec 3811, 3813, 3821]

5.1.1 Recommended A1-A3 GWP limits:

Specify A1-A3 GWP limits for fossil emissions (GWPF), with reference to WMPA and manufacturer EPDs (see Table 5.1).

> ***The WMPA EPD for "Solid, Finger-Jointed and Laminated Timber Products including timber preservation options" (2019) [12] covers most of the timber framing produced in New Zealand. The EPD expired in October 2024 and appears unlikely to be updated as an industry-wide document. Various timber suppliers have prepared EPDs to cover their specific product range.***

Table 5.1: A1-A3 GWP for NZ timber framing products

Product	Treatment	Approx A1-A3 GWPB (biogenic)	Approx A1-A3 GWPF (fossil)	Comments
Surfaced, kiln-dried radiata pine	*H1.2*	*-790 kgCO_2e/m^3*	*75 kgCO_2e/m^3*	*WMPA EPD (2019) [12]*
	H3.1/H3.2	*-790 kgCO_2e/m^3*	*120 kgCO_2e/m^3*	
	H4	*-790 kgCO_2e/m^3*	*95 kgCO_2e/m^3*	
Glulam	*H1.2*	*-800 kgCO_2e/m^3*	*145 kgCO_2e/m^3*	*WMPA EPD (2019) [12]*
	H3.1/H3.2	*-800 kgCO_2e/m^3*	*210 kgCO_2e/m^3*	
	H4	*-800 kgCO_2e/m^3*	*175 kgCO_2e/m^3*	
Laminated Veneer Lumber (LVL) radiata pine (NZ generic)	*Untreated*	*-910 kgCO_2e/m^3*	*195 kgCO_2e/m^3*	*Scion Research report 2010 [47]*
	Treated	*-910 kgCO_2e/m^3*	*300 kgCO_2e/m^3*	
Futurebuild LVL	*Untreated*	*-920 kgCO_2e/m^3*	*95 kgCO_2e/m^3*	*Carter Holt Harvey (CHH) EPD (2023) [12]*
	H1.2	*-920 kgCO_2e/m^3*	*105 kgCO_2e/m^3*	
Nelson Pine LVL	*Untreated*	*-1090 kgCO_2e/m^3*	*305 kgCO_2e/m^3*	*Nelson Pine EPD [12]*

5.1.2 Sustainable sourcing of timber

According to the international standards for EPDs, if the biogenic carbon sequestered in the timber product is to be reported in A1-A3, the timber must be able to demonstrate its sustainability, for example through FSC or Programme for the Endorsement of Forest Certification (PEFC) certification or equivalent, or using timber that is sourced from countries that account for Article 3.4 of the Kyoto Protocol and report increasing forest carbon pools. [48]

> *New Zealand chose not to account for land-use, land-use change and forestry (LULCF) activities under Article 3.4, which means that timber from New Zealand would require FSC or PEFC certification for carbon removals to be included in A1-A3 when carrying out an LCA in accordance with EN 15804. Approximately 60-70% of locally grown timber is PEFC or FSC certified [45], particularly timber sourced from the large central North Island forests.*
>
> *Supplier commentary: FSC certification requires full chain-of-custody records and is only available from selected timber yards. 30-40% of NZ timber comes from small-scale mills which are generally not set up for chain-of-custody records at present. For residential projects and/or projects away from urban centres, designers may prefer to specify Controlled Wood if there are no specific LCA reporting requirements. Controlled Wood is not FSC-certified and does not require chain-of-custody records but has been assessed to ensure it does not originate from specific unacceptable sources [49].*
>
> *FSC rules allow Controlled Wood to be incorporated into products labelled "FSC-Mix". Suppliers estimate that Controlled Wood covers roughly 90% of the market supply in NZ*.

5.1.3 End-of-life considerations for timber

Although this guide focuses on the "upfront" embodied carbon emissions associated with LCA modules A1-A3, the following commentary on whole-of-life emissions might be useful. While timber offers substantial up-front carbon benefits, its treatment at end-of-life can significantly influence whole-of-life emissions in an LCA.

- **Likely end-of-life scenario (New Zealand context):** landfill disposal is currently the predominant end-of-life pathway for treated structural timber in New Zealand. If complete biodegradation were to occur (producing biogenic methane and CO2) the resulting global warming potential (GWP) would exceed the climate benefit achieved through initial carbon sequestration. However, empirical studies show that the degradation of treated structural timber in landfill conditions occurs at a rate significantly slower than the 100-year assessment horizon typically applied in carbon accounting for Environmental Product Declarations (EPDs). As a result, the biogenic carbon contained in structural timber can be considered effectively sequestered over the long term – whether the material remains in use within the built environment or is ultimately disposed of in landfill. To support long-term carbon storage, designers are encouraged to design for building longevity by considering durability, maintenance needs and future functionality including energy efficiency. [46]
- **End-of -life scenario in LCA calculations:** under EN 15804+A2 (effective since 2022), EPDs for timber must assume all sequestered carbon is released at end-of-life (Stage C). Designers should be aware of the following:
  - ☐ IStructE publication "How to Calculate Embodied Carbon" (version 3) was published in January 2025 and provides new guidelines for how to account for end-of-life emissions from timber when preparing an LCA.[50]
  - ☐ The MBIE Whole-of-Life Embodied Carbon Assessment Technical Methodology requires that stored biogenic carbon be reported separately alongside emissions from Stages A1–D in embodied carbon assessments. This allows the likely benefit of long-term sequestration to be considered.
  - ☐ The MBIE ECO2ALP embodied carbon database, currently in trial, follows this methodology. Tools such as BRANZ LCAQuick can be used to generate reports aligned with this approach.

5.2. Mass Timber

[Masterspec 3813]

5.2.1 Recommended A1-A3 GWP limits

Specify A1-A3 GWP limits for mass timber with reference to relevant industry EPDs (see Table 5.2). The energy source used for manufacturing can have a significant impact on the embodied carbon, and this is reflected in the EPD figures for lifecycle stage A1-A3.

Table 5.2: A1-A3 GWP for common mass timber products supplied to New Zealand

CLT manufacturer	Treatment	Approx A1-A3 GWPB (biogenic)	Approx A1-A3 GWPF (fossil)	Comments
RedStag	Untreated	-780 kgCO_2e/m^3	70 kgCO_2e/m^3	From EPD (2022) [12]
	H1.2	-780 kgCO_2e/m^3	75 kgCO_2e/m^3	
	H3.1/H3	-780 kgCO_2e/m^3	135 kgCO_2e/m^3	
	H4	-780 kgCO_2e/m^3	105 kgCO_2e/m^3	
XLam	Untreated	-740 kgCO_2e/m^3	250 kgCO_2e/m^3	From EPD (2021) [10]
	T3 plus	-740 kgCO_2e/m^3	265 kgCO_2e/m^3	

Note: For glulam and LVL, refer to Section 5.1 above.

Table 5.3: A1-A3 GWP for common mass timber products supplied to New Zealand

Product	Treatment	Approx A1-A3 GWPB (biogenic)	Approx A1-A3 GWPF (fossil)	Comments
CHH Ecoply	H3.1 LOSP	-950 kgCO_2e/m^3	255 kgCO_2e/m^3	From CHH EPD (2023) [12]
	H3 CCA	-950 kgCO_2e/m^3	235 kgCO_2e/m^3	From CHH EPD (2023) [12]
Australian plywood 9mm exterior ply (structural)	Exterior	-770 kgCO_2e/m^3	425 kgCO_2e/m^3	Wood Solutions EPD (expired 2022) [10]
Australian plywood 25mm plywood flooring	Interior	-770 kgCO_2e/m^3	435 kgCO_2e/m^3	Wood Solutions EPD (expired 2022) [10]

5.2.2 Life-cycle scope

It is suggested that embodied carbon is specified for life cycle Stages A1-A3 (material production), which excludes emissions associated with transport of imported CLT products to New Zealand. To give an idea of the relative influence of transport emissions, the XLam EPD (2021) [10] estimates transport emissions of approximately 25 kgCO_2e/m^3 for transport from CLT from their plant in Wodonga (Victoria) to Auckland.

5.3. Plywood

5.3.1 Recommended A1-A3 GWP limits

Specify A1-A3 GWP limit for plywood with reference to relevant industry EPDs (e.g. refer to Table 5.3). It is anticipated that there will be a wide range of embodied carbon footprints for imported products.

6. CONSTRUCTION PHASE SUBMISSION REQUIREMENTS

6.1. Specification guidance

Specify documentation requirements for GWP-limited materials, allowing for a range of documentation types that are appropriate to the scale of the project. The following example is proposed for commercial scale projects:

Documentation requirements for GWP-limited materials

To demonstrate compliance with A1-A3 GWP limits in this specification, submit the following documentation for specific products (or sector average products) from the specific country of manufacture:

a) Verified Type III EPD to ISO 14025 and EN 15804; or

b) Verified CFP (PCF) to ISO 14067 or PAS 2050, with independent third-party verification to a reasonable level of assurance; or

c) EPD/CFP prepared using a pre-verified LCA or EPD tool that is approved by EPD International.

Alternative documentation

The following may be used only where documentation listed above is unavailable:

i. Type II environmental declaration to ISO 14021, confirming compliance with the specified A1-A3 GWP limits.

ii. EPD/CFP for a similar product from the specific country of manufacture,

iii. EPD/CFP that has expired within the last 5 years,

For alternative documentation types ii and iii, the A1-A3 GWP values must be multiplied by a conservative value conversion factor of 1.2;

Evolution of documentation requirements
The demand for environmental verification documents is still in its growth phase in the New Zealand construction industry. In the medium-term future, the intention is to phase out the alternative documentation pathways that are currently proposed. When choosing the level of documentation required, specifiers should consider a range of factors, e.g.:

- ***Cost:*** *recognising the considerable expense of preparing EPD documentation.*
- ***Availability:*** *meeting the market where it is at, so the specification is achievable and allows competitive pricing.*
- ***Continuous improvement:*** *incentivising suppliers to invest in verified environmental documentation. The verification process is often a valuable tool for improving the efficiency of a supplier's operations, as well as improving environmental performance.*

Rationale for specifying high quality documentation
The level of data quality required by the suggested specification clauses is at the high end of the scale (for example, in the hierarchy defined by Table 2 in the NZGBC Embodied Carbon Methodology [3]. The rationale for this is:

- ***Quantifiability:*** *the A1-A3 lifecycle stages are usually easier to quantify than other life cycle stages, making it feasible to achieve higher levels of certainty.*
- ***Fairness:*** *the GWP information has the potential to impact procurement decisions, so an evidential basis for product claims is warranted.*
- ***Simplicity:*** *third-party verification reduces the level of judgement required for construction reviewers such as structural engineers and architects. If the project submissions will be reviewed by an experienced LCA practitioner (for example, on a Green Star project), it may be appropriate to relax the documentation requirements based on their guidance.*

Guidance on EPDs – for engineers
Guidance on how to distinguish and compare between higher quality and lower quality EPDs is provided in the SESOC Journal article "Demystifying EPDs, PCRs and other TLAs" [51]

Conservative value conversion factor
This is based on Finland's national building construction emissions database methodology, where a conversion factor of 1.2 must be applied to generic or typical GWP data to allow for the average level of variance across different product groups. This avoids an overly optimistic emissions estimate and encourages the creation of new EPD data [52].

6.2. Type II environmental declarations

Type II environmental declarations are self-declared without mandatory third-party certification, and have the following attributes:

- Claims must be substantiated with documented evidence.
- The claim must clearly describe the environmental aspect being asserted (e.g. compliance with specified A1-A3 GWP limits).
- The scope must be clearly defined, whether it refers to the entire product or only a component.

It is envisaged that suppliers could engage a local LCA Certified Practitioner (LCACP) to prepare a statement to serve as a Type II environmental declaration to ISO 14021 for a product, or a range of products. This is intended as a cost-effective entry-level pathway while third-party verification processes are being developed. The statement could be re-used within the validity conditions stated in the documentation (e.g. 5-year time limit, or any significant change in production method).

6.3. Examples of documentation

- **Verified CFP:** a domestic concrete supplier imports low-carbon cement from Taiwan. CFP documentation to ISO 14067 is provided by the cement manufacturer and is included as a reference document in the ready-mix supplier's submission.
- **Pre-verified EPD tool:** a ready-mix concrete supplier finds the administration cost to develop and maintain verified EPDs for their complete range of mix designs is prohibitive, so their design engineer enters the mix design parameters into the pre-verified EPD tool developed by the Global Cement and Concrete Association [53]. Outputs include a self-declaration, background report with input data and results, and a digital EPD (not third-party verified).
- **Type II environmental declaration for timber framing:** a Taranaki timber mill engages a local LCA Certified Practitioner to prepare a statement confirming the A1-A3 GWP for their range of treated structural timber framing. The LCA practitioner refers to EPDs available for similar products in the market (current and expired) and makes appropriate allowance for any differences in their client's manufacturing process. A 5-year validity period and expiry date are included in the statement. Supporting documentation (e.g., EPDs, reports) can be supplied on request.
- **EPD for similar steel product:** a square hollow section has been supplied by a Japanese steel mill which does not have an EPD available for their product. The NZ distributor searches their records and finds an EPD for hollow sections from a

different Japanese mill. The distributor checks that both mills use the same manufacturing process (e.g. hot rolled coil from an electric arc furnace is roll formed and seam welded) and applies a conservative value conversion factor of 1.2 to the A1-A3 GWP values in the EPD, finding that the weighted average A1-A3 GWP for all structural steel on the project is still within the A1-A3 GWP limit that was specified. In this case, the conversion factor allows for some potential variation between the products, such as the percentage of recycled steel which is used by the two suppliers. The distributor submits an email explanation, attaching copies of the two mill certificates and EPD.

7. SUSTAINABILITY BEYOND CARBON

This section provides guidance on specification to achieve sustainability outcomes that are broader than consideration of embodied carbon, recognising the need for:

- Protection of natural resources and ecosystems,
- Social equity and
- Financial and economic responsibility.

7.1. Design and construction management

[Masterspec 1226SP]

7.1.1 Sustainable Construction Practices

Are methods of planning, designing, building, and operating structures in ways that minimise negative environmental impacts, promote resource efficiency, support social well-being, and ensure long-term economic viability. The following references provide a starting point:

- ☐ MBIE Sustainable Construction Procurement Guidelines [54]
- ☐ NZGBC Frameworks – refer Section 7.3
- ☐ NZCIC Guidelines – updated design guidelines to include ESD inputs [55]

7.1.2 Environmental Management Planning

Environmental Management Plans can be specified for construction works. These generally include:

1. Project overview & environmental assessment
2. Roles & responsibilities
3. Mitigation measures and environmental controls
4. Regulatory compliance requirements
5. Monitoring & reporting systems
6. Emergency response procedures
7. Training and inductions
8. Community engagement
9. Continuous improvement and review

7.2. Waste management

[Masterspec 1256]

7.2.1 Resource Efficiency in the Building and Related Industries (REBRI)

This programme provides practical guidance to minimise construction and demolition waste across New Zealand. It offers a toolbox that helps builders plan for waste reduction, assess waste streams and manage materials so they are reused, recycled, or recovered instead of going to landfill. REBRI promotes resource efficient practices that reduce disposal costs, save on raw materials and lower environmental impacts. It is structured around the waste hierarchy: reducing waste at source, reusing materials for as long as possible and recycling when reuse is not feasible [56].

- **REBRI guidance** is maintained and updated by BRANZ, with support from the Ministry for the Environment. Guidance was most recently refreshed in 2024. All REBRI guides, plans, templates, and on-site signage kits are published and maintained on the BRANZ website [56].
- **Specifications:** REBRI guidance is commonly referenced within technical specifications, such as the Masterspec templates for waste management [1256], LBC projects [1226LB], and sustainable projects [1226SP].

7.3. Sustainable building certifications

[Masterspec 1226GB, 1226LB]

7.3.1 New Zealand Green Building Council (NZGBC)

Offers several certification programmes that promote healthier, low carbon, and sustainable buildings across New Zealand [57]. Key tools include:

- **Homestar,** which rates new homes on health, efficiency, and sustainability;
- **Green Star**, which assesses commercial buildings, communities, and interiors; and
- **NABERSNZ,** which measures the operational energy performance of existing buildings.
- **Net Zero Building certification,** which provides a carbon neutral standard for building operations [58].

These programmes support good practice design, reduced emissions, and improved building quality, and they are widely adopted across the sector. NZGBC also provides training, advocacy, and resources to accelerate sustainable construction and align the industry with national carbon reduction goals.

- **NZGBC guide on up-front carbon:** refer to “A practical guide to upfront carbon reductions” [59].
- **Specification guidance:** refer to SESOC journal article ‘Demystifying Green Star and Homestar for Structural Engineers’ [60]. Specifiers should be aware of the following requirements:

- ☐ Reference buildings: all Green Star buildings must demonstrate a minimum 15% reduction in GWP compared with a reference building. When setting the reference building, refer to the appendix in the NZGBC Embodied Carbon Methodology for GWP baseline values for materials [3].
- ☐ Responsible Product Values (RPV): Green Star credits can be earned by specifying products with Responsible Product Value. This is a score assigned to a product that has been assessed against the Responsible Products Guidelines which consider sustainability, health, and transparency.

7.3.2 The Living Building Challenge (LBC)

The LBC is one of the world's most rigorous regenerative building certification programmes, requiring real-world, 12-month verified performance. It evaluates projects across seven "Petals": Place, Water, Energy, Health + Happiness, Materials, Equity, and Beauty—each addressing a core aspect of sustainability, from net-positive water and energy to toxin-free materials, social justice, and ecological regeneration. Buildings must operate within their site's resource limits, enhance human well-being, avoid harmful substances, and contribute positively to communities and ecosystems [61].

- **Specification guidance:** strategies used by successful project teams typically include [62]:
  - ☐ Whole life-cycle carbon assessment (LCA) to evaluate material impacts and select low-emission construction assemblies
  - ☐ Ecological performance benchmarking, using tools such as the Society for Ecological Restoration's Five-Star Method to guide habitat restoration
  - ☐ Water balance modelling and on-site reuse strategies to meet net positive water requirements
  - ☐ Operational energy modelling validated through real-time metering of building energy systems
  - ☐ Red List-compliant procurement, with detailed tracking of ingredients declarations for all materials.

7.4. Concrete

7.4.1 Recycled aggregate

To reduce waste and disturbance to the environment, consider the specification of recycled aggregate. Background information and a model specification is provided in 'TR14 Best Practice Guide for the use of Recycled Aggregates in New Zealand' [63].

- **Coarse recycled aggregate:** the model specification in TR14 includes guidance on replacement rates as shown in Table 7.1 below.
- **Recycled fine aggregate:** can be used in non-structural concrete with strength not exceeding 17.5MPa, but should not be used in structural concrete [63].
- **Manufactured sand:** is a by-product generated by the crushing of rock aggregates in quarries, which is sometimes specified to promote circularity and reuse of materials (for example, in NSW, Australia [64]). Most concrete plants in NZ already use manufactured sand (PAP) in their concrete mixes, which suggests there is little to be gained from specifying this in an New Zealand context.
- **Non-structural concrete:** we recommend that designers consider specifying a minimum of 20-30% recycled coarse aggregate for all non-structural concrete in urban areas, particularly in the Auckland region. The use of 10% recycled fine aggregate could also be considered. At these levels of replacement, the mix design should not require an increase in water or cement/binder proportions [63] (noting that additional cement would increase GWP). Greater use of recycled aggregate is likely to increase the carbon footprint.
- **Structural concrete:** consult with concrete suppliers to confirm a practical specification, based on the availability, cost, and performance of local recycled aggregates.

Recycled aggregates are becoming increasingly economical in Auckland due to inaccessibility of aggregates from the urban area. In other areas such as Ōtautahi Christchurch, the close accessibility of river run

Table 7.1: Maximum recycled coarse aggregate replacement rates

Specified compressive strength	Percentage on Coarse Aggregate Fraction		
Recycled aggregate type	**Leftover concrete aggregate (LCAgg)**	**Recycled aggregate (RA)**	**Recycled concrete aggregate (RCA)**
Non-structural concrete (up to 17.5MPa)	100%	100%	100%
17.5MPa structural concrete	100%	-	100%
20MPa structural concrete	100%	-	50%
25-35MPa structural concrete	40%	-	30%
40-50MPa structural concrete	15%	-	10%

gravel means that cost of producing natural aggregates is low compared with cost of recycled aggregate processing [63].

7.4.2 Biodiversity offset and compensation

A few quarries in New Zealand have independently reviewed biodiversity offset/compensation programmes to deliver net ecological gains, particularly in the Auckland region. Opportunities to source aggregate from these quarries can be discussed with concrete suppliers.

7.5. Steel

7.5.1 The Sustainable Steel Council NZ (SSC)

The SSC is a group of NZ steel industry bodies and companies that are committed to the circular economy and New Zealand's low-emissions future, and the important role that steel can play in this. SSC offers member certification with three progress tiers (Bronze, Silver, Gold), which are audited against a range of sustainability criteria such as waste management, carbon emissions, health and safety, employee welfare, social impacts, governance, responsible sourcing, and alignment with the UN Sustainable Development Goals. SSC also provide certification for two audit schemes:

- **Responsible Product Certification:** qualifies a steel supplier for Responsible Product Values under the Green Star Buildings scheme operated by the NZBGC.
- **Sustainable Practices Audit:** provides a demonstration of sustainability credentials, and acts as a staircase towards future Responsible Products Certification [65].

For clients with strong sustainability goals, specifiers are encouraged to discuss the option of specifying an SSC certification requirement for steel distributors and/or fabricators.

7.5.2 ResponsibleSteel:

ResponsibleSteel offers two certification pathways for steel manufacturers: **Core Site Certification and Certified Steel** [66].

- **Core Site Certification** is the first step and assesses sites against more than 300 requirements covering key environmental, social, and governance (ESG) topics such as pollution, biodiversity, labour rights, water stewardship, and community impacts. Achieving Core Site Certification requires corporate-level commitment and full-site alignment.
- **Certified Steel** builds on this foundation by requiring sites to meet Progress Level criteria for both decarbonisation and responsible materials sourcing. Sites that reach at least Progress Level 1 for both areas can market their output as "Certified Steel." These Progress Levels reward continuous improvement and drive industry-wide transition toward responsible, near-zero steel production.

*ResponsibleSteel's Progress Report 2025 [35] identified 87 steelmaking sites around the world with **Core Site Certification**, including a handful in Australia and South-East Asia. This represents 7-8% of global production. Interested specifiers should enquire with local steel distributors about the potential availability of steel from mills with Core Site Certification.*

*Only one steelmaking site has gone on to achieve **Certified Steel** status at the time of publication, gaining certification in 2024. Specifiers are encouraged to monitor developments in Certified Steel availability over coming years.*

7.6. Timber

7.6.1 Sustainable forestry management certifications

FSC Certification (Forest Stewardship Council) and PEFC (Programme for the Endorsement of Forest Certification) both certify sustainably managed forests, but they differ in their approach:

- **FSC** uses a top-down system with international standards that are adapted nationally, while
- **PEFC** uses a bottom-up approach where it endorses independent national certification schemes against its international criteria.

The FSC Standard for New Zealand targets areas across the three pillars of sustainability including protecting native forests and guarding biodiversity, living wage requirements, protection of waterways, erosion, and partnership with Māori. [67]

If the designer is going to include a specification clause requiring the timber to be sourced from an FSC certified forest (for example, for Green Star compliance), then the supplier needs to be able to provide a FSC Chain of Custody (CoC) Certification, ensuring that the source of their timber is tracked and verified as either grown in an FSC certified forest, is recycled material, or originates from a controlled source. In addition, all companies in the supply chain must meet the core labour requirements (which include no child labour, no forced labour, no discrimination and no denying the rights of association and collective bargaining)[68].

7.6.2 Certification in the New Zealand context [45]

- **FSC:** around 60-70% of structural wood products made and sold in New Zealand or exported abroad is FSC certified.
- **PEFC:** a small proportion of New Zealand forests are certified with the Programme for the Endorsement of Forest Certification.
- **Demand:** certified timber is more in demand, driven by international standards and other schemes for assessing sustainability, such as Green Star.

- **Uncertified forests:** the remaining 30-40% of forests in New Zealand that are not FSC or PEFC certified are mostly small-holder forests, less than 2000 hectares in size. In the current market these small holders often find that the administrative cost of certification is not commercially justified.
- **FSC labels:** three different products can be specified for FSC timber [69]:
  - ☐ FSC 100%: all materials used come from FSC-certified forests.
  - ☐ FSC Recycled: the product is made from 100% recycled materials.
  - ☐ FSC Mix: the product is made with a mixture of materials from FSC-certified forests, recycled materials, and/or FSC-controlled wood. While Controlled Wood doesn't come from FSC-certified forests, it mitigates the risk of the material originating from unacceptable sources.

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APPENDIX A:

'SESOC On Track' A1-A3 GWP limits for concrete in Aotearoa New Zealand

Table A.1: maximum A1-A3 GWP for concrete to comply with 'SESOC On Track'(1) limits

Grade	ISC 2020 baseline	NZGBC baseline 2020	2026	2027	2028	2029	2030	2031(1)	2032(1)
10 MPa	-	225	205 Z	190 Z	180 Z	165 Z	150 Z	145 Z	140 Z
17.5 MPa	-	235	215 Z	200 Z	190 Z	175 Z	160 Z	150 Z	145 Z
20 MPa	285	250	225 ZP	210 ZP	195 ZP	180 ZP	165 ZP	155 ZP	150 ZP
25 MPa	315	275	245 ZP	230 ZP	215 ZP	200 ZP	185 ZP	175 ZP	165 ZP
30 MPa	345	300	270 ZP	250 ZP	235 ZP	220 ZP	200 ZP	190 ZP	180 ZP
35 MPa	390	325	305 ZP	285 ZP	265 ZP	245 ZP	225 ZP	215 ZP	205 ZP
40 MPa	440	350	345 ZP	320 ZP	300 ZP	275 ZP	255 ZP	240 ZP	230 ZP
45 MPa	495	380	385 ZP	360 ZP	335 ZP	310 ZP	285 ZP	270 ZP	260 ZP
50 MPa	550	410	430 ZP	400 ZP	375 ZP	345 ZP	320 ZP	300 ZP	290 ZP
60 MPa	-	445	475 ZP	445 ZP	415 ZP	385 ZP	355 ZP	330 ZP	320 ZP

Location factor, Z (provisional(4)):

Coarse aggregate type	North Island	South Island
Crushed	Kirikiriroa Hamilton, Z=1.02 Tāmaki Makaurau Auckland, Te Whanganui-a-Tara Wellington, Tauranga, rest of Te Ika-a-Māui North Island(3), Z=1.0	Ōtepotī Dunedin, Z=1.0
River gravel	Heretaunga Hastings, Whakaoriori Masterton,Te Papaioea Palmerston North, Z=0.98	Ōtautahi Christchurch, Z = 0.93 Whakatu Nelson, Waihopai Invercargill, Tāhuna Queenstown, rest of Te Waipounamu South Island(3), Z=0.98

Placement and handling factor, P (provisional(4)):

- In-situ and precast concrete, placed without pumping, P=1.00
- Pumped in-situ concrete, P=1.05
- Precast prestressed floor units, P=1.10
- Grout infill for block masonry, P=1.10 for 13mm aggregate, P=1.30 for 7mm aggregate
- Self-compacting concrete, tremie/CFA mix for piles, or similar SCC mixes P=1.15
- Sprayed in-situ concrete, P=1.30
- Fibre-reinforced concrete – consult supplier

Notes:

1. **Objectives:** The 'SESOC On Track' limits for Global Warming Potential (GWP) are aligned with the *"A Net-Zero Carbon Concrete Industry for Aotearoa New Zealand: Roadmap to 2050"* [5] developed by Concrete NZ. A key assumption is that 50% of the total emissions reductions must be achieved by 2030, in line with IPCC recommendations (42% reduction is targeted in concrete production, with the remainder coming from design optimisation, reduced A4-A5 emissions during construction, and carbon absorption by the concrete).

   The "on track" reductions are roughly linear between 2026-2030, although it could be argued that faster reductions are achievable in the early years (2026-2028), before flattening out to a similar limit by 2030.

   Reductions in GWP beyond 2030 are indicative only and will depend on future developments such as the scale of market demand for low-carbon binders, decarbonisation of domestic cement production, and how quickly emerging low-carbon cement technologies can be brought to market.

2. **How to specify:** to specify GWP from this table, calculate the A1-A3 GWP limit for each concrete type and enter these numbers into your project specifications.

3. **Rural availability:** for plants with no SCM silo(s) available (typically small independent ready-mix suppliers in rural areas), we recommend specifying the lesser of NZGBC baseline or 2026 limits. Lower GWP values may be achievable in discussion with the local ready-mix supplier(s) – refer to Appendix B for guidance on consultation with nearby production plants.

4. **Provisional data:** the proposed Location Factors (Z) and Placement and handling factors (P) are based on a limited dataset with high-level review by several of the larger ready-mix suppliers only. Concrete NZ is developing a more comprehensive data set which will offer more accurate guidance once published.

5. **Impacts of SCMs:** designers should be aware of the potential effects of increased use of supplementary cementitious materials (SCMs), as outlined in Appendix C. For slabs on grade with highly worked surface finish (e.g. in warehouses), we recommend that designers consult with the ready-mix supplier(s) before specifying a GWP limit that is more than 20% below the ISC2020 baseline.

6. **Cost implications:** when sourcing concrete from ready-mix suppliers in major urban centres, a 2% cost premium for every 10% reduction in GWP has been suggested as a rule of thumb. Modest GWP reductions could well be cost neutral. A larger cost premium may apply for precast concrete due to the cost of additives to increase early age strength. These estimates are indicative only and cannot be guaranteed.

7. **Units and rounding:** GWP units are kgCO2e/m3 and have been rounded up to the nearest 5 kgCO2e/m3 (including baseline values).

8. **Lifecycle stages**: the reported GWP values cover life cycle stages A1–A3 (raw material supply, transport, and manufacturing). Emissions related to precast concrete fabrication processes are excluded due to limited EPD data currently available in New Zealand.

9. **Date of placement:** the applicable GWP limit for a concrete grade depends on the calendar year at the commencement of construction (this aligns with UK practice [70]). For projects with a construction duration longer than 2 years, the specifier will need to devise an appropriate method for reducing the GWP limits during construction (e.g. apply different A1-A3 GWP limits to different phases or consent stages).

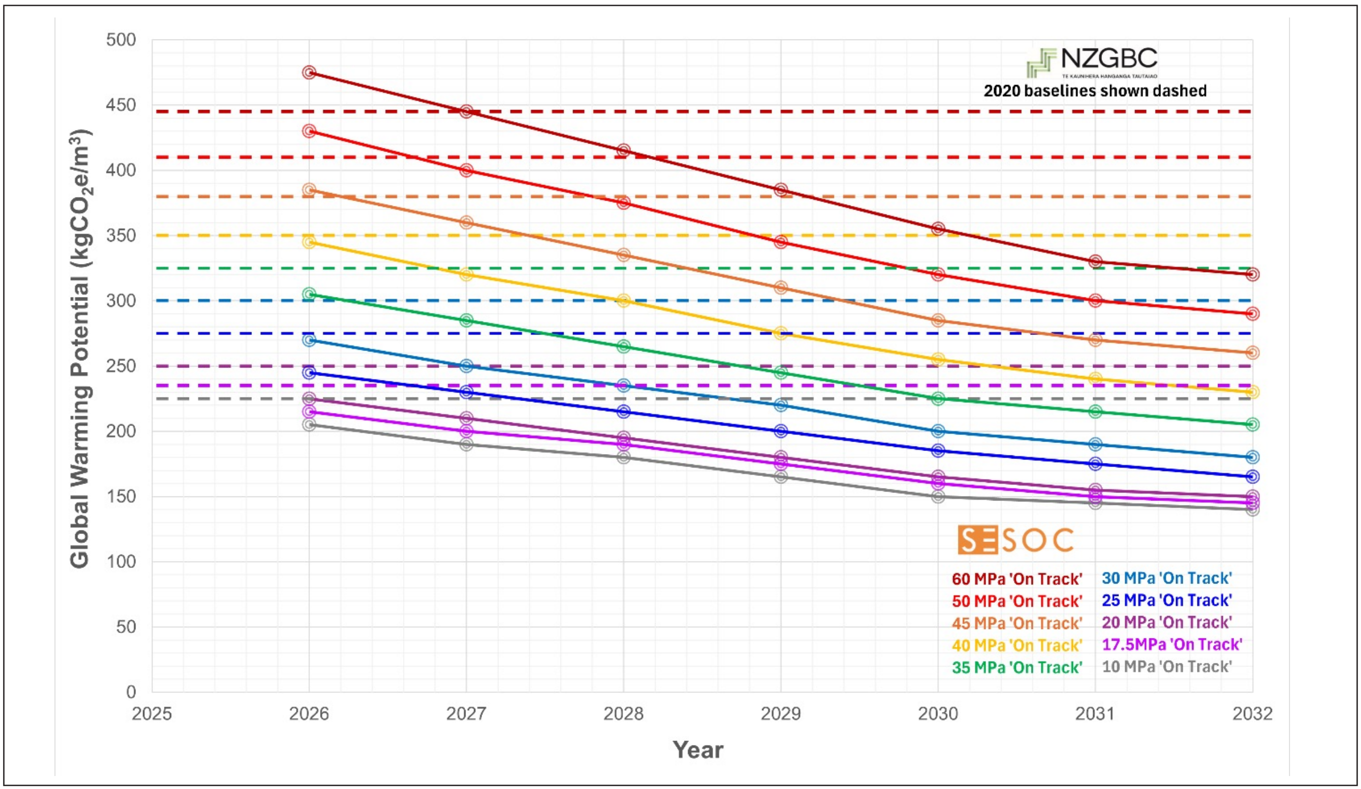

Figure A.1: 'SESOC On Track' GWP limits compared with NZGBC baselines (2020)

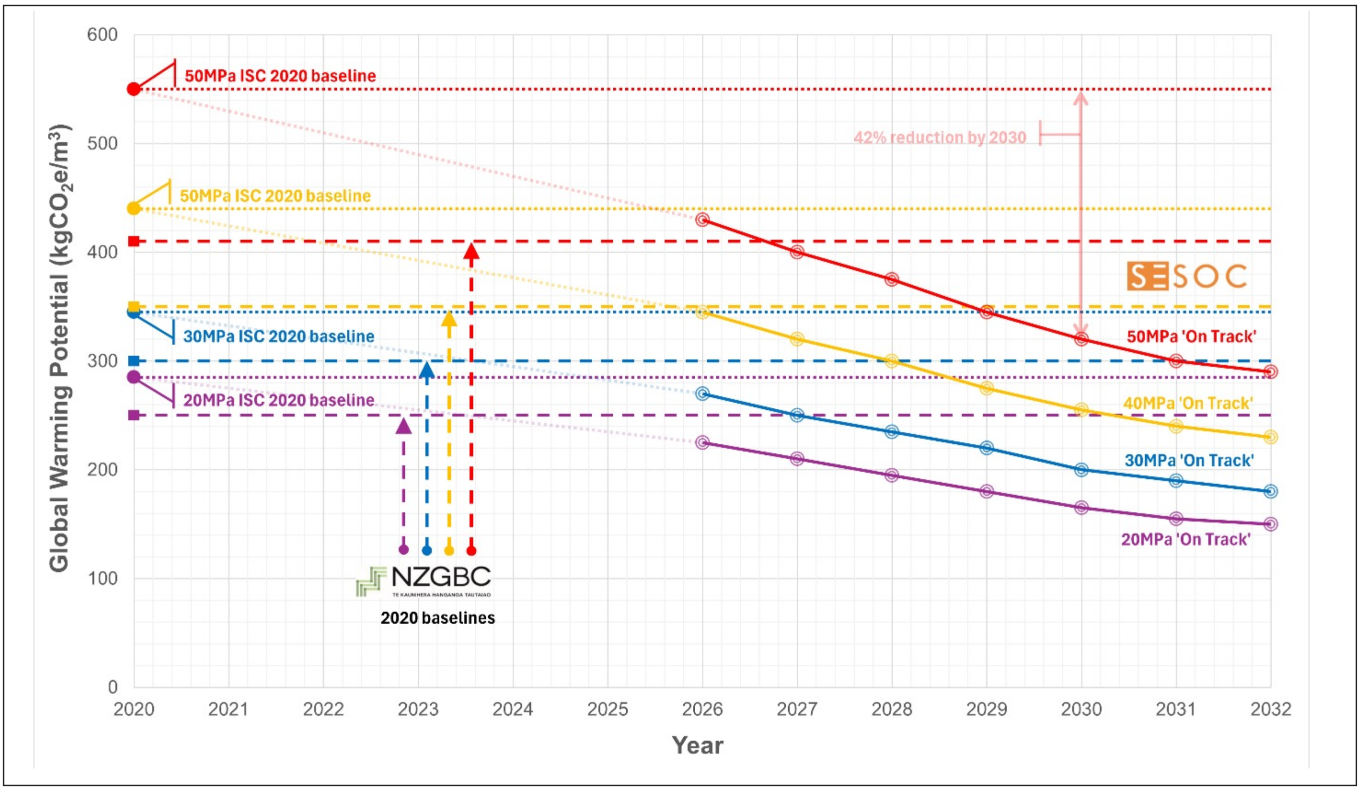

Figure A.2: Comparison of selected 'SESOC On Track' GWP limits with ISC & NZGBC baselines

Figure A.3: Baselines, GWP limits and cautionary limits for 30MPa concrete

APPENDIX B:

'SESOC LC' (lower carbon) A1-A3 GWP limits for concrete in New Zealand

Table B.1: maximum A1-A3 GWP for concrete to comply with 'SESOC LC' limits(1)

Grade	ISC 2020	NZGBC baseline 2020	2026 (~30%)*	2027 (~35%)*	2028 (~40%)*	2029 (~45%)*	2030 (~50%)*
20 MPa	285	250	215 ZP	195 ZP	175 ZP	160 ZP	145 ZP
25 MPa	315	275	235 ZP	215 ZP	190 ZP	175 ZP	160 ZP
30 MPa	345	300	255 ZP	235 ZP	210 ZP	195 ZP	175 ZP
35 MPa	390	325	280 ZP	260 ZP	235 ZP	220 ZP	200 ZP
40 MPa	440	350	300 ZP	285 ZP	265 ZP	245 ZP	225 ZP
45 MPa	495	380	320 ZP	310 ZP	300 ZP	275 ZP	250 ZP
50 MPa	550	410	340 ZP	335 ZP	330 ZP	305 ZP	275 ZP

* Approximate average reduction in GWP below ISC 2020 baseline is shown in brackets below each year.

Location factor, Z (provisional(4)):

Coarse aggregate type	North Island	South Island
Crushed	Kirikiriroa Hamilton, Z=1.02 Tāmaki Makaurau Auckland, Te Whanganui-a-Tara Wellington, Tauranga, rest of Te Ika-a-Māui North Island(3), Z=1.0	Ōtepotī Dunedin, Z=1.0
River gravel	Heretaunga Hastings, Whakaoriori Masterton,Te Papaioea Palmerston North, Z=0.98	Ōtautahi Christchurch, Z = 0.93 Whakatu Nelson, Waihopai Invercargill, Tāhuna Queenstown, rest of Te Waipounamu South Island(3), Z=0.98

Placement and handling factor, P (provisional(4)):

- In-situ and precast concrete, placed without pumping, P=1.00
- Pumped in-situ concrete, P=1.05
- Precast prestressed floor units, P=1.10
- Grout infill for block masonry, P=1.10 for 13mm aggregate, P=1.30 for 7mm aggregate
- Self-compacting concrete, tremie/CFA mix for piles, P=1.15
- Sprayed in-situ concrete, P=1.30

Notes:

1. **Objectives:** The 'SESOC LC' (Lower Carbon) GWP target is intended to represent a higher level of performance in the design and specification of concrete for reduced emissions, for use in appropriate applications.
2. **Important considerations:**
   - **Availability:** it is anticipated that concrete meeting this GWP level can be supplied by ready-mix providers in main urban centres (Tāmaki Makaurau Auckland, Kirikiriroa Hamilton, Tauranga, Whanganui-a-Tara Wellington, Otautahi Christchurch, and some others). These plants make up 30-40% of the ready-mix yards in Aotearoa New Zealand, but supply something like ~80% of the concrete by volume. For plants with no SCM silo(s) available (typically small independent ready-mix suppliers in rural areas), the "Lower Carbon" limits may not be achievable.
   - **Consultation with suppliers:** before specifying concrete from Table C.1, it is important that the specification is discussed with concrete suppliers to confirm the practicality of the mix for the proposed application. Greater or lesser GWP savings may be achievable for specific applications or in specific locations, which may influence the design approach. Plants may have the option of bringing in portable silos for SCMs for large scale projects. It is recommended that specifiers identify the 2-3 concrete production plants that are closest to the project site and contact their mix design engineers to discuss project requirements before finalising the specification. An online map showing plant locations & contact details are available at: https://rmcplantaudit.org.nz/audited-plants/
   - **Cost:** a suggested rule of thumb is 2% cost premium for every 10% reduction in GWP. A larger cost premium may apply for concrete with special characteristics other than 28-day strength (e.g., workability, early strength) due to the cost of additives.
   - **Impacts of SCMs:** designers must understand how increased use of supplementary cementitious materials (SCMs) can affect concrete performance, as outlined in Appendix C. The specific impacts depend on the type and dosage of SCM used, and this means the SESOC LC mix will not be suitable for certain applications. For example, specifying a GWP limit more than 30% below the ISC2020 baseline is generally not recommended for structural slabs, particularly those involving large pour areas or highly-worked surface finishes.
3. **How to specify:** to specify GWP from this table, calculate the A1-A3 GWP limit for each concrete type and enter these numbers into your project specifications.
4. **Units and rounding:** GWP units are kgCO2e/m3 and have been rounded up to the nearest 5 kgCO2e/m3 (including baseline values).
5. **Lifecycle stages:** the reported GWP values cover life cycle stages A1–A3 (raw material supply, transport, and manufacturing). Emissions related to precast concrete fabrication processes are excluded due to limited EPD data currently available in Aotearoa New Zealand.
6. **Date of placement:** the applicable GWP limit for a concrete grade depends on the calendar year at the commencement of construction (this aligns with UK practice [70]). For projects with a construction duration longer than 2 years, the specifier will need to devise an appropriate method for reducing the GWP limits during construction (e.g. apply different A1-A3 GWP limits to different phases or consent stages).

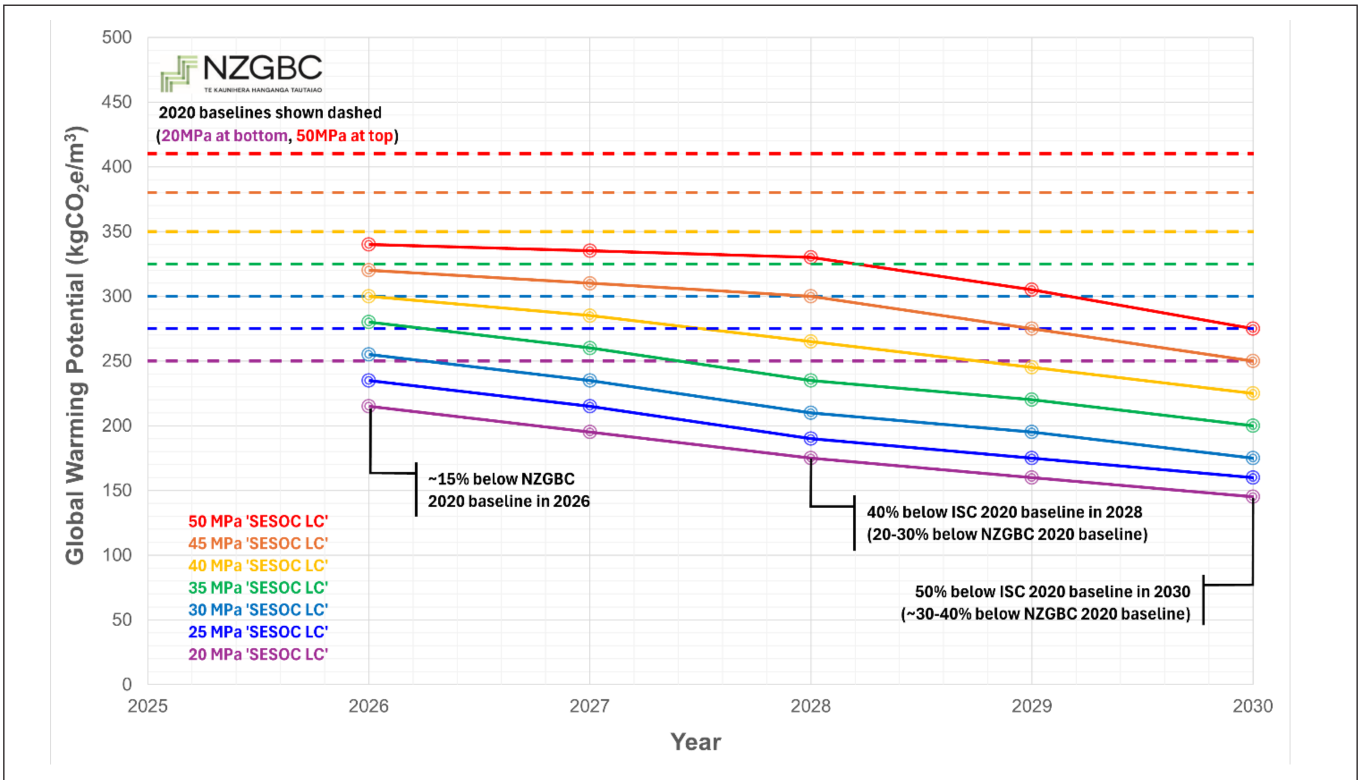

Figure B.1: 'SESOC LC' GWP limits compared with NZGBC baselines (2020)

APPENDIX C:

Guidance on concrete with supplementary cementitious materials

C.1 SCM Performance

Refer to links below for non-strength properties of concrete:

- Supplementary Cementitious Material (SCM) Research (Concrete NZ) [71]
- Constituents of Concrete – Supplementary Cementitious Materials (CCA Australia) [72]
- ER66 – Removing the barriers to the use of significant levels of SCMs in concrete production in NZ (BRANZ) [73]
- If the specified concrete strength grade is governed by durability requirements rather than strength, further optimisation of A1-A3 GWP may be possible in discussion with the mix designer.

C.2 Placement and curing

Impacts on concrete placement vary for different SCMs, but the overall tendency is to reduce the amount of free water rising to the surface of the concrete. Increasing SCM dosage will tend to slow down the bleed time, set time, and early strength gain. It is important for designers and contractors to understand that the pour size, placement/finishing methods, and curing requirements may need to be adjusted to a suit mix design with higher SCM content.

Common considerations are outlined below:

Finishing time

- Slower bleed rates require adjusted finishing techniques and timing.
- Inappropriate finishing techniques will have an exaggerated impact for large concrete pours with highly worked surface finishes.
- If environmental conditions cause rapid drying, the surface may appear ready even while bleeding continues below.
- Premature finishing can cause surface delamination.

Curing

- SCMs undergo secondary hydration reactions which continue after initial cement hydration, so extended water curing is required to achieve target strength and durability.
- Slower bleed and set rates increase the risk of plastic shrinkage cracking when surface evaporation exceeds the bleed rate; early-age curing must be carefully managed.
- Water curing is important for concrete with SCMs but tends to result in a less uniform slab colour compared with curing compound use.
- Avoid practices such as prematurely drying the slab for sawcutting.

Restrained early thermal contraction

- Large temperature swings between day and night can cause early-age thermal contraction cracking before adequate strength develops. SCMs reduce early strength, increasing susceptibility.
- The risk of cracking varies by region (e.g. common in Otautahi Christchurch, uncommon in Tāmaki Makaurau Auckland).
- Mitigation options include reducing external restraint, selecting an appropriate concrete mix, and managing placement temperature (e.g. evening pours).

Slump

- It is recommended that concrete slump is selected by the concrete mix designer for concrete with specified GWP limits.This gives the mix designer more flexibility to achieve the specified strength while taking account of concrete placement and curing requirements.

C.3 Strength gain for concrete with high SCM content

EN 1992-1-1 defines three different classes of cement, which refer to the rate of strength development in the concrete, and are primarily determined by the SCM content as a percentage of total cement content:

- Class R (rapid): CEM I (at least 95% Ordinary Portland Cement)
- Class N (normal): GGBS > 35% or Fly Ash > 20%
- Class S (slow): GGBS > 65% or Fly Ash > 35%

For concrete using these cement classes, EN 1992-1-1 provides calculation methods to estimate mean compressive strength at a specific age, and parameters for estimating long-term deformations (e.g., creep, shrinkage and stiffness properties).

Concrete with SCM content in the “Class S” range is normally suited to use in foundations, retaining walls, larger columns, and transfer slabs. These concretes will not be appropriate when early striking times are required, or when temporary loads are expected to cause high stresses during construction. Concrete mixes with high SCM content should not be used in thin elements subject to rapid drying. [17]

When using concrete with SCM content in the “Class S” range, the specification of 56-day strength should be considered. When 56-day strength is specified, the long-term strength should be similar to an OPC (Class R) mix with the same strength specified at 28 days [17].

APPENDIX D:

Recommended A1-A3 GWP limits for low carbon steel

The following table has been created with reference to the HERA Design Guide: "How to specify low carbon steel" [24] and the NZGBC Embodied Carbon Methodology 2.0 , Table 8 [3].

HERA's recommended limits for A1-A3 GWP have been based on the Aotearoa New Zealand-specific "default ordinary" 2025 value, with a "targeted 10% reduction (annually) against the previous year's benchmark as a target to incentivize and reflect improving decarbonisation within the steel industry." [24] NZGBC have calculated a lower baseline value for hot-rolled sections in particular.

Recommended A1-A3 GWP limits are provided in Table D.1

The recommended A1-A3 GWP limits in Table D.1 have been based on the following assumptions:

1. **Hot rolled sections:** proposed 2026 limits are based on a mix of 80% sections at NZGBC baseline and 20% sections at NZGBC conservative value.
2. **Hollow sections:** the input feed for manufacturing hollow sections can change at short notice and is governed by demand in international markets. We have proposed that HERA's GWP limits for hollow sections are increased by ~5% to align with that of hot-rolled plate from 2027 onwards, since this is a potential feed stock.
3. **Domestic supply of EAF steel:** NZ Steel plans to commence supply of steel plate and coil manufactured using their new Electric Arc Furnace (EAF) at Glenbrook in 2026. These products can be turned into welded sections and cold-formed steel products in the domestic market. The EAF will increase the proportion of recycled steel and reduce the carbon emissions from steelmaking. NZ Steel are expecting their "average crude steel embodied carbon to start off up to 1.6t CO_2-eq per tonne of steel, and then reduce further with increased scrap ratios against a world average of 1.9t $CO2_2$-eq/t."[74] We have assumed a value of 1.85kg CO_2e/kg based on the EPD published in June 2025 for Pacific Steel reinforcement [12].
4. **HR plate & coil**: proposed 2026 limits are based on HERA guidelines. When steel becomes available from the NZ Steel EAF, the updated 2026 limits are based on a mix of ~80% NZ Steel (EAF) and ~20% steel at NZGBC conservative value.
5. **Welded sections:** to allow for the additional carbon footprint arising from fabrication of welded sections, HERA's Design Guide has applied a factor of 1.075 to the GWP for steel plate. For simplicity, recommended GWP limits are the same as for HR plate. This would correspond to a mix of ~85% NZ Steel (EAF) and ~15% steel at NZGBC conservative.
6. **Cold-formed sections:** ~0.3 kgCO_2e/kg has been added to allow for manufacturing/roll-forming processes. The 2026 limits are based on a mix of ~80% NZ Steel (EAF) and ~20% steel at NZGBC conservative value.
7. **Target reductions:** reductions of approximately 10%/year have been applied, starting from 2026.
8. **Units and rounding:** GWP units are kgCO_2e/kg and have been rounded up to the nearest 0.05 kgCO_2e/kg (including baseline values).
9. **Lifecycle stages:** GWP values are for life cycle stages A1-A3. Emissions from domestic pre-fabrication of steel connections etc may be excluded.

Table D.1: Recommended A1-A3 GWP limits for structural steel used in NZ by calendar year

<table>
<tr><th>Steel type</th><th>NZGBC 2020 baseline</th><th>NZGBC 2020 conservative</th><th>HERA 2025 baseline *</th><th>2026 limit*</th><th>2027 limit*</th><th>2028 limit*</th></tr>
<tr><td>Hot rolled sections(1)</td><td>1.95</td><td>3.75</td><td>3.20</td><td>2.30</td><td rowspan="4">2.10</td><td rowspan="4">1.90</td></tr>
<tr><td>Hollow sections *** (CHS/SHS, G350)(2)</td><td>2.55</td><td>2.55</td><td>2.45</td><td>2.20</td></tr>
<tr><td>HR Plates & HR Coil(4)</td><td>3.15</td><td>3.85</td><td>3.85</td><td>3.45
(2.30)**</td></tr>
<tr><td>Welded beams and columns(5)</td><td>3.45</td><td>3.85</td><td>4.15</td><td>3.75
2.30)**</td></tr>
<tr><td>Cold-formed sections(6)</td><td>3.80</td><td>4.15</td><td>N/A</td><td>4.10
(2.60)**</td><td>2.35</td><td>2.10</td></tr>
</table>

* Values in bold match HERA recommendations [24], rounded as per Note 8 below.
** Values in brackets are intended to be applied after the NZ Steel EAF has started supplying the NZ market (2026).
*** We recommend specifying a unique GWP limit for this product category on projects were there is a significant proportion of hollow sections larger than 650mm diameter/width, to be determined by a project-specific study.

APPENDIX E:

Example specification clauses for concrete

[Work in progress – refer to SESOC website or Masterspec online from July 2026]

APPENDIX E:

Example specification clauses for structural steel

[Work in progress – refer to SESOC website or Masterspec online from July 2026]

APPENDIX G:

Example specification clauses for structural timber

[Work in progress – refer to SESOC website or Masterspec online from July 2026]

Signing Documents During Construction (Small works)

It has come to light that some engineers are still failing to properly review documentation prior to issuing instructions or signing off inspections. Site engineers need to be properly briefed before attending site, particularly for a start-up contract. Sending junior engineers is only recommended if they are familiar with the type of work and properly understand the process in terms of what is required of the site engineer.

The first port of call for the site engineer is to review the Council consented documents. On several occasions I have been involved with complaints where a job has turned sour and the inspecting engineer signed off a concrete pour without consulting the geotechnical engineer (or having evidence that the geotechnical engineer had preceded them to site) and without a building consent in place.

So, minimum requirements when arriving on site are:

1. Check with the contractor that a building consent document is on site. Refer ACENZ / ENZ guidance on guidance on construction monitoring;
2. Also other documentation such as drawings and specification need to be checked on site;
3. Review the council documentation and see whether a geotechnical engineer certification is required and whether there are any special requirements for the structural engineer to sign off;
4. Ensure that the contractor has safety procedures in place;
5. Discuss check points and test requirements with the contractor;
6. On subsequent site visits ensure that propping looks satisfactory and if not sure query the contractor;
7. Ensure that the contractor is provided with a site note listing inspection(s) and attach photos – state whether passed, partial pass or fail and what is required;
8. Ensure that the Contractor has the site note with attached photos, preferably before their booking with a Council.

Editor


SARJEANT GALLERY

A SURVEY OF STRUCTURAL MODELLING PRACTICE FOR UNREINFORCED MASONRY BUILDINGS IN AOTEAROA NEW ZEALAND

Azhari, Y.[1], Giouvanidis,A.I.[2], Ingham, J.[3]

ABSTRACT

Structural modelling is central to seismic assessment and retrofit efforts for unreinforced masonry (URM) buildings, yet limited information exists on how these buildings are modelled in industry practice in Aotearoa New Zealand. A nationwide survey of structural modelling practice in the context of URM seismic assessment and improvement was undertaken to investigate modelling approaches, factors influencing method selection, sources of material properties, modelling assumptions, technical challenges, confidence in model reliability, and perceptions of existing guidance. A total of 45 responses were obtained from practitioners having a wide range of experience. The results indicate that simplified static analysis methods are most commonly used, particularly for low-rise buildings with regular configurations that make up most of the national URM building stock. Advanced numerical analyses are applied less frequently and are generally reserved for complex structures or cases requiring more detailed analysis. Modelling decisions are driven primarily by building complexity, project constraints, performance objectives, and the availability of reliable data. Material properties are typically derived from code-recommended values, engineering judgement, and simple in-situ testing, and uncertainties are managed through conservative assumptions and sensitivity checks. Respondents identified modelling challenges, including building complexity, uncertain load paths and boundary conditions, software limitations, and the interaction between existing construction and retrofit works. Confidence in modelling results was generally moderate. Existing guidelines were considered adequate for typical buildings but less effective for complex cases. Respondents in general expressed strong support for harmonised modelling guidelines to improve the consistency of modelling results and reliability of URM seismic assessment practice in Aotearoa New Zealand.

Keywords: modelling practice, structural analysis, survey, unreinforced masonry.

1 INTRODUCTION

Unreinforced masonry buildings in Aotearoa New Zealand

Māori first arrived in Aotearoa New Zealand between approximately 1320 and 1350 CE. Early Māori construction traditions were derived from Polynesian building practices and were adapted to the colder climate of Aotearoa NZ, resulting in predominantly timber structures with smaller semi-subterranean forms, rectilinear plans, low walls, and gabled roofs (Brown, 2009). The first European to reach Aotearoa NZ was Abel Tasman in 1642, while British interest in Aotearoa NZ increased during the 1830s (Stubbs et al., 2023). Early colonial buildings were typically utilitarian, plain, and solid in character but Renaissance-influenced architectural styles became dominant in public and commercial buildings during the final three decades of the nineteenth century. By this time, unreinforced brick masonry was increasingly used because timber construction was considered vulnerable to fire (Shaw & Hallett, 1995). Construction of unreinforced masonry (URM) buildings continued until the 1931 Mw 7.8 Hawke's Bay earthquake that effectively ended widespread URM construction and led to the development of the first seismic design provisions in Aotearoa NZ (NZSS, 1935).

The existing URM building stock in Aotearoa NZ is dominated by low-rise structures that are typically about 100 years old and generally represent the oldest surviving commercial buildings in the country (Figure 1). In 2010 the total number of URM buildings in Aotearoa NZ was estimated to be approximately 3,750 (Russell & Ingham, 2010). These buildings are commonly used for mixed commercial and residential purposes and are often located along main streets at the centres of many provincial towns and community hubs in larger cities. The URM building stock features similarity in design across all of Aotearoa NZ, with approximately 90–95% comprising relatively simple structural geometry, while the remainder of the building stock serves as larger historic buildings and town landmarks such as town halls, railway stations, museums and public theatres (Goodwin et al., 2009; Russell & Ingham, 2010).

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Figure 1: Typical low-rise URM buildings

The poor seismic performance of URM buildings is well-known in Aotearoa NZ (Blaikie & Spurr, 1992; Ingham & Griffith, 2010; Dizhur et al., 2011; Moon et al., 2014). Both in-plane and out-of-plane failure mechanisms present significant hazards, particularly where buildings are located near public spaces (Derakhshan et al., 2014; Dizhur et al., 2013; Dizhur et al., 2017). As a result, the assessment and strengthening of URM buildings have become major engineering and policy concerns. Guidelines, including the Detailed Seismic Assessment Guidelines (commonly referred to as C8), NZS 1170.5, and related procedures (e.g. ASCE 41-23), provide recommended methods for evaluating seismic performance. These frameworks define analytical procedures, acceptance criteria, and performance objectives for existing URM buildings. Within this framework, structural modelling is a critical component for estimating seismic response and identifying governing failure mechanisms in URM structures.

Modelling of URM buildings

A wide range of methodologies have been developed for the seismic assessment and structural modelling of URM buildings. These methods are used to predict the complex behaviour of URM buildings and meet the different objectives of engineering analysis. They range from simplified calculation-based procedures to advanced numerical simulations. The choice of approach is usually governed by building characteristics, available information, required level of accuracy, and project constraints. Structural modelling has therefore become an essential tool in using methods beyond the basic procedures provided in codes and guidelines for existing URM buildings.

Lessons learned from blind prediction studies provide clear evidence of the challenges associated with URM modelling (Mendes et al., 2017; Cattari & Magenes, 2022; Calderini et al., 2024; Tomić et al., 2024). Results from these blind prediction studies were largely varied, not only across different modelling approaches but also across analyses conducted by different modellers using the same method. This variability demonstrates the strong influence of modelling assumptions and user decisions on the analysis results.

A notable example is a blind prediction exercise on a three-storey European masonry building reported in a Special Session of the European Conference on Earthquake Engineering. Participating teams used a variety of modelling strategies (Parisse et al., 2021). The predicted damage patterns and collapse mechanisms were generally similar, yet the type and extent of failure modes differed across analysis results obtained using different approaches. More importantly, the predicted capacity curves and peak ground accelerations (PGA) were significantly scattered. The variability was particularly pronounced in brick masonry buildings, meaning that clearer modelling procedures are needed to support consistent professional practice. Similar conclusions were drawn from the URM nonlinear modelling benchmark project funded by the ReLUIS consortium and the Italian Department of Civil Protection (Cannizzaro et al., 2022). In that study, the scatter of results was reduced by adopting consistent modelling assumptions. However, noticeable differences remained, especially in the ultimate displacement capacity of the pushover curve, which is a parameter that significantly affects the seismic assessment results. These blind prediction studies collectively indicate that reliable structural modelling requires expertise in the selected modelling approach, material representation, and software implementation. Developing such expertise typically demands considerable time and experience. In professional practice, however, engineers are often required to undertake complex analyses within tight timeframes dictated by project budgets and contractual obligations. At the same time, software developers aim to provide user-friendly interfaces and default parameters to facilitate wider use. While simpler software features improve accessibility, they may also enable inexperienced users to perform sophisticated analyses without fully understanding the assumptions or limitations associated with the selected method or the software that is used.

Despite the importance of modelling decisions, current building codes provide limited guidance on the selection and application of modelling approaches, and substantial reliance is placed on engineering judgement during model development. All these issues highlight the need for frameworks and recommendations to support the proper use of these modelling approaches and raise awareness among users about the reliability of the model outputs.

The results of a nationwide survey of practitioners involved in the seismic assessment of unreinforced masonry buildings in Aotearoa NZ are presented in this study. The modelling approaches, reasoning behind modelling choices, material assumptions, challenges encountered, and perceptions of reliability of the current modelling approaches and guidelines were investigated. The broader objective of this study was to identify areas where additional guidance, standardisation, or research may be beneficial.

2 SURVEY METHODOLOGY

Survey design

The survey was designed to investigate the current industry practice in the structural modelling of URM buildings in Aotearoa NZ. The questionnaire consisted of multiple sections covering respondent demographics, modelling approaches, sources of model properties and assumptions, modelling challenges, and the perceptions of practising engineers regarding the codes and guidelines available for the seismic assessment of URM buildings.

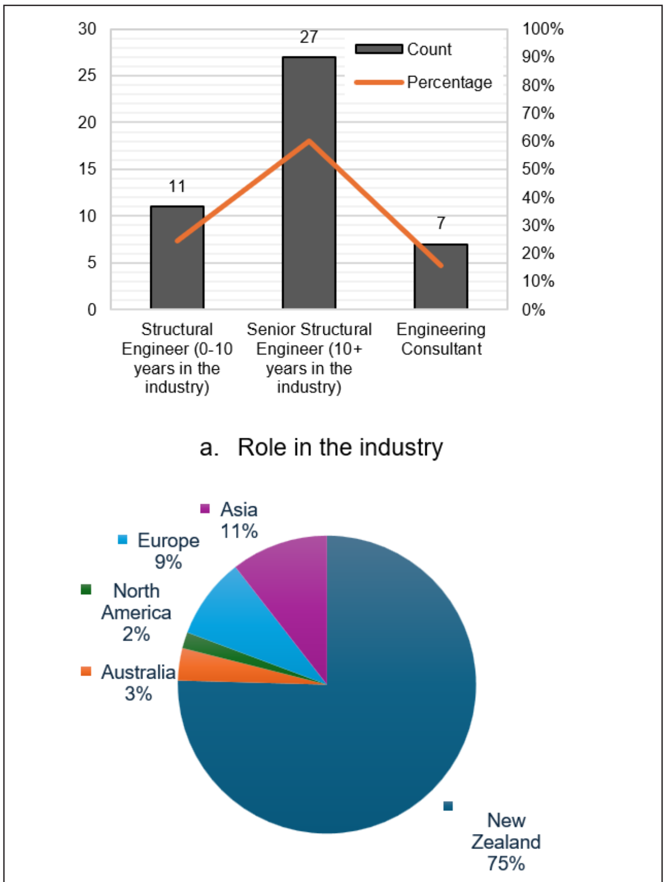
The questionnaire included a combination of closed-ended and open-ended questions. The closed-ended questions employed multiple-choice or Likert-type formats to enable quantitative analysis of modelling approaches, confidence levels, and influencing factors, with several questions allowing multiple selections when appropriate. The open-ended questions were designed to obtain qualitative insights into the reasoning behind the modelling choices, challenges encountered in practice, and suggested improvements to codes and guidelines. This mixed-methods approach captured both statistical trends and practitioner perspectives. Ethics approval for the study was obtained from the University of Auckland Human Participants Ethics Committee prior to data collection (Ref. UAHPEC28901). Participation was voluntary and anonymous, and no personally identifiable information was collected without consent.

Respondents and survey distribution

The inclusion criteria for respondents were structural engineers practising in Aotearoa NZ who have experience in the seismic assessment or modelling of URM buildings. The survey was distributed primarily through the Structural Engineering Society of New Zealand (SESOC) membership, which includes a large proportion of practising structural engineers in the country. Distribution channels utilised were professional mailing lists and networks associated with SESOC. Due to voluntary participation and self-identification of respondents as having relevant experience in URM modelling, the sample reflects practitioners actively engaged in this area of work rather than the broader engineering population.

A total of 45 valid responses were obtained and approximately 50% represented fully completed surveys. Among the 45 respondents, 60% identified themselves as Senior Structural Engineers with more than 10 years of industry experience, 24% identified themselves as Structural Engineers with less than 10 years of experience, and 16% identified themselves as Engineering Consultants (Figure 2a). In this context, a structural engineer refers to a specialist focusing specifically on the design, analysis, and safety of structures such as buildings and bridges, whereas an engineering consultant refers to a licensed expert providing strategic advice, design, and project management services across a range of engineering disciplines.

In terms of experience specifically related to the seismic assessment of unreinforced masonry (URM) buildings, 38% reported more than 10 years of experience, 31% reported 6 to 10 years of experience, 18% reported 2 to 5 years of experience, and 13% reported less than 2 years of experience. The majority of respondents indicated that their professional work and experience were based in Aotearoa NZ (75%), while the remaining 25% reported experience that also covered other regions, including Asia, Europe, Australia, and North America (Figure 2b).



a. Role in the industry

b. Region of experience

Figure 2: Respondent demographics

Data analysis

Responses were analysed using a combination of quantitative and qualitative methods. Quantitative analysis was applied to closed-ended responses to determine frequencies, distributions, and dominant trends. Qualitative responses to open-ended questions were analysed using thematic coding to identify recurring themes and common categories. This process involved grouping similar responses and identifying frequently occurring keywords through word-cloud illustration when appropriate to provide a structured interpretation of qualitative data.

3 SURVEY RESULTS AND ANALYSIS

Modelling strategies

Modelling efforts were most commonly undertaken to support retrofit design decisions, identify governing failure mechanisms, estimate building force capacity, understand load paths, obtain building displacement capacity, and obtain force resultant values for structural members. The distribution of the modelling objectives is presented in Figure 3.

Respondents reported a wide range of calculation and modelling approaches for URM buildings, and the results reflected both the diversity of building characteristics and the absence of a single standardised methodology for URM seismic assessment. As shown in Figure 4, a calculation-based simplified static method (e.g. force-based calculation, code-based calculation) was selected by 23% of respondents, software-aided linear static analysis was selected by 15% of respondents, a calculation-based equivalent frame method was chosen by 13% of respondents, and software-aided nonlinear static (pushover) analysis was selected by 12% of respondents. Other approaches mentioned by respondents included simplified lateral mechanism analysis (SLaMA) and a calculation-based macro-element method. These approaches appear to be favoured because they are relatively straightforward to implement, computationally efficient, and widely understood among practitioners. The practice trend is also consistent with the characteristics of the URM building stock in Aotearoa NZ, which consists largely of low-rise buildings with relatively regular configurations that allow simplification and generalised assumptions. More advanced techniques were used less frequently

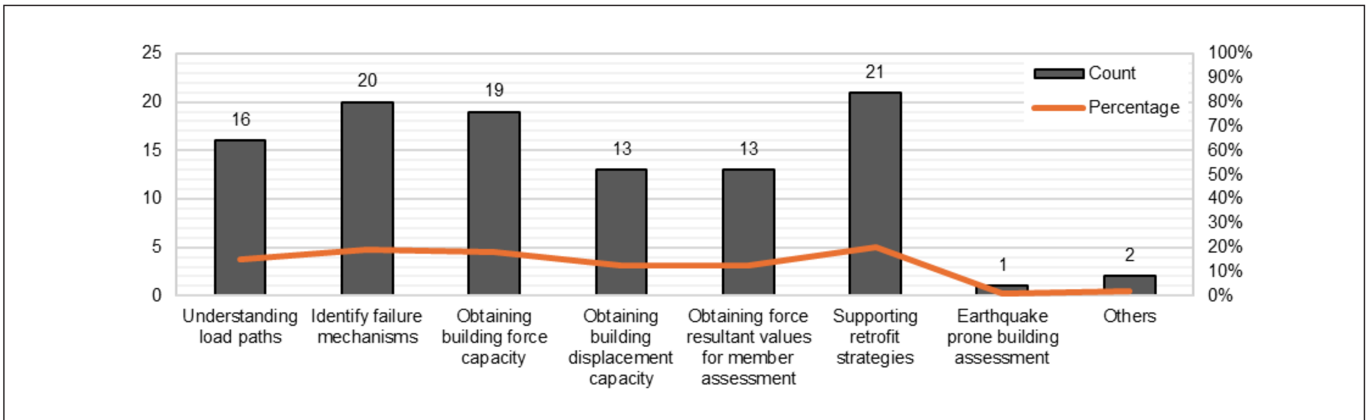


Figure 3: Modelling objectives

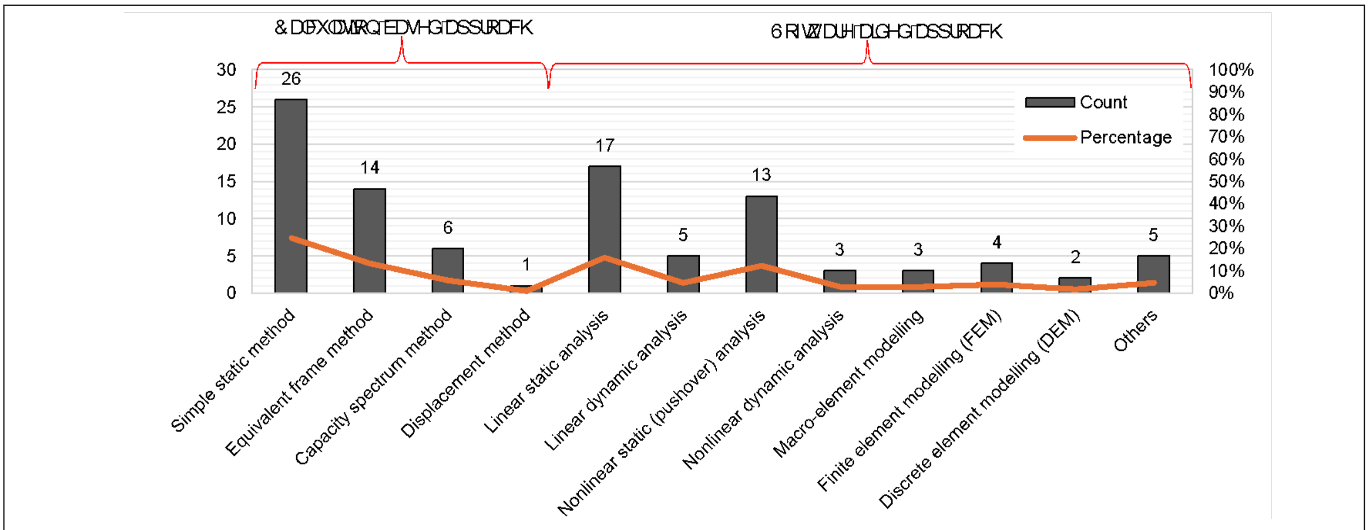


Figure 4: Modelling and analysis approach

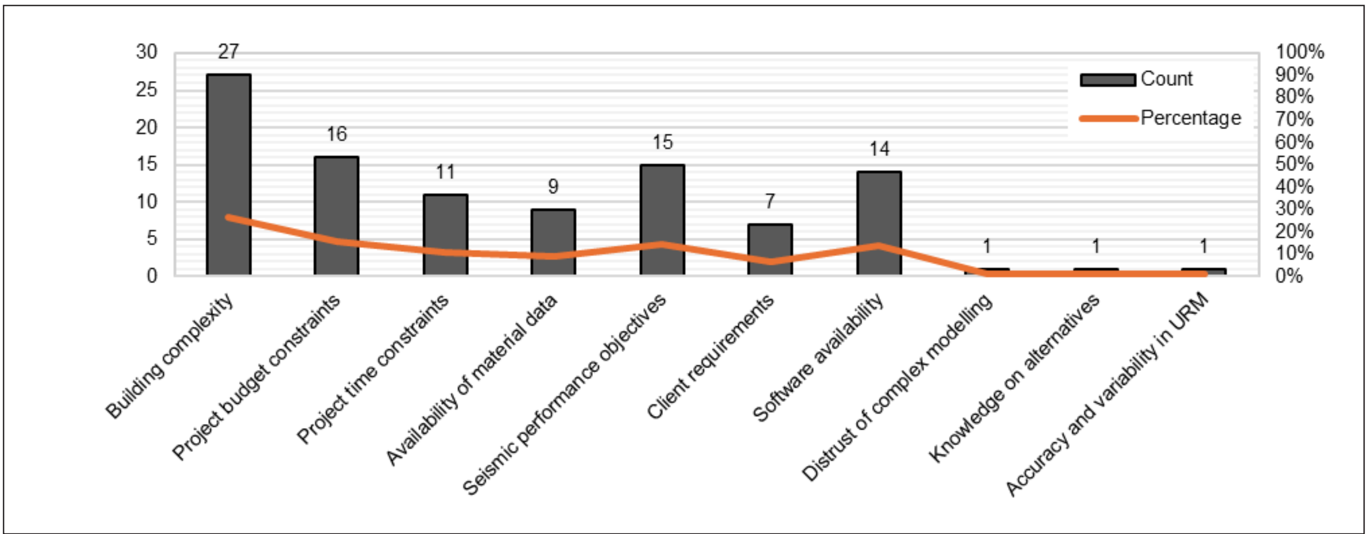


Figure 5: Reasoning behind modelling choices

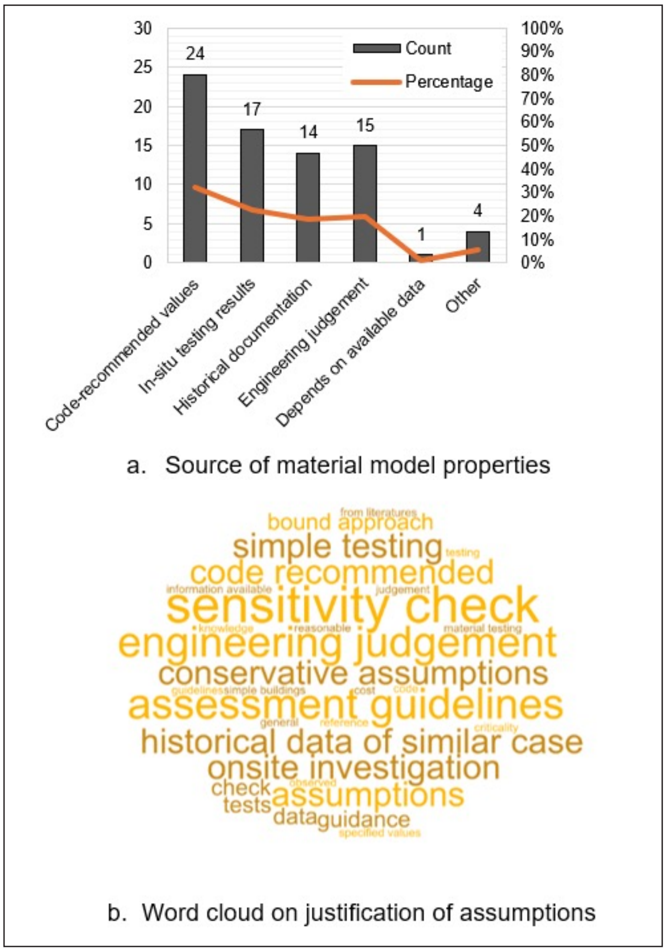
and were typically reserved for complex structures or situations requiring greater analytical detail. Nonlinear dynamic analysis, macro-element modelling, finite element modelling (FEM), and discrete element modelling (DEM) were reported by a smaller proportion of respondents, likely due to their computational cost and the need for specialised expertise.

The selection of modelling strategy was influenced by multiple factors, with building complexity identified as the dominant consideration (26%), as shown in Figure 5. Practical constraints, such as project budget limitations (16%), time constraints (11%), and availability of data (9%), also played a significant role, which accounted for over a third of responses and reflected the trade-offs between analytical rigour and project feasibility. Seismic performance objectives were another major reason (15%) as higher performance targets typically require more advanced modelling approaches. Software availability (14%) was also influential, indicating that modelling choices are partly shaped by access to suitable tools and familiarity with specific platforms. Client requirements that represent the role of stakeholder expectations in determining the level of analysis undertaken accounted for 7% of responses. Factors related to scepticism towards advanced modelling were scarcely mentioned as the reasoning behind the selection of approach. Distrust of complex modelling, lack of knowledge or guidance on alternative methods, and concerns regarding variability and uncertainty in URM behaviour each accounted for only 1% of responses. Overall, it is suggested from the results that modelling strategies in practice are driven primarily by technical demands and practical constraints rather than personal expertise and preferences. This observation is important because it may help the development of multi-level modelling guidelines that link the level of analytical complexity imposed by building characteristics, project constraints, and the consequences of modelling decisions.

Material modelling

Considerable variability was observed in how the material properties were determined and incorporated into the URM models. The sources of material model properties adopted in URM analysis and the rationale for material modelling assumptions in the absence of comprehensive data are presented in Figure 6. The code-recommended values were used most frequently as the source of material model properties (32%), which indicated a strong reliance on guidance and seismic provisions when detailed testing data are unavailable (Figure 6a). In-situ testing results (23%) were also used when feasible. The scale of testing mentioned varied considerably among respondents, ranging from simple scratch tests to full on-site investigations. In some responses, the extent of testing undertaken was not specified. Engineering judgement (20%) and historical documentation or published literature (19%) were also commonly used. This result suggests that practitioners rely on professional experience and previously documented information when first-hand data are limited. A small number of responses did not identify a specific source of material properties and instead indicated that modelling assumptions depended on the resources available for the project or on discrepancies between recommended values and observed building conditions.

Keywords of how assumptions are justified in practice are presented in Figure 6b. The prominence of terms such as engineering judgement, conservative assumptions, sensitivity check, code-recommended values, and on-site investigation indicates that engineers rely heavily on experience-based reasoning supported by the available references. Historical and similar case studies that were mentioned suggest that information from comparable buildings was used to supplement limited first-hand data. Practitioners also mentioned that adjustment of material property values during the modelling process is relatively common and is primarily undertaken to assess sensitivity and improve the realism of predicted behaviour. Sensitivity analyses were used to establish upper and lower bound capacities and to determine whether further investigation or testing was warranted. Overall, the results indicate that uncertainty in material properties is typically managed through conservative assumptions, verification/sanity checks, and iterative analysis.



a. Source of material model properties

b. Word cloud on justification of assumptions

Figure 6: Material modelling practices

Modelling challenges

Respondents identified numerous technical challenges associated with the modelling of URM buildings (Figure 7). The most frequently reported challenge was the overall complexity of modelling (26%) caused by difficulties associated with irregular geometry, mixed structural systems, heritage features, and uncertain construction details. Uncertainties in load paths (16%) and boundary conditions (16%) were also major concerns, particularly where diaphragm behaviour or connectivity between structural components was unclear. Software limitations and computational demands accounted for 12% of responses. Budget and time constraints were cited by 12% of respondents. Lack of reliable material data accounted for 10% of responses and was often compounded by deterioration and prior modifications.

Other challenges (9%) mentioned were a range of practical issues not captured by predefined categories. These challenges included difficulties in identifying the actual construction form from site investigations, limited confidence in complex numerical modelling or software results, concerns regarding diaphragm capacity and performance, and challenges in modelling systems beyond those explicitly covered by existing guidelines. Complex structural configurations, irregular geometry, and the presence of heritage features further increased modelling difficulty. Several dominant themes related to the challenges associated with modelling complex URM buildings are presented in Figure 8a. Terms such as computational time, software limitations, complex URM behaviour, and variability and inconsistencies indicate that advanced modelling is constrained not only by analytical difficulty but also by practical feasibility. URM structures generally exhibit non-homogeneous behaviour that is challenging to characterise, particularly when material properties and construction details are uncertain. The mention of hand calculation limitations suggests that simplified static methods may

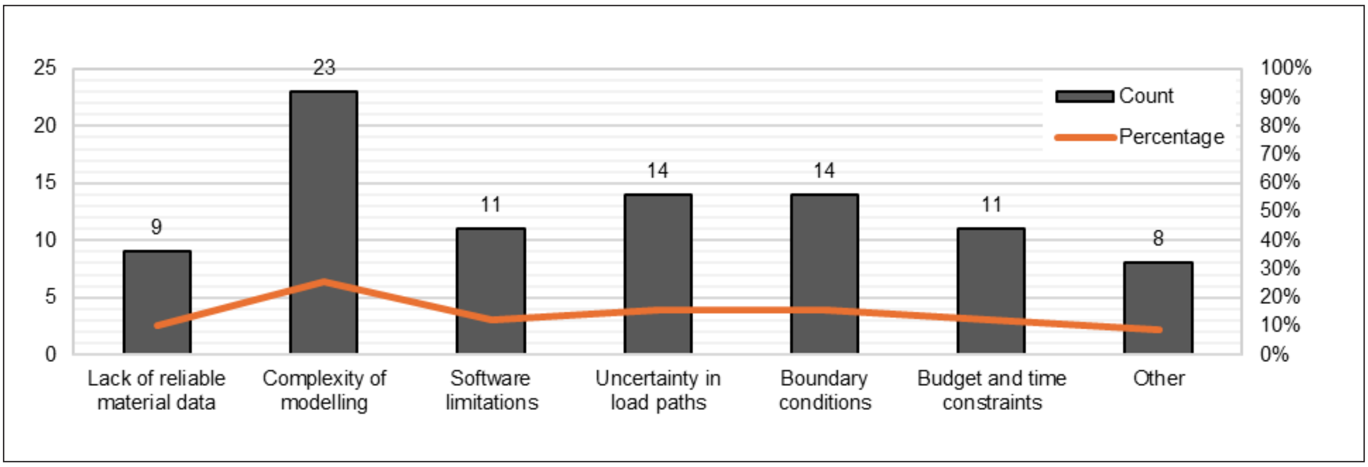


Figure 7: Challenges when modelling URM buildings

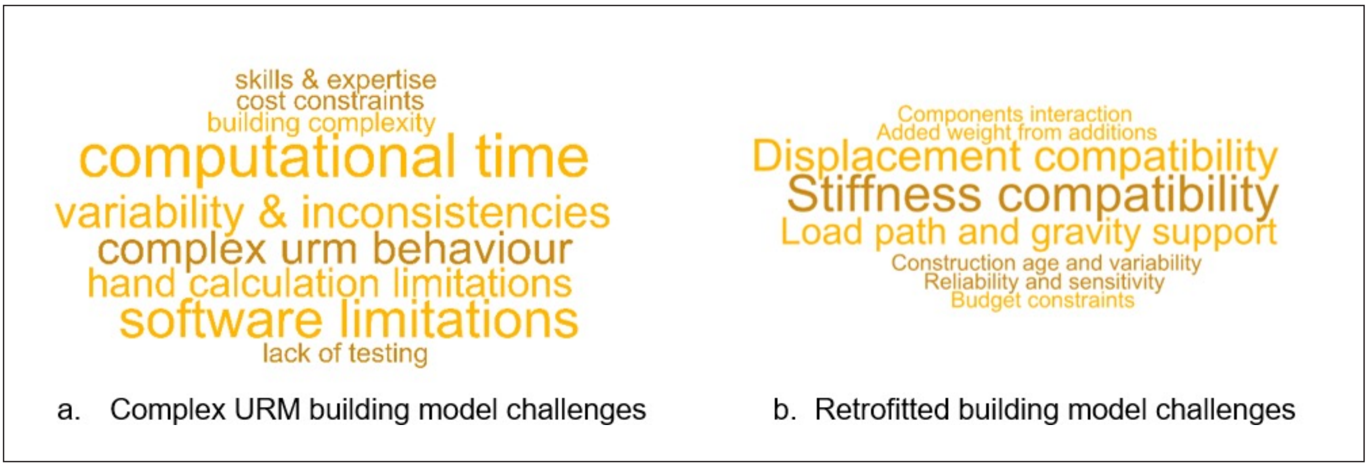


Figure 8: Challenges when modelling complex and retrofitted URM buildings

be insufficient for complex buildings, yet the required alternatives impose significant computational demands. The mention of cost constraints and expertise further indicates that advanced modelling approaches were not considered as pragmatic solutions. Specific technical difficulties faced by practitioners when modelling complex URM buildings included modelling out-of-plane wall behaviour, multi-leaf or cavity walls, irregular geometry, and buildings with unknown internal construction. To manage these challenges, engineers reported using a combination of conservative assumptions, simplified modelling techniques, targeted investigations, and engineering judgement.

Modelling of retrofitted URM buildings introduces additional complexity due to the interaction between existing materials and new strengthening systems. Respondents reported challenges related to stiffness compatibility, load redistribution, and displacement compatibility (Figure 8b). These compatibility issues arise from the difficulty of predicting how strengthening interventions interact with the original construction. Retrofitting often introduces new materials and structural systems with different stiffness and strength characteristics that can alter load distribution and deformation patterns. Additional concerns, such as added weight from additions, component interaction, and construction age and variability, emphasise the uncertainty associated with existing conditions and the potential for unintended consequences following the retrofitted works. The mention of reliability and sensitivity indicates that engineers must carefully evaluate the robustness of modelling assumptions, particularly when small changes in parameters may significantly influence the modelling results. In the context of complex retrofitted building models, some respondents also questioned the practical usefulness of highly detailed modelling when strengthening interventions are typically required regardless of the level of analytical precision achieved.

Confidence in the reliability of URM modelling results was generally moderate (Figure 9). Most respondents reported being somewhat confident (54%), while a substantial proportion expressed neutral views (34%). Only a small fraction of respondents indicated very high confidence (4%), and 8% of respondents reported low confidence. This distribution is further evidence that practitioners recognise both the usefulness and the limitations of commonly used modelling approaches. The predominance of simplified static analysis methods and the significant technical challenges associated with URM modelling likely contribute to this cautious confidence. While such approaches are practical and consistent with guideline procedures, the inherent uncertainties in URM modelling limit the level of confidence in detailed predictions.

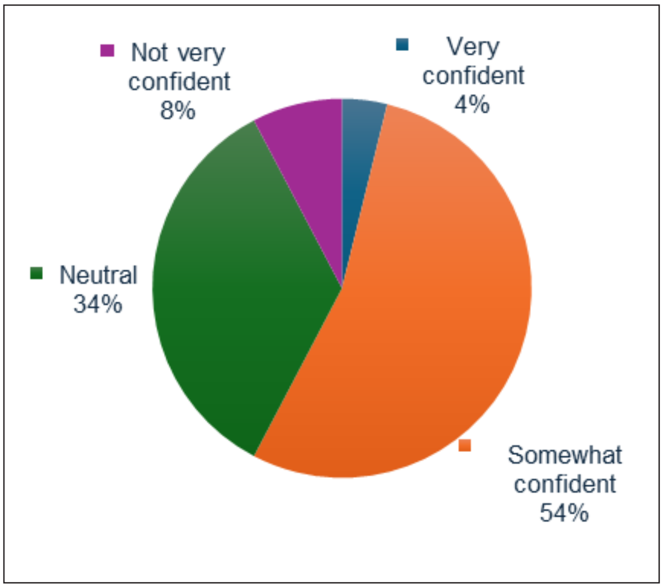
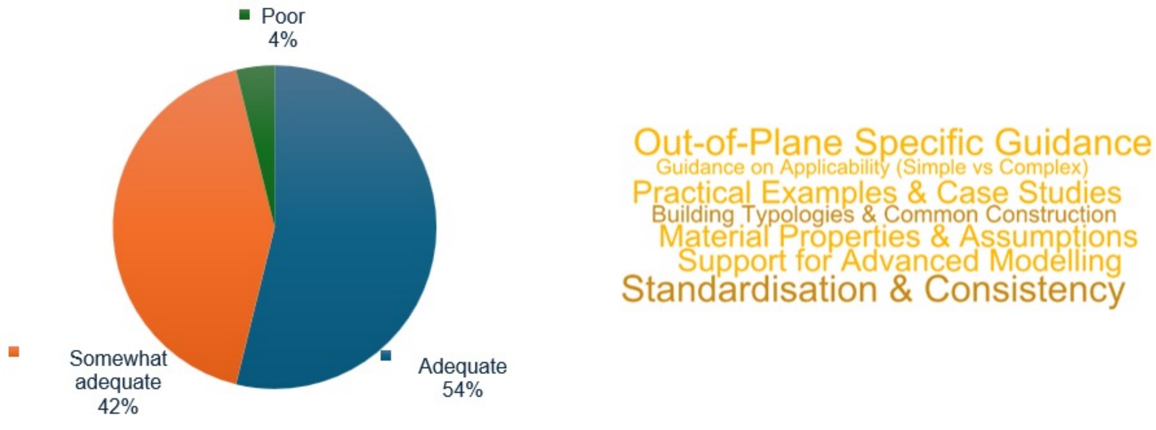


Figure 9: Confidence in the reliability of URM modelling



a. Adequacy of the current seismic guidelines for URM buildings

b. Aspects that should be better addressed in the guidelines

Figure 10: Respondent views on the seismic assessment guidelines for URM buildings

Code and guideline development

The majority of respondents indicated frequent use of established guidance documents for seismic assessment, particularly the Detailed Seismic Assessment Guidelines and NZS 1170.5. These documents were generally regarded as adequate for typical cases, with 54% of respondents rating them as adequate and 42% as somewhat adequate (Figure 10a). Only a small proportion of respondents (4%) considered the guidance to be poor. Despite this overall positive assessment, respondents noted that the guidelines are less effective for complex or unusual buildings.

Respondents also identified several aspects that should be better addressed or clarified in the Aotearoa NZ seismic assessment guidelines for URM buildings (Figure 10b). The most prominent themes relate to the need for improved guidance on out-of-plane behaviour as a critical failure mode for URM structures. Practitioners also highlighted the importance of clearer guidance on the applicability of different assessment methods for simple versus complex buildings, as well as the corresponding practical examples and case studies. Additional suggestions included better characterisation of material properties and assumptions, guidance on common building typologies and construction forms, and stronger support for advanced modelling approaches.

Strong support was expressed for the development of harmonised guidelines specifically addressing URM modelling in Aotearoa NZ. Respondents indicated that greater standardisation could improve consistency in assessment outcomes, reduce uncertainty associated with modelling assumptions, and enhance confidence in the reliability of results. Such guidance was seen as particularly valuable for well-supported and defensible engineering decisions across projects involving existing URM buildings.

4 CONCLUSION

A snapshot of the current industry practice regarding the structural modelling of URM buildings in Aotearoa NZ was presented based on a survey of structural engineers engaged in assessment and retrofit projects. Based on survey results, the following conclusions were drawn:

1. Structural modelling of older URM buildings is inherently complex, primarily due to uncertainties in material properties, construction details, boundary conditions, and the structural behaviour of ageing buildings.
2. A wide spectrum of modelling approaches is used in practice, ranging from simplified calculation-based methods to advanced numerical modelling. Engineers typically select methods based on building complexity, available data and resources, and project constraints.
3. The results from a survey of the adopted modelling strategies indicated a preference for the simpler static approach aligned with the existing guidelines for seismic assessment of URM buildings when this approach is adequate for decision-making, while reserving advanced modelling techniques for cases involving greater complexity or higher consequence.
4. Engineers reported cautious confidence in modelling results and recognised both the usefulness of the existing practical tools and the limitations imposed by uncertainty and variability. Verification through sanity checks and sensitivity analysis remains common practice.
5. Existing national guidelines were generally considered adequate for typical assessments but less effective for complex cases, particularly in relation to out-of-plane behaviour, boundary conditions, and advanced modelling approaches.

6. There was strong support within the professional community for the development of more harmonised guidelines to improve consistency and reliability in URM seismic assessment across Aotearoa NZ.

Overall, the results of the survey highlight the need for further development of practical modelling frameworks that covers multi-level objectives and complexity to support reliable and consistent seismic assessment of URM buildings in Aotearoa NZ. Several limitations of the survey results should be acknowledged. First, the sample size represents only a subset of structural engineers in Aotearoa NZ who were exposed to the survey, and therefore, the results may not fully capture the diversity of practice across all firms and regions. Second, participation was voluntary and thus introduced the potential for self-selection bias, as engineers with a strong interest or experience in URM assessment may have been more likely to respond. In addition, some questionnaires were incomplete, so not all responses were available for every question. Despite these limitations, the survey provided valuable insight into the current professional practices and trends in URM modelling in Aotearoa NZ.

ACKNOWLEDGEMENT

The authors would like to thank the Structural Engineering Society of New Zealand (SESOC) for assisting with the distribution of the survey to its membership. The authors also thank all respondents for their time and valuable contributions to this study.

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Brain Teasers

1. Cantilevered deck: An engineer came across a reinforced concrete cantilever deck (about 1.6m wide). The deck was lower on the outside edge due to creep and appeared to have corrosion stains on top, most likely due to corroding reinforcing steel. The engineer's solution was to prop the deck on the outside edge to prevent collapse. Was this a sensible solution?

2. Moving load: A beam with a span L = 10 m is crossed by a truck with two axle loads: Pb = 60 kN (rear, bigger) and Ps = 40 kN (front, smaller), separated by a distance d = 5 m. Find the maximum bending moment in the beam.

3. Intermediate timber partition: An engineer has designed a building with long span suspended concrete floors (precast with topping). The architect has indicated an intermediate line of timber framing at each level. Should this raise a red flag for the engineer?

Answers on page 114

Tips & Traps

Portal frame buildings

HERA has finalised Steel Portal Frame Design Guide – Volume 1, with release scheduled for April 2026. The technical development of the guide is complete, and the document is currently undergoing final formatting. The guide provides comprehensive, practice-focused guidance for the structural design of steel portal frame buildings, reflecting current New Zealand standards and industry best practice. It covers loading and load combinations, analytical modelling, structural analysis, and design of primary portal frame steelwork. A detailed worked portal frame design example is included to demonstrate the practical application of the recommended design methodology.

TECHNICAL INSIGHTS ON THE DESIGN OF A PULTRUDED FIBRE REINFORCED POLYMER (FRP) TREATED WATER TANK ROOF STRUCTURE

Cheng, K.[1], Ivory, C.[2], Young, I.[3]

Abstract

Treated water tanks are important structures that help provide clean drinking water. The interiors of these tanks can be subjected to chlorine and humidity which can create a corrosive environment. Beca has been involved with replacing an existing mild steel roof structure with a pultruded Fibre Reinforced Polymer (FRP) roof structure over a treated water tank with the aim of achieving better durability. The roof design followed the guidance of the new American standard 'ASCE/SEI 74-23 Load and Resistance Factor Design (LRFD) for Pultruded Fiber Reinforced Polymer (FRP) Structures' released at the end of 2023. This paper highlights technical insights on designing with pultruded FRP which covers material properties, member design and connection design. This includes flexure, shear, tension, compression and combined actions. In summary, the benefits and limitations of pultruded FRP are presented. The content in this paper is intended solely as supplementary commentary and should not be relied upon as a substitute for applicable design standards. Relevant codes and standards remain the primary benchmark and guidance for pultruded FRP design.

1.0 INTRODUCTION

1.1 General

Water tanks are a key infrastructure for maintaining water supply to modern society. In Aotearoa New Zealand and many countries across the world, water tanks are typically designed for higher importance levels and a longer design life than average.

Due to their importance given the role they play in communities, these structures are typically constructed with high strength materials such as steel and concrete. While these materials are well-known, each has different limitations in this context.

Recently, the use of fibre reinforced polymers/plastics (FRP) has increased in the civil engineering industry (Qureshi, 2022). FRP has demonstrated success with strengthening or retrofitting existing steel, timber and reinforced concrete structures. However, through pultrusion machinery, numerous FRP suppliers now offer structural profiles that can be used for member design in a structural system.

Pultruded FRP is a composite material that comprises fibreglass strands held together by a plastic resin matrix as shown in the detail in Figure 1-1. It is naturally corrosion resistant which makes it ideal for liquid-retaining structures exposed to chemicals such as chlorine. As shown in Table 1-1 it trades stiffness for better durability compared with steel and concrete, offering unique solutions for unconventional scenarios.

Table 1-1: Comparative Summary of Key Mechanical and Durability Characteristics of FRP, Steel and Concrete.

	FRP	Steel	Concrete
Chemical Corrosion Resistance	✓		✓
Lightweight	✓		
Low Maintenance	✓		
Ductility		✓	
High Tensile Strength	✓	✓	
High Compressive Strength			✓
High Stiffness		✓	

Recently, Beca collaborated on a project that involved the replacement of a roof structure over an 80 m x 90 m treated drinking water tank for municipal supply. The existing water tank is a buried concrete structure with perimeter walls and interior columns that support the roof.

The existing roof structure consisted of a typical mild steel cladding, purlin and rafter system. As mentioned, FRP is well suited to corrosive environments. Thus, the

PAPER CLASS & TYPE: GENERAL REFEREED

[1] BE(Hons), ME, Beca
[2] BE(Civil), Beca
[3] BE(Hons), Beca

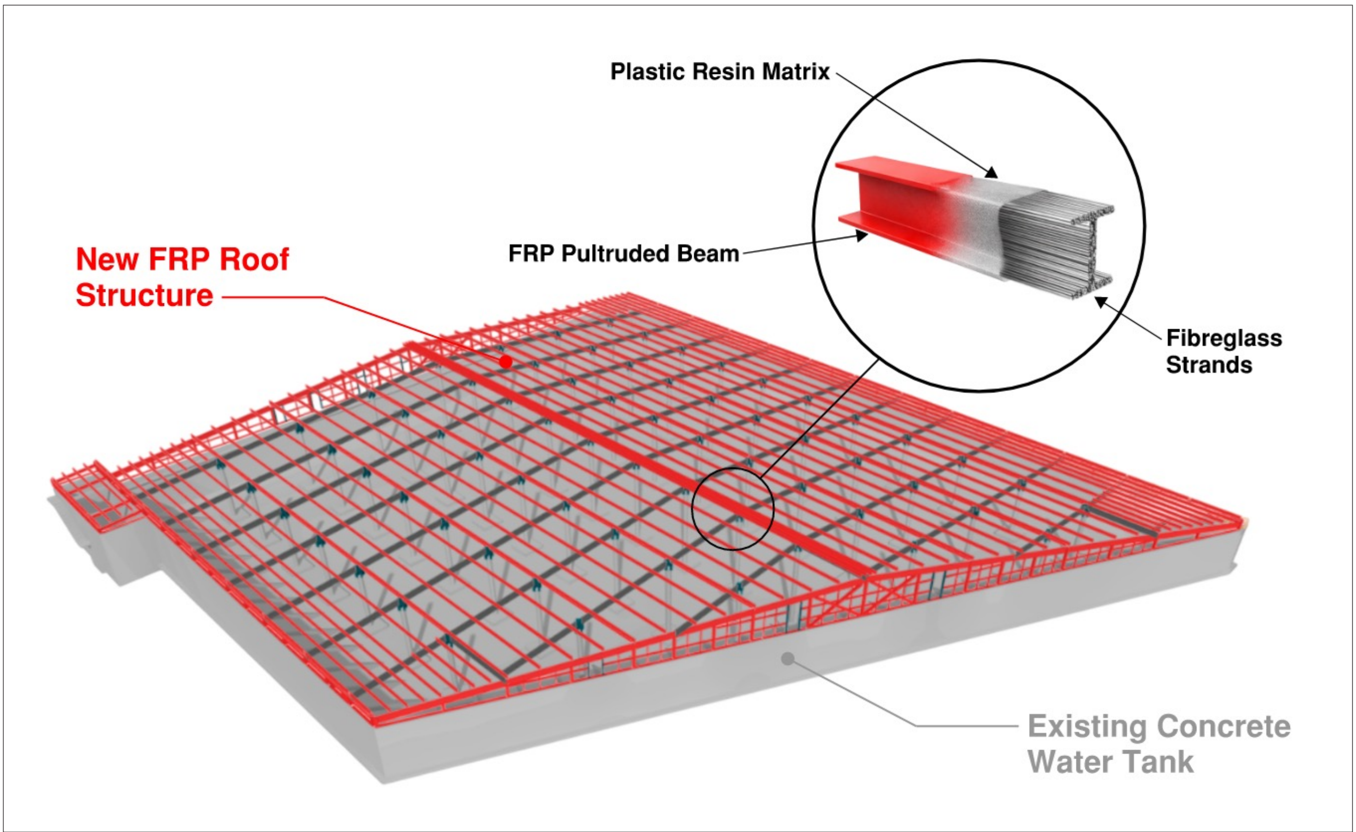

Figure 1-1: Visual Summary of Roof Structure Replacement over 80 m x 90 m Treated Water Tank.

scope of the project was to replace the entire mild steel roof system with FRP members, stainless steel (SS) connections and aluminium cladding.

With no roof bracing in the original design, the lateral system of the roof relied on transferring the tributary area of each reinforced concrete column as a cantilever. For this reason, none of the connections transferred moment and were treated as pinned.

In New Zealand, there is currently no local standard for pultruded FRP design. However, there are several notable standards and guides available internationally:

- **American Pre-Standard** (ASCE, 2010): Pre-Standard for Load & Resistance Factor Design (LRFD) of Pultruded Fiber Reinforced Polymer Structures.
- **American Standard** (ASCE, 2023): Load and Resistance Factor Design (LRFD) for Pultruded Fiber Reinforced Polymer Structures, ASCE/SEI 74-23.
- **European Design Guide** (Clarke, 1996): Structural Design of Polymer Composites: EUROCOMP Design Code and Background Document.

The focus of this project revolved around the American pre-standard (ASCE, 2010) and its official release (ASCE, 2023)

1.2 Aim

The aim of this paper is to highlight the technical insights gained from the design of a new FRP roof structure. Due to the absence of a local standard for pultruded FRP design, this paper will also help bridge application of the American standard ASCE/SEI 74-23 to a structural design located in New Zealand.

For a full disclaimer, this paper should not be used as a primary design guide and should only be used as supplementary information. Existing design standards including ASCE/SEI 74-23 remain the primary guidance for pultruded FRP design. For this project, the design process is illustrated in Figure 1-2 and further expanded in each section of the paper.

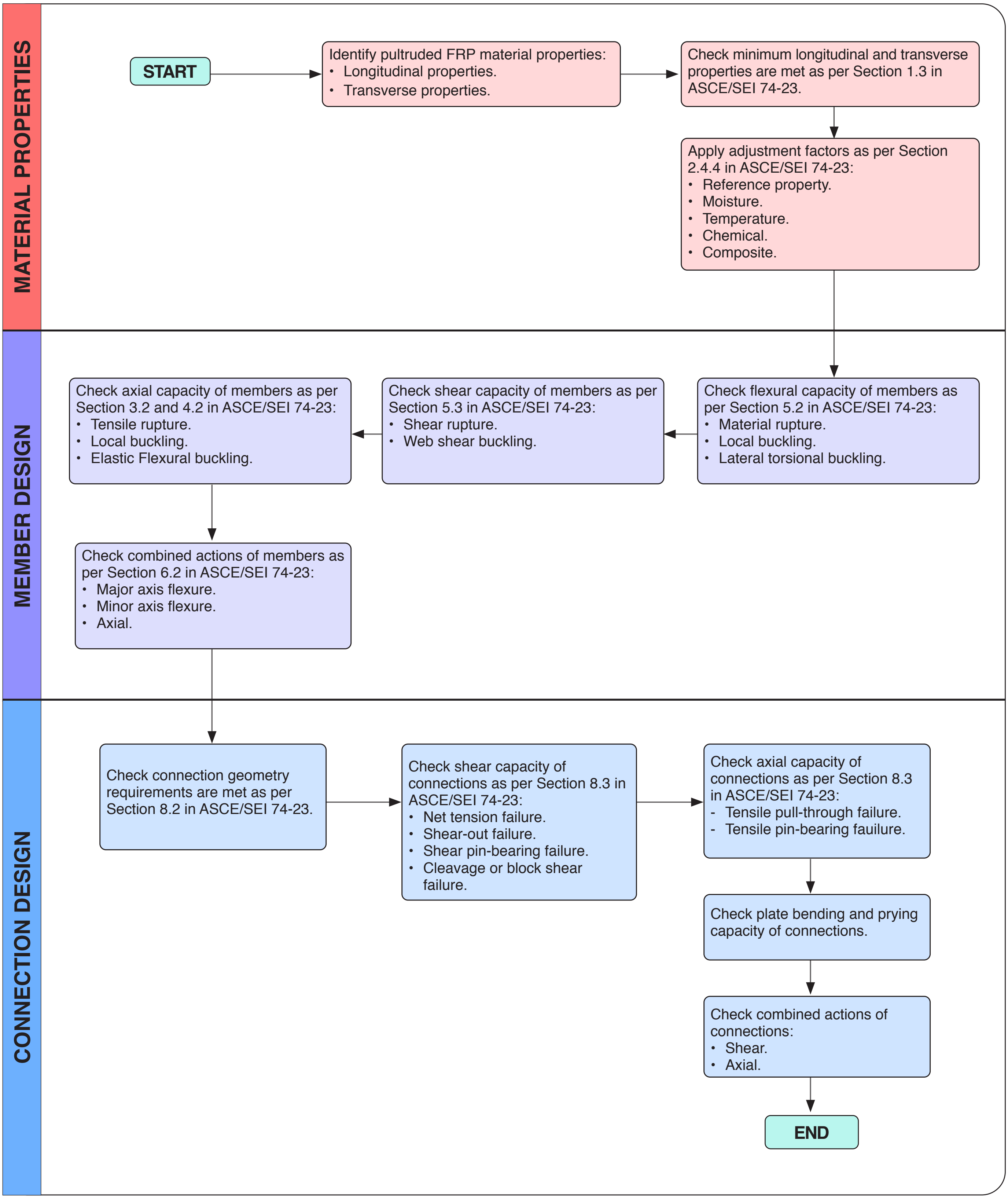

Figure 1-2: Flowchart of Technical Concepts Covered for Pultruded FRP.

2.0 STRUCTURAL DESIGN

2.1 Material Properties

FRP is a composite that is inherently anisotropic where the mechanical properties depend heavily on the orientation and direction of the material. As mentioned, this composite consists of fibreglass pultrusion held together by a resin matrix. Considering a uni-directional layout, FRP exhibits its maximum strength along the longitudinal axis when forces pull parallel to its fibres. However, when loaded perpendicularly, the fibres offer little resistance and rely on the resin matrix which reduces the strength along the transverse axis.

This behaviour mirrors that of timber, which is also an anisotropic material. In timber, the properties are dictated by its grain, which follows the natural growth direction of the wood and is similar to the fibre pultrusion in FRP as illustrated in Figure 2-1.

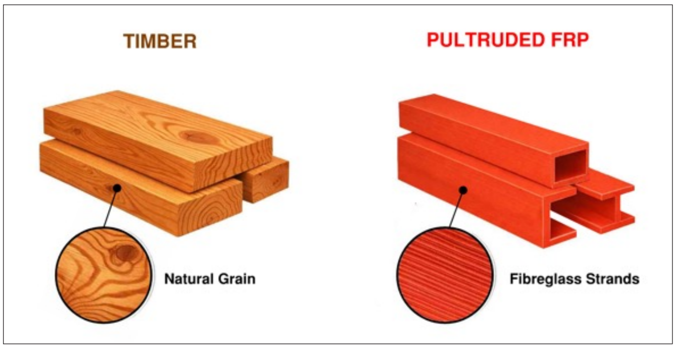

Figure 2-1: Anisotropic Similarities Between Timber and Pultruded FRP.

The strength of FRP will vary between manufacturers. Unlike homogenous materials like steel and concrete, composite material properties are heavily dependent on the manufacturing process and base materials. As shown in Table 2-1, while there is no fixed value, they typically fall between a specific range (20000 – 30000 MPa) in terms of stiffness. Furthermore, while FRP exhibits higher stiffness than timber, it remains significantly less stiff than steel.

Table 2-1: Comparison of the Young's and Shear Modulus for FRP, Steel and Timber.

Material	E (MPa)	G (MPa)
FRP Longitudinal	20,000 – 30,000	3,100
FRP Transverse	5,500 – 7,000	2,900
Steel	205,000	79,300
Timber Longitudinal	8,000 – 14,000	440 – 1,250
Timber Transverse	1,100	50 – 100

2.1.1 Minimum Material Properties and Adjustment Factors

As the mechanical properties for pultruded FRP may vary between manufacturers, ASCE/SEI 74-23 provides minimum required properties as a pre-requisite (ASCE, 2023). When designing with this code, it is important to ensure the properties used from a FRP manufacturer meet these requirements.

In addition to minimum required properties, ASCE/SEI 74-23 also introduces material adjustment factors that alter the nominal mechanical properties used in design (ASCE, 2023). These include factors relating to reference material strength, environmental exposure (moisture, temperature, and chemical effects), and composite assemblies. A key takeaway is that pultruded FRP is highly sensitive to environmental conditions and the nominal strength can vary significantly when compared to isotropic materials such as steel and concrete.

2.2 Member Design

Progressing to the member design of FRP, to recap, all member components of the roof structure were replaced with FRP members. As shown in Figure 2-2 the main components to note included the purlins, rafters and struts. Overall, the final member sizes selected were twice as deep as the previous mild steel members. Additionally, for purlins, I sections were adopted instead of traditional channel sections to achieve more member capacity.

As mentioned, the key trade off with FRP is the significantly lower stiffness. Larger spans and higher design actions will typically require deeper sections than what is expected with steel.

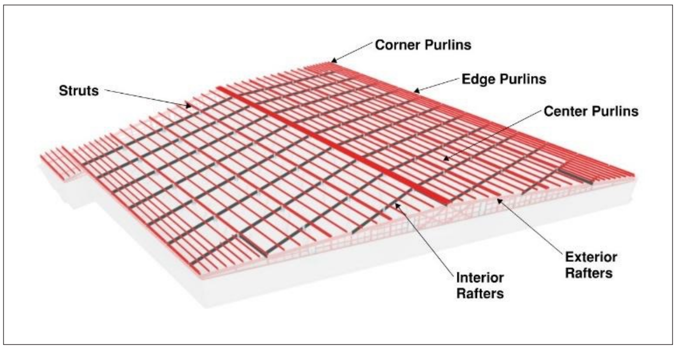

Figure 2-2: FRP structure roof member components.

2.2.1 Flexure

Flexure is critical for the design of FRP members. There are three main failure modes:

- Lateral Torsional Buckling (LTB).
- Local Buckling (LB).
- Flexural Rupture.

Similar to steel design, LTB will typically govern the member capacity. However, if sufficient lateral restraint has been provided, LB will govern instead. Rupture is the least common failure mode and is associated with the mechanical nature of the material.

On a related note, the flexural design formula for FRP is relatively similar to that of steel. As an example, focusing on LTB, Equation 2-1 (ASCE, 2023) from ASCE/SEI 74-23 shares the same principles of Equation 2-2 (Standards New Zealand, 1997) from New Zealand steel design standard NZS 3404. Both equations have a torsional, warping and buckling component.

$$M_n = C_b \frac{\pi}{L_b} \sqrt{E_{L,f} I_y G_{LT} J + \left(\frac{\pi E_{L,f}}{L_b}\right)^2 I_y C_w} \qquad (2\text{-}1)$$

$$M_{oa} = \sqrt{\left\{\left(\frac{\pi^2 E I_y}{L_e^2}\right)\left[GJ + \left(\frac{\pi^2 E I_w}{L_e^2}\right)\right]\right\}} \qquad (2\text{-}2)$$

Where:

- E, G = Young's and shear modulus
- $E_{L,f}$ = Longitudinal flange Young's modulus
- G_{LT} = In-plane shear modulus
- J = Torsional constant
- I_y = Second moment of inertia about the y axis
- I_w, C_w = Warping constant
- L_b, L_e = Points between restraints, effective length
- C_w = LTB modification factor

While these formulas are similar, due to the anisotropic nature of FRP, the longitudinal and transverse properties will vary. It is important to ensure that the direction of the fibres has been considered to avoid incorrectly deriving the member capacity for FRP.

Furthermore, unlike the pre-standard which provided LTB capacities for square and rectangular sections (ASCE, 2010), ASCE/SEI 74-23 only includes provisions for I sections. For all other section types, engineering judgement is required to determine the LTB capacity.

2.2.2 Shear

As with steel, shear typically does not govern the design for FRP members. There are two main failure modes, both of which generally have high capacities:

- Shear Rupture.
- Web Shear Buckling.

In addition to these failure modes, ASCE/SEI 74-23 has several specific shear requirements for concentrated loads. There is an additional check for:

- Material Failure at Web-Flange Juncture.
- Web Compression Buckling.

Focusing on web-flange juncture failure, members with a depth greater than 305 mm require vertical bearing stiffeners at all interior locations directly under a concentrated load (ASCE, 2023). As an example, illustrated in Figure 2-3, the 610 mm deep rafter of the roof structure should require stiffeners beneath every purlin and above every intermediate column.

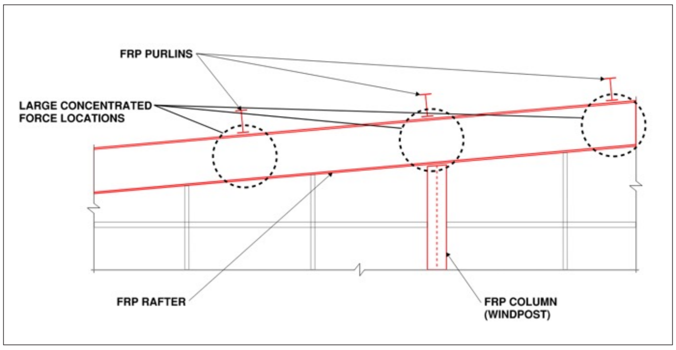

Figure 2-3: Concentrated Force from Purlins and Column acting on Rafter.

Given the scale of the roof, the number of required locations can be substantial. It is also important to highlight that FRP cannot be welded; hence, the installation of stiffeners can be difficult.

While there is a stiffener requirement for deep beams (> 305 mm), Borowicz & Bank (2013) have demonstrated that deep pultruded FRP beams with a span-to-depth ratio of less than 4:1 experienced web buckling before material failure at the web-flange junction, so there is a need for engineers to exercise some judgment regarding the requirement of stiffeners.

2.2.3 Axial Capacity

In terms of tension, FRP has good tensile strength and is only governed by rupture. For compression, FRP has less capacity and has two main failure modes:

- Axial Local Buckling (LB).
- Elastic Flexural Buckling.

For axial LB, both flange and web buckling needs to be considered. Similarly, both the major and minor axis for elastic flexural buckling needs to be considered. Depending on the lateral restraint provided about the major and minor axis of a section, either failure mode can govern. Moreover, ASCE/SEI 74-23 also requires an additional axial capacity check for serviceability limit state (SLS) design actions. However, this does not usually govern the design.

2.2.4 Combined Actions

Lastly, ASCE/SEI 74-23 requires a combined actions check for concurrent design actions. This consists of major axis flexure, minor axis flexure and axial. This check is most likely to govern the overall design if the member is subjected to multiple design actions.

2.3 Connection Design

For connections, an important point to raise is that pultruded FRP members can only be connected through bolted connections. Unlike steel, there is no option for welding.

In ASCE/SEI 74-23, only design guidance for simple shear and axial checks are given (ASCE, 2023). The standard does not provide guidance for bolted connections for complex moment frames. For this reason, FRP has a limitation when it comes to designing full moment connections. Regarding the roof structure as shown in Figure 2-4, all connections were designed as pinned with a simple bolted SS cleat.

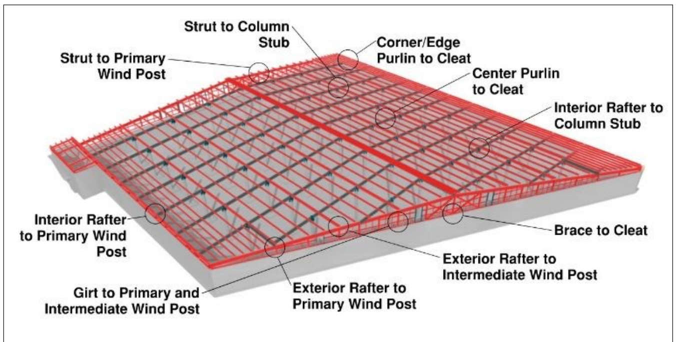

Figure 2-4: FRP Structure Connection Components.

2.3.1 Shear

Shear design is critical with FRP. Like timber, FRP has requirements for bolt edge distance and spacing. As a brief overview, taking a 2 x 2 geometry as an example in Figure 2-5, the failure modes for FRP mirror that of steel bolted connections.

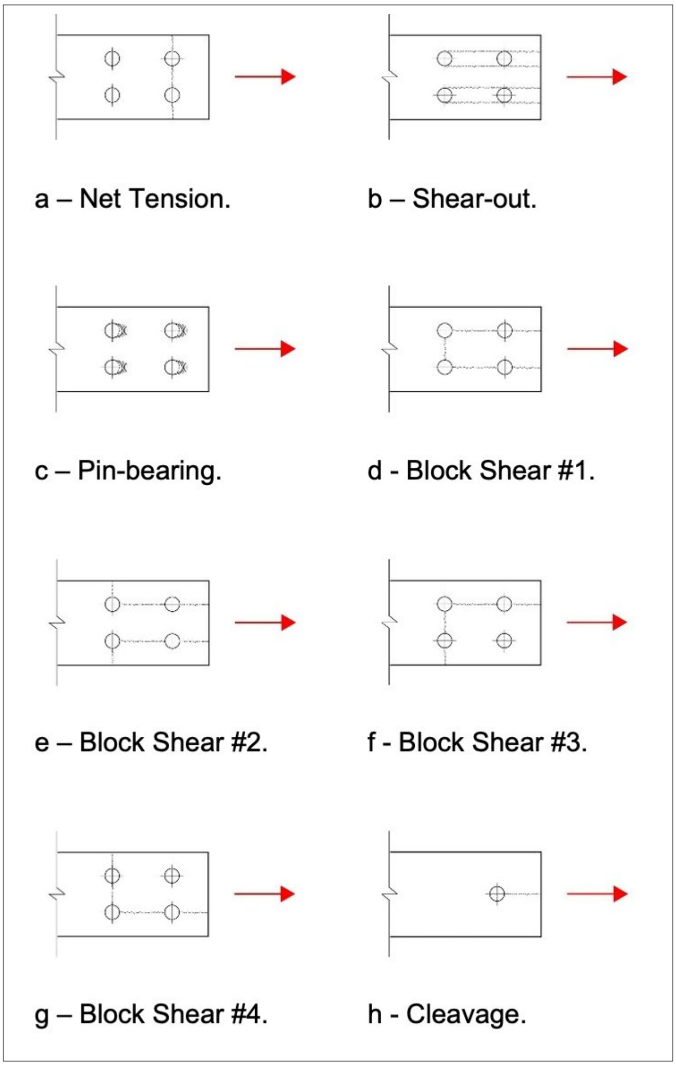

Figure 2-5: Types of FRP Shear Failure Modes.

Note that some configurations can alter the capacity of the connection. For example, if both edges parallel to the force are bounded by a flange, net tension need not be considered. Further to this, there is an additional geometry requirement in ASCE/SEI 74-23 if the net tension capacity in the code is utilised; otherwise, experimental testing is required (ASCE, 2023). Similarly, for shear-out, this failure mode also need not be considered if the perpendicular edge of the connection is bounded by a flange. Further to this, it is also recommended that connections be detailed symmetrically with brackets on both sides to maximise capacity. Although single lap shear configurations are possible, they result in reduced capacity.

For most cases, typically net tension or pin-bearing will govern the design. Cleavage only applies for connections with one row of bolts and block shear applies for connections with multiple bolt rows.

It is worth noting that ASCE/SEI 74-23 only covers connections within a 3 x 3 geometry. To derive the capacities of larger bolt groups and staggered bolts, experimental testing is required (ASCE, 2023).

2.3.2 Axial

For axial, as illustrated in Figure 2-6, the primary failure mode is bolt pull-through of the material. Axial forces typically do not govern the design for connections. The design actions are usually imposed by the minor axis shear and tension force of a member.

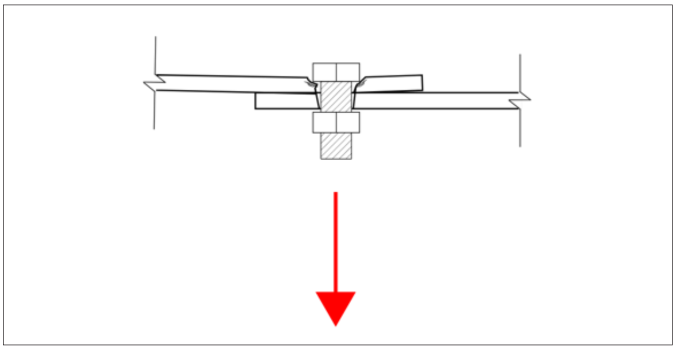

Figure 2-6: Tensile Axial Pull-through Failure.

2.3.3 Plate Bending and Prying

While FRP axial failure is uncommon, if the adjacent plate being bolted towards is FRP, the capacity of that plate may govern the connection design instead. Taking an example from the roof structure, in Figure 2-7 the strut imposes an axial force through a bolted connection to the flanges of the column. In this case, flange bending will govern before any axial or shear failure of the other components in the connection.

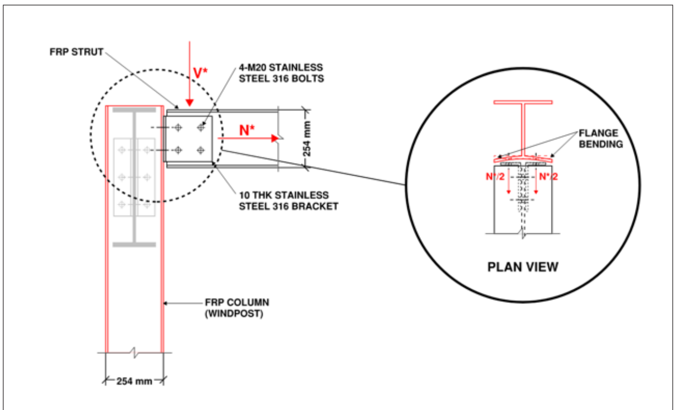

Figure 2-7: FRP Flange Bending Example for Strut Connection.

Due to the lower stiffness of FRP, bending in plates of a connection system can be critical. Although not explicitly covered in ASCE/SEI 74-23, it is important to consider the prying and flexural capacity of the flange plate as required in NZS 3404 (Standards New Zealand, 1997) for a typical bolted steel connection.

2.3.4 Combined Actions

Additionally, another key concept that is not covered in ASCE/SEI 74-23 is the consideration of combined actions subjected to a bolt group, taking the rafter bolt group connection for the roof structure as an example. As the rafter is subjected to simultaneous design actions, the combined shear and axial force as shown in Figure 2-8 is typically critical.

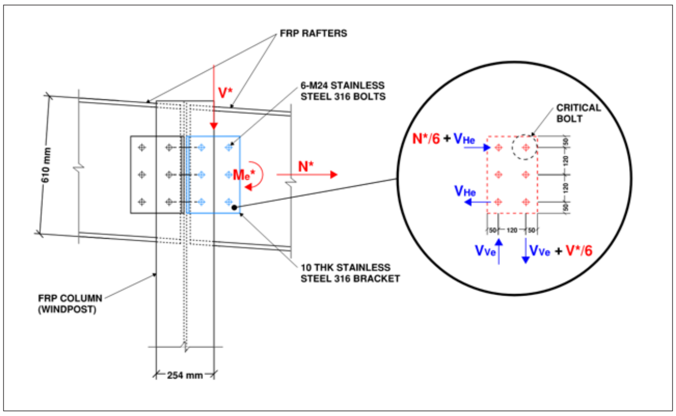

Figure 2-8: Combined Actions Example for Rafter Connection.

Connections in structural systems are often subjected to concurrent design actions which should be considered to determine critical shear. While ASCE/SEI 74-23 does not have a requirement for resolving these loads, it is recommended that the forces in connections with bolt groups are resolved and compared against the failure mechanisms listed in Figure 2-5. Guidance on resolving bolt groups can be adopted from Australian steel standard AS 4100 (Standards Australia, 2020).

3.0 DISCUSSION

Overall, this project demonstrated that FRP is well suited to environments where durability and corrosion resistance are primary drivers. The replacement of the existing mild steel roof system with pultruded FRP members and SS connections provided a solution to reduce maintenance requirements and offer constructability advantages due to its lightweight nature (Qureshi, 2022).

However, while FRP has its benefits, several important technical limitations were identified throughout the design process and are important to consider:

- With an absence of a design standard particular to FRP in New Zealand, local engineers may find it difficult to adopt international standards.
- Members selected were twice as deep with limited stiffness for long-span applications.
- With no option for welding, connections are limited to simple bolted configurations.
- There are limited design provisions for bolt group behaviours and complex moment connections.

With a much lower stiffness, FRP may not be a suitable alternative for very large spans. Regarding the anisotropic nature of pultruded FRP, this also required careful consideration as it may not be suitable to all structural systems. Unlike isotropic materials, mechanical properties varied depending on fibre orientation making it critical to ensure correct longitudinal and transverse properties were specified in the design.

Furthermore, as mentioned, using only bolted connections limits the use of FRP to structural systems with simple connections that do not transfer moments. ASCE/SEI 74-23 covers only the basics of connection design and, in particular, prying and combined action checks are not covered. This highlights an area where further code guidance would assist designers using FRP.

4.0 CONCLUSION

In some settings, pultruded FRP can offer a durable, low-maintenance solution for structures exposed to corrosive environments, outperforming conventional materials like mild steel in long-term serviceability (Ghadimi et al., 2017). This paper demonstrates its effectiveness in water tanks subjected to chlorine exposure. However, several design limitations were identified, including the material's anisotropic nature, lower stiffness and connection detailing limitations.

Despite these considerations, FRP has potential to be an innovative solution for projects beyond liquid retaining structures. With continued development of structural standards and increased industry familiarity, its adoption in future infrastructure projects is expected to expand.

5.0 ACKNOWLEDGEMENT

The authors would like to acknowledge Beca for enabling this project to be conducted and completed. In addition, a very special thanks to Jacopo Siracusa, Technical Director of Beca, for his invaluable advice and review of this paper.

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SEISMIC PERFORMANCE AND LOSS ASSESSMENT OF LIGHT TIMBER FRAME RESIDENTIAL HOUSES IN NEW ZEALAND: STATE OF THE ART

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ABSTRACT

Past earthquake experiences in Aotearoa New Zealand indicate that light timber-frame (LTF) residential housing stock in Aotearoa NZ could suffer significant damage in major earthquakes, leading to considerable downtime and economic losses for the community. It is necessary to develop a rigorous approach to predict seismic damage on LTF residential houses and estimate the subsequent economic losses. This paper provides an overview of recent research advances in the fields of seismic performance assessment and seismic loss models for LTF residential houses in Aotearoa NZ. The authors systematically review the evolution of residential houses in Aotearoa NZ, experimental and simulation studies of plasterboard bracing walls and LTF buildings, numerical modelling methods currently used for wood shear walls, and prevailing building seismic loss estimation models. In addition, recent technological advancements and current design recommendations relevant to such LTF houses and bracing walls are examined. Possible future research directions are recommended in order to better understand seismic performance and develop a loss estimation framework for LTF residential houses in Aotearoa NZ.

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INTRODUCTION

Generally, in light timber-frame (LTF) residential houses, walls provide stiffness and resistance to lateral wind and seismic forces [1]. In North America, LTF shear walls are commonly sheathed with plywood or oriented strand board (OSB) and fastened with nails. However, in Aotearoa NZ, plasterboard bracing walls are widely used as the gravity and lateral load-resisting systems for LTF residential houses [2]. Most residential houses in Aotearoa NZ are low-rise (single or double storied), over 90% of which include LTF proprietary bracing wall systems made of plasterboard [3]. A plasterboard bracing wall has plates and studs made of timber, sheathed on one side or both sides with plasterboard panels . Optional fixing methods for installing plasterboard to timber framing include adhesive, screws, and nails, with glue-and-screw being the more commonly used fastening system. Edges of the sheathing panels can be blocked or unblocked. The plasterboard for walls is also known as gypsum wallboard (GWB) or drywall.

Low-rise LTF structures normally have a low probability of collapse under earthquakes because wood is a material with a high strength-to-weight ratio. LTF residential houses are lighter than concrete and steel buildings of similar sizes, thus attracting lower seismic forces. Typically, LTF shear walls have intrinsic redundancy which makes the whole structure very robust against collapse which means that LTF residential houses are not very prone to structural failures and therefore achieve life safety objectives [4]. According to previous research [5, 6], low-rise LTF houses could sustain a storey drift of 6% before reaching the collapse limit state.

However, collapse avoidance is not the only target of seismic design. LTF residential houses may still suffer severe damage under high-intensity earthquakes even if they do not collapse. For example, in the 2010 Darfield earthquake and the 2011 Otautahi Christchurch earthquake, unprecedented damage to LTF residential houses with plasterboard walls was recorded. The

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estimated total economic losses to residential houses caused by the 2010-11 Waitaha Canterbury earthquake sequence was around $12B, about 30% of the total losses [7].

Over the years, many studies have investigated the seismic performance of plasterboard bracing wall systems and explored empirical seismic loss models for typical Aotearoa NZ residential houses. However, no studies have systematically scrutinised the progress made on these topics and reviewed the current state-of-art to identify the knowledge gaps and needs for further research, with a view to enhancing the seismic design and performance of LTF residential houses. Moreover, the relationships between earthquake intensity, seismic damage, and economic loss for Aotearoa NZ LTF residential houses are not well understood. This paper revisits and summarises the development of bracing wall systems in Aotearoa NZ residential houses, and reports on the characteristics of plasterboard bracing walls. It also reviews experimental, numerical and analytical investigations on plasterboard bracing walls and LTF houses, and explores current literature to understand the seismic loss assessment models used for LTF residential houses in Aotearoa NZ. While the scope of this paper is primarily limited to conventional construction materials and methods used in Aotearoa NZ residential houses, some overseas studies are included to demonstrate the current state of knowledge and research on these topics in other countries.

History of Residential Housing and Development of Bracing Walls in Aotearoa NZ

In Aotearoa NZ, there are three predominant residential housing typologies: (a) typical 1930s timber frame bungalows, (b) 1940-1960 timber frame houses, and (c) post-1980s brick veneer timber frame houses (Figure 1). They make up over 95% of the residential houses stock (by value and quantity) [7]. Aside from these three types, other typologies include the pre-1940 unreinforced masonry houses (which were phased out of construction following the 1931 Ahuriri Napier Earthquake), the post-1980s houses with reinforced concrete tilt-up slabs, and houses made of reinforced hollow concrete blocks [7].

The lath-and-plaster system is the earliest internal lining used for timber walls in Aotearoa NZ [8]. In this system, wood laths are nailed across the wall studs and plaster is forced into the gaps between the laths and covers the full wall [9]. Lath-and-plaster can only provide minimal lateral capacity and fails in a brittle mode at low loads. The main lateral load resistance comes from the diagonal braces. As shown in Figure 2, some braces are cut between studs, and some are fitted into slots cut into the studs. According to the Waitaha Canterbury earthquake survey [5], the use of lath-and-plaster on the exterior of houses was common in early 1900s houses. Some cases were observed where sheets of the plaster were detached from the lath, and both the plaster and the lath broke away from the wall (Figure 3).

Figure 1: Predominant residential housing typologies: (a) typical 1930s timber frame bungalow; (b) 1940-1960 timber frame house; (c) post-1980s brick veneer timber frame house [7].

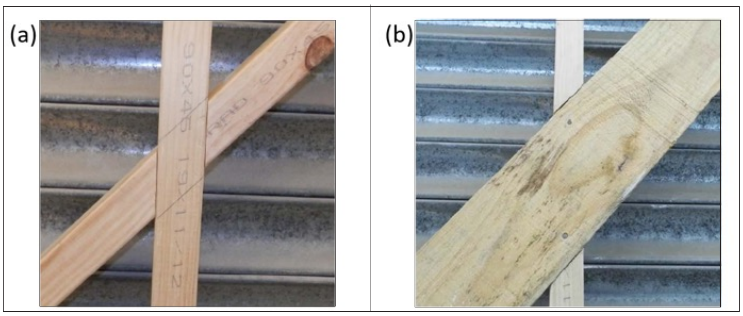

Figure 2: Diagonal timber braces: (a) cut between studs; (b) a let-in brace [10].

Figure 3: A failure example of lath and plaster wall [5].

Weatherboard is commonly used in the exterior walls of the pre-1940s timber frame bungalows. As shown in Figure 4, the weatherboard is fixed to wall studs with nails at some distance from the bottom of each weatherboard. The resistance of this kind of wall is expected to be provided by the moment couples between the horizontal lines of nails and the friction of one board against the next. Figure 5 illustrates the hysteresis loops of a 2.4 m long weatherboard wall tested by BRANZ [9]. Although the hysteretic load-drift curves of the weatherboard wall were fat and stable, the maximum load was very low: around 1 kN. Therefore, weatherboards cannot be considered as bracing materials. The interior side of the wall needs to be lined with much stiffer panels in order to provide bracing capacity.

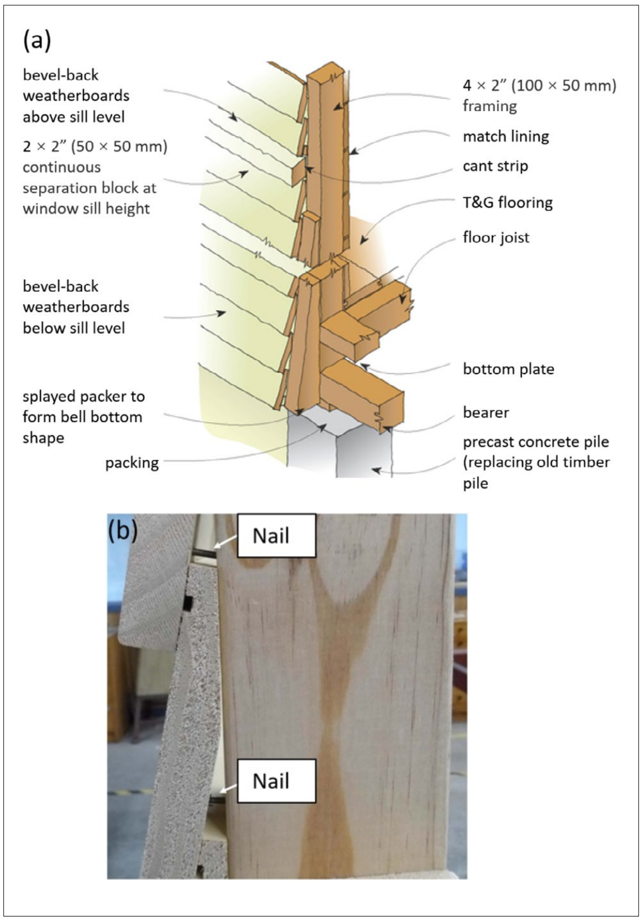

Figure 4: Bell-cast horizontal weatherboard: (a) overall construction [13]; (b) nails detail [9].

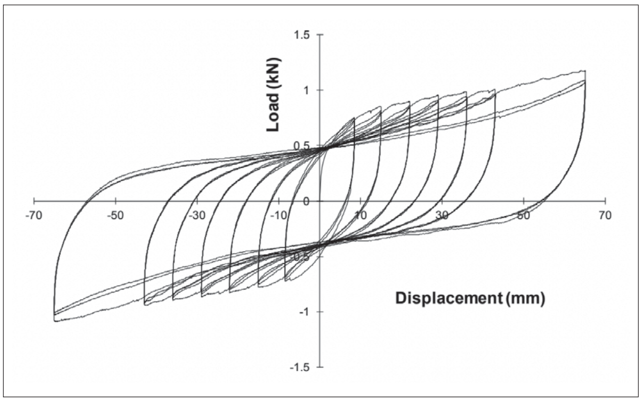

Figure 5: Hysteresis loops of a 2.4 m long weatherboard wall [9].

Fibrous plaster sheets were first introduced during the 1920s and 1930s in Aotearoa NZ [11], and developed to replace the lath-and-plaster system [8]. Fibrous plaster, also known as Hessian fibre-reinforced gypsum, is a type of plasterboard sheet reinforced with a mixture of fibres. Figure 6 illustrates the construction of fibrous plaster. Beattie et al. [8] stated that the product fitted within the description of a generic bracing system in the early versions of NZS 3604 [12] and was expected to act as a bracing element. However, the bracing capacity of fibrous plaster sheets is very low, and the diagonal braces mainly provide the lateral capacity.

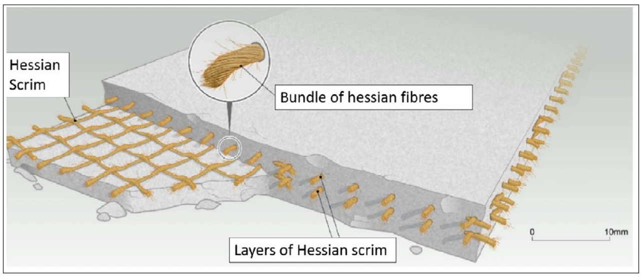

Figure 6: Section of fibrous plaster with exposed scrim layer [14].

By the middle of the 20th century softboards and hardboards were commonly used for lining [5]. Softboards and hardboards are defined as wood fibreboard with low and high densities. Softboards are usually used for lining living rooms and bedrooms, whereas hardboards are used for lining utility rooms, i.e. bathrooms, kitchens and laundries. Softboards are generally fixed with steel clouts or glue, while hardboards are fastened with brads. They also provide little bracing capacity.

Plasterboards have been commonly used in Aotearoa NZ since the 1920s and domestic manufacturing began in 1925. Plasterboards and wood-based panels became the predominant wall-lining materials in the 1930s [11]. Using plasterboards has several advantages over timber-based boards including lower material cost, and better fire protection [15]. Compared with the earlier versions of bracing wall systems, plasterboards can meet greater bracing demand in modern LTF houses [8]. Therefore, diagonal bracing is no longer needed in plasterboard bracing walls. A typical plasterboard bracing wall is shown in Figure 7. The bottom plates of the walls are bolted or coach-screwed to the foundation beam. The plasterboards are fixed to the timber frame by fasteners (normally by screws). Sometimes hold-downs are used at the wall ends. Plasterboards used in Aotearoa NZ LTF walls include standard plasterboard, bracing plasterboard with a higher density core or fibreglass reinforcing in its core, fire-resistant plasterboard and water-resistant plasterboard. Plasterboard products have to conform to manufacturing and performance specifications within AS/NZS 2588 [16].

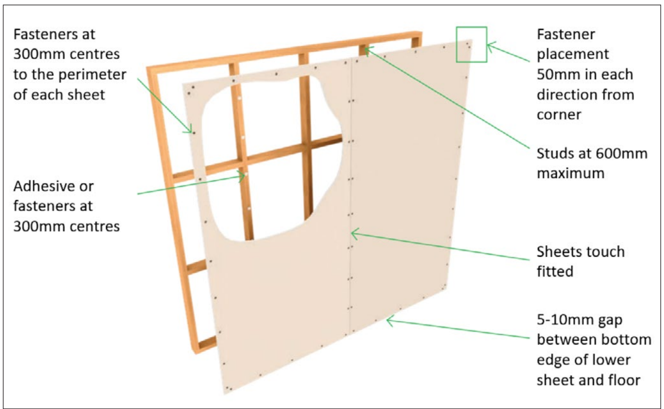

Figure 7: A typical plasterboard bracing wall [17].

Characteristics of Plasterboard Bracing Walls

General Performance of Plasterboard Bracing Walls

Plasterboard bracing walls in Aotearoa NZ residential houses are the main structural element that resists in-plane shear forces. In other countries where LTF houses are widely used, such as the United States and Canada, LTF walls are usually sheathed with plywood sheets or oriented strand boards (OSB) on one side only, or alternatively with plywood or OSB one side and plasterboard on the other [1, 15, 18]. Regardless of the material of the panels used, these walls can all be categorised as light wood-frame shear walls sheathed by panels. They have similar mechanisms in resisting the racking loads. However, because plasterboards are weaker and more brittle than wood-based panels, plasterboard bracing walls have different racking responses compared to bracing walls sheathed with mainly wood-based panels [10, 19].

Chen et al. [1] tested and compared the performance of OSB sheathed walls and Type X plasterboard sheathed (on one side) walls used in Canada. Type X plasterboard is a special fire-resistant plasterboard popular in North America. Special glass fibres are intermixed with gypsum to reduce the size of the cracks that form as the crystalline water is driven off during fire, thus extending the length of time the panels maintain their structural integrity. The common thicknesses of Type X plasterboard include 12.7 mm, 15.9 mm and 25.4 mm. In the Aotearoa NZ market, there are some similar plasterboards which have fire ratings longer than the standard plasterboards. Available thicknesses include 10 mm, 13 mm, 16 mm and 19 mm. Figure 8 illustrates the load-displacement hysteretic responses of two walls of the same size and made of Canadian Spruce-Pine-Fire framing members. The only difference was that SW-01 was sheathed by 12.5 mm thick OSB on one side using 8d (ø3.5 × 63.5 mm) common wire nails while SW-03 was sheathed by 15.9 mm Type X plasterboard using drywall screws #6 (ø2.87 × 50.8 mm). The spacing scheme of the nails and screws is the same, i.e. spacing at 152 mm on the centre along the panel edges and 305 mm along intermediate studs. The results showed that the plasterboard walls had much lower strength, less energy dissipation, and lower deformation capacity and ductility.

Wang et al. [19] collected a series of P21 test data (introduced in the next section) of bracing walls used in Aotearoa NZ LTF houses and analysed the effect of the sheathing material on the lateral performance of the walls. In these wall specimens, plasterboards were fixed to timber framing with screws, while plywood panels were fixed with nails. The average maximum loads and the average drift ratios at the maximum loads of the walls sheathed by different materials are shown in Figure 9(a) and 9(b), respectively. Each pair of two adjacent columns represents the values of two wall types with the same construction details but different sheathing materials. The maximum loads of plasterboard bracing walls were lower than that of plywood sheathed walls, and the drifts of plasterboard bracing walls at the maximum loads were also lower than those of the plywood sheathed walls. The test results indicated that, compared to plywood sheathed walls, plasterboard bracing walls are less ductile with lower energy dissipation capacity.

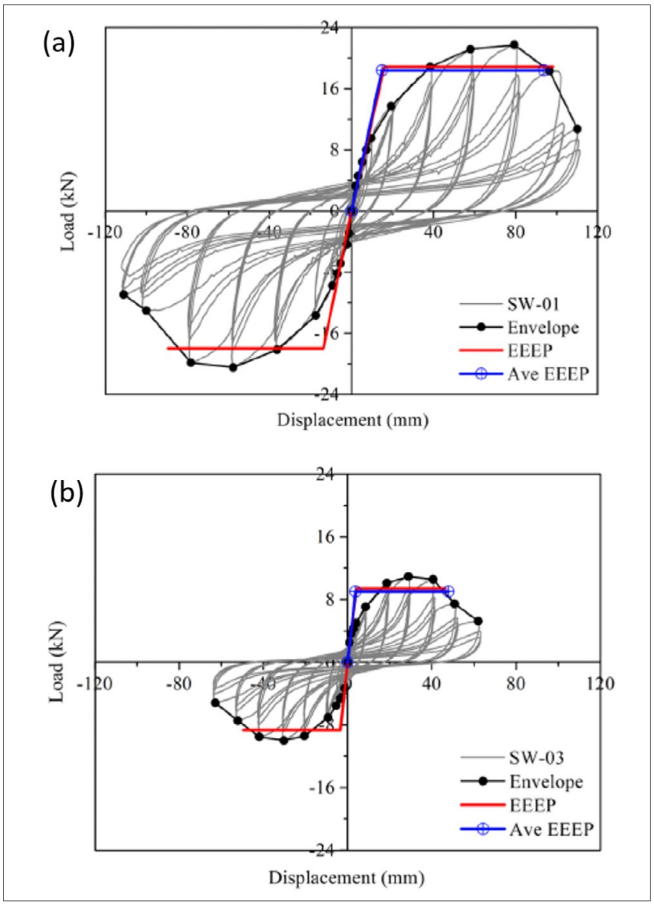

Figure 8: Load-displacement curves of shear walls under reverse cyclic loading: (a) shear wall sheathed by OSB; (b) shear wall sheathed by plasterboard [1].

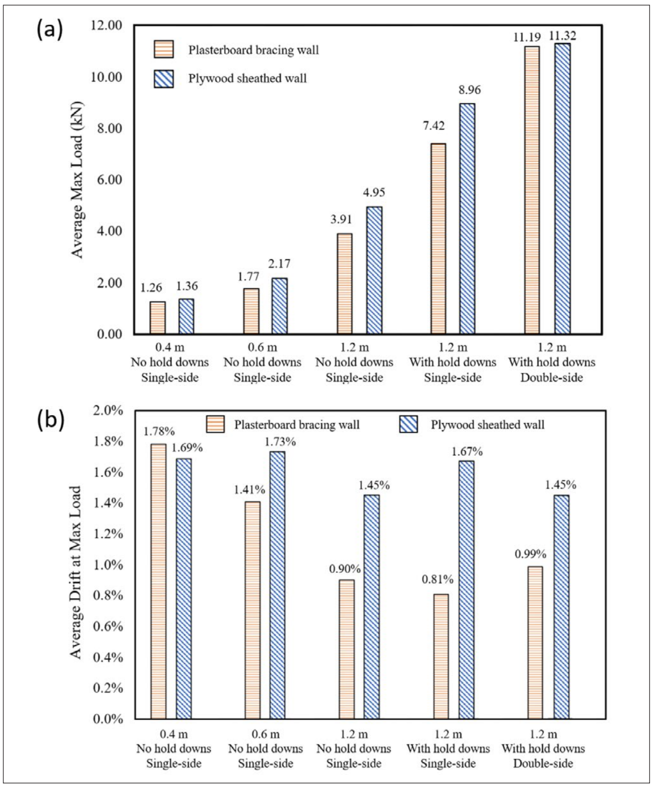

Figure 9: Effect of sheathing material on: (a) maximum loads; and (b) drifts at the maximum loads [19].

Plasterboards are also used as linings for other wall systems. For example, they can be installed as interior linings of light (cold-formed) steel frame load-bearing walls. Previous studies [20, 21] found that plasterboard can increase initial stiffness and modestly increase the strength of the walls. However, plasterboards are not considered the main bracing material in this system, and the performance of light steel frame walls is different from that of LTF plasterboard bracing walls. Plasterboard-sheathed timber walls are also used as infill walls in reinforced concrete (RC) frames and steel frames [22, 23]. In such systems, plasterboard infill walls are not designed to be lateral load-resisting elements, but rather non-structural elements. A series of quasi-static tests on plasterboard infill walls within RC frames were performed by Tasligedik et al. [24]. The cyclic performance of the plasterboard infill walls showed a higher peak load and lower drift ratio at the peak load compared to structural plasterboard walls.

In summary, the performance of plasterboard bracing walls used in Aotearoa NZ is different from that of walls sheathed by wood-based panels, light steel frame walls with interior plasterboards, and infilled plasterboard walls in RC/steel frames. Many overseas researchers have studied the effects of plasterboard on the seismic performance of LTF shear walls braced by wood-based panel products [18, 25, 26], and the performance of the whole LTF houses [27–30]. However, there are only a few Aotearoa NZ-based studies on plasterboard bracing walls. As plasterboard bracing walls are the main lateral load-resisting systems of typical Aotearoa NZ LTF houses, developing a good understanding of their seismic behaviour is key to a reliable assessment of the seismic performance of Aotearoa NZ LTF residential housing stocks.

Timber-frame Bracing Design in Aotearoa NZ Standards

In Aotearoa NZ, buildings are designed to resist structural design actions, the general principles of which are outlined in AS/NZS 1170.0 [31]. AS/NZS 1170 Parts 1, 2, 3 and NZS 1170.5 [32] specify the permanent, wind, snow and ice, and earthquake design actions, respectively. For timber-framed residential houses, NZS 3604 [12] is referenced as an Acceptable Solution for Building Code clause B1 Structure (buildings will withstand likely loads, including wind, earthquake, live and dead loads). It provides methods and details for Aotearoa NZ timber-framed houses and small buildings for achieving code compliance.

Current seismic design standards are generally developed to achieve life safety at ultimate limit state (ULS) events and control deflections at serviceability limit state (SLS) events. The inter-storey deflection limit at ULS in NZS 1170.5 is 2.5% of the corresponding storey height or lesser as may be prescribed in the appropriate material standard. For SLS, the drift limit is specified as 0.33%. LTF houses designed per NZS 3604 could easily achieve life safety performance targets at design-level earthquakes [2].

In NZS 3604, the earthquake bracing demand is determined by the building location, subsoil type, the building size, roofing and cladding weights, and floor live loads. The demand is developed based on the equivalent static method which is a force-based approach according to NZS 1170.5. The design base shear force, V, is determined by the following equation:

$$V=C_d\,(T_1)\,W_t \tag{1}$$

where $C_d\,(T_1)$ is the horizontal design action coefficient derived by assuming a ductility of μ and a fundamental period of T_1, and W_t is the seismic weight. The equivalent static horizontal force (F_i) at each level (i) is obtained from the following equation:

$$F_i = F_t + 0.92V\frac{W_i h_i}{\sum_{i=1}^{n} W_i h_i} \tag{2}$$

where F_t = 0.08V at the top level and zero elsewhere, W_i is the seismic weight of level i, and h_i is the height of level i. The earthquake forces (bracing demand) are also presented in "bracing units" (BUs) where 1 kN equals 20 BUs.

NZS 3604 [12] specifies so-called P21 tests to evaluate the bracing ratings of bracing wall elements. The P21 test method, developed by BRANZ [33], aims to determine the seismic bracing capacity of proprietary LTF shear walls and ensure that these walls have adequate strength, stiffness, elastic recovery, and resistance under cyclic loads. Figure 10 shows the P21 test setup. The P21 test is a slow cyclic racking test performed by applying a lateral load at the top of the test specimen. The bracing rating of a specified bracing wall system is determined by experimentally subjecting three nominally identical full-scale specimens to an incremental series of cyclic lateral in-plane displacement sets and measuring the force that the wall resists within a defined displacement range. P21 tests are often conducted on a standard wall length of 1.2 m. For longer walls up to 2.4 m in length, the seismic rating per metre length is assumed to be the same as for 1.2 m long walls [4]. Overall, the P21 test method is similar to other overseas test methods for lateral force resisting systems such as the ASTM E2126 standard. One difference is that in the ASTM E2126 standard, the racking load is applied to the test specimen through a load beam which is fixed to the top plate of the test wall, whereas the P21 test specifies that the horizontal load is applied in the middle of the test wall, as shown in Figure 10(a). In addition, the P21 test uses supplementary uplift restraints at each end of the test specimen. Construction details of the restraint are shown in Figure 10(b). A bolt or coach screw providing a sliding attachment between the angle and the end of the specimen through a slotted hole is also acceptable.

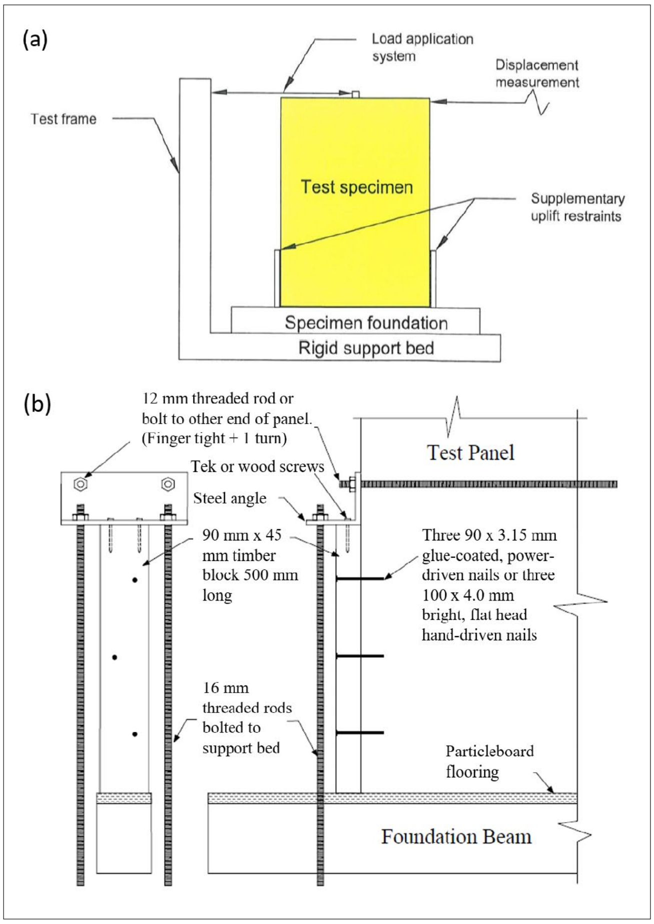

Figure 10: P21 Test arrangement [12]: (a) setup; (b) supplementary uplift restraint.

Apart from satisfying the bracing demand, the bracing elements are required to be evenly distributed along notional "Bracing Lines" in each direction (along and across the ridge) of the building. It is specified that the bracing lines in any storey shall be placed at not more than 6 m centres apart. On each bracing line, the minimum bracing provision is the greater of 100 BUs or 50 % of the total bracing demand divided by the number of bracing lines in the direction being considered. Besides, the minimum bracing resistance for each external wall in any storey shall be no less than 15 BUs/m of external wall length. As a result, LTF residential houses constructed to NZS 3604 have an 'egg-crate' structural form and are considered to be reasonably regular both in plan and elevation [4].

The wall bracing demand in the current version of NZS 3604 (revised in 2011) has been re-examined by several studies. Liu [2] analysed the expected earthquake performance of a case study LTF residential house with the minimum standard seismic bracing provision by using the direct displacement-based approach. The deflection requirement at ULS for plasterboard bracing walls was determined to be 1% storey drift based on the available P21 test results. Figure 11 shows the relationship between the response acceleration (Sa) and response displacement (Sd) for the site of the case study building, which was calculated by the direct displacement-based method. The bracing capacity of this building is equivalent to Sa=0.4g. As shown in Figure 11, the bracing walls need to deflect to 70 mm (3.0% drift) in a 500-year event even if the bracing system could maintain strength and 20% equivalent viscous damping beyond 22mm deflection (i.e., 1% drift). It was concluded that the expected seismic deflection of the conventional LTF house (with plasterboard bracing walls) designed per NZS 3604 would be larger than the specified deflection limit of 2.5% storey drift at ULS. The author suggested the seismic bracing demand in NZS 3604 potentially needs to be increased by 40% at ULS.

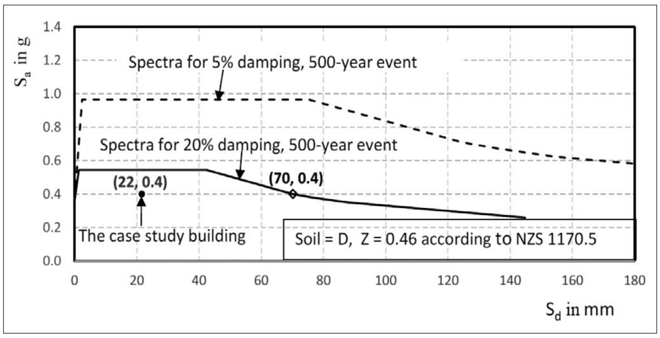

Figure 11: Relationship between spectral acceleration and spectral displacement [2].

The design guidance for LTF bracing systems, suggested by BRANZ [4], is based on the extension of the above research. It was reported that many new LTF houses use specifically designed bracing elements that are out of the scope of NZS 3604. The guidance highlighted that potential stiffness incompatibility between conventional LTF bracing walls and specifically designed bracing elements could lead to significant earthquake damage to LTF houses. A step-by-step seismic design procedure for specifically designed bracing elements was suggested, in which the storey drift limit at the ultimate limit state was set at 1%. Several methods for enhancing the racking performance of plasterboard walls were suggested including improving the connection details between timber frame members and improving the hold-down details at wall bases.

In terms of dynamic characteristics, there are no specific recommendations for timber-frame structures in the design standards. When applying time history analyses, NZS 1170.5 requires 5% viscous damping for all modes whose period is less than the incremental time step included in the analysis. If Rayleigh damping is used, there shall be no more than 5% of critical damping in the two first translational modes.

Experiment Studies on Plasterboard Bracing Walls and LTF Residential Structures

Bracing Wall Elements

Wolfe [35] tested 30 plasterboard sheathed walls under monotonic loads to determine the plasterboard's contribution to the wall racking resistance. Some walls had diagonal wood braces or metal strap braces while others did not, and the wall-length range was 8, 16, and 24 feet (2.44, 4.88, and 7.32 m). The typical test setup is shown in Figure 12. For plasterboard sheathed walls without braces, one had nail failure initially in the tension corners and the other two exhibited nail failure distributed along the top or bottom plates rather than concentrated at the corners. The results showed the total bracing capacity of the wall with a diagonal brace and sheathed with plasterboards was equal to the sum of these elements' resistance when tested independently. The ultimate shear strength of the tested walls showed an approximately linear relationship with the wall length, but the wall initial stiffness showed a nonlinear relationship with wall length (approximately a power function). It was also found the walls with plasterboard oriented horizontally showed over 40% higher strength and stiffness than those with vertically oriented plasterboards. Thurston [36] explained that the plasterboards in this study had an unconfined edge at the ends which could crack more easily when nails are used. This weakness was mitigated by taping and Gib-stopping when plasterboards were sheathed horizontally. It was mentioned that Aotearoa NZ plasterboards have the same unprotected edges when oriented vertically, and some strength gain with horizontal construction was also observed in some unpublished BRANZ tests.

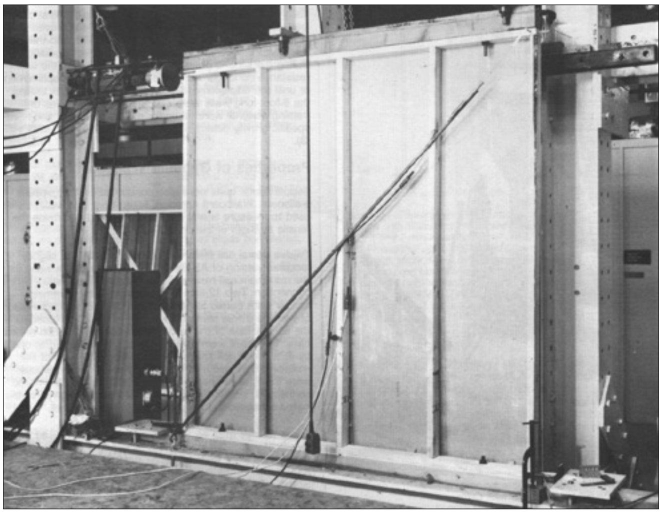

Figure 12: Wall test setup [35].

The openings in walls can also significantly affect the performance of plasterboard bracing walls. Dishongh and Fowler [37] compared the performance of plasterboard sheathed (both sides) walls with and without openings. Eight tests were conducted including three continuous diaphragm walls, three walls with door openings, and two walls with window openings. It was concluded that a wall with a central window opening could be treated as two separate full height bracing walls.

Thurston [36] conducted P21 tests on 10 long plasterboard walls with openings under pseudo-static reverse-cyclic loads. It was found that the performance of walls with large window openings or door openings (with hold-down straps on the edges bounding the door) could be obtained by adding the performance of the two separate walls between the openings. But, for a wall with a door opening in which straps were not used, adding the two segments' performance would overestimate the wall performance. The racking strength appeared to be very close between the walls where plasterboards joined at the window openings (as shown in Figure 13(a), where the nearest joint is 300mm or more away from the vertical opening edge) and those where plasterboards were cut at the opening (as shown in Figure 13(b), sheathing sheet edges coincided with the window or door trimmer studs). The racking deformations were analysed and scrutinised. The conclusion was the two most dominant components were the rocking of the entire panel and sheet rotation relative to the frame due to fastener slips. The former one contributed 60% to 100% of the total deformation and the second one contributed 10-50%.

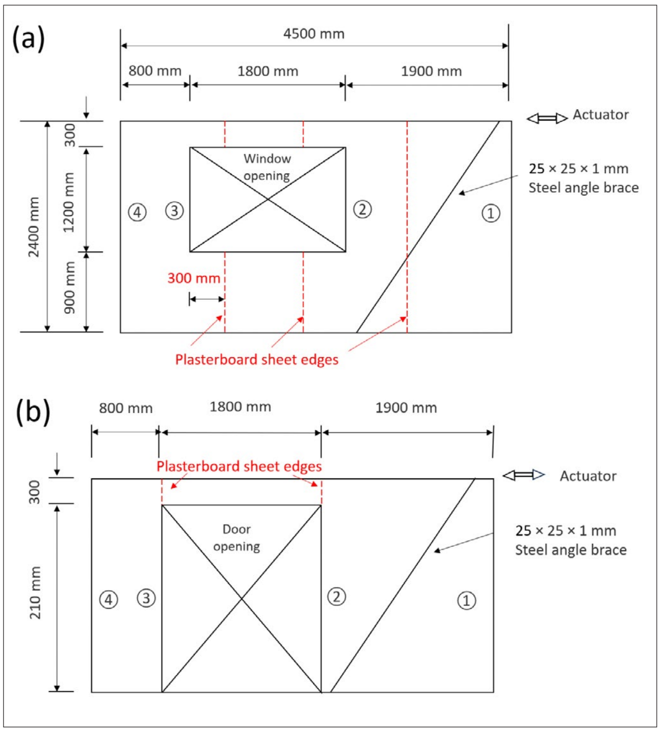

Figure 13: The wall case where plasterboards joined at the window openings [36].

Liu and Carradine [3] analysed P21 test results of 12 plasterboard walls in terms of stiffness/strength degradation, displacement capacity, superposition applicability, and failure mechanisms. It was found that the plasterboard walls showed significant strength degradations under cyclic loading. For the same displacement level, the racking strength in the third cycle was 15% to 25% lower than that in the first cycle. The maximum equivalent damping ratio of these walls was around 15%, which means the plasterboard bracing walls had limited energy-dissipating capacity.

Experimental studies were also conducted overseas to evaluate the performance of plasterboard sheathed walls. Although different design standards and product standards were involved, these studies provided insightful knowledge to understand the performance of plasterboard bracing walls typical of Aotearoa NZ construction methods. Chen et al. [1] conducted tests on 12 shear walls sheathed with OSB alone, Type X plasterboard only, and a combination of OSB and plasterboard under monotonic and reversed cyclic lateral loads. The specimens followed construction practice in Canada, where Type X plasterboard (gypsum wallboard, GWB) is commonly used for fire rated wood-frame walls and can be used for shear wall applications [38]. The test setup is shown in Figure 14. It is noted that the end studs and the bottom plates of the test specimens were firmly bolted down to the rigid foundation beams. The test programme

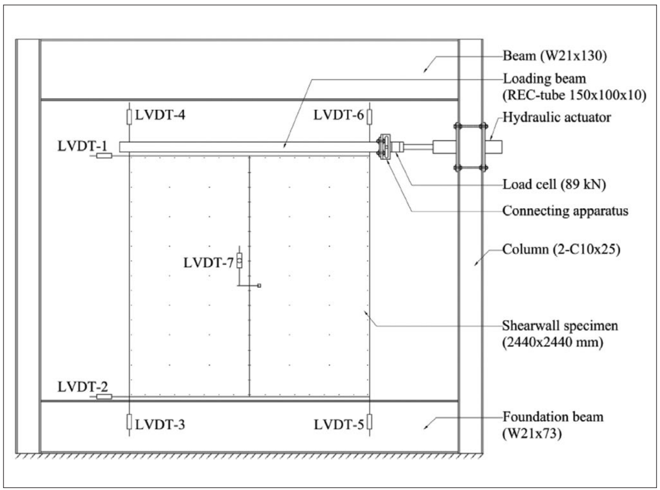

Figure 14: The wall test setup [1].

Figure 15: Typical plasterboard sheathed shear wall (overseas cases) failure modes: (a) fastener bending; (b) panel edge tear out; (c) plasterboard fissure failure; (d) end nails yielding and withdrawal; (e) hold-down deflection; (f) wood crushed by anchor bolt [15].

concluded that the racking resistance of shear walls sheathed with OSB one side and plasterboard on the opposite side can be estimated by summing those of shear walls with OSB or plasterboard alone (the direct superposition rule). Using joint tapings and two sides of plasterboard could increase the strength and decrease the ductility ratio, and the walls with the panels placed vertically provided higher strength and energy dissipation than the walls with the panels placed horizontally.

Lafontaine et al. [15] tested eight full-scale Type-X plasterboard sheathed shear walls under reversed cyclic loading to investigate the effect of fastening parameters. Typical failure modes included fastener bending, panel edge tear out, plasterboard fissure failure, end nails yielding and withdrawal, hold-down deflection, and wood crushed by anchor bolts (as shown in Figure 15). Common parameter variations in LTF plasterboard shear wall constructions that affect the response were found to be fastener type, panel orientation, shear wall length, joint compound type, and loading type.

Building Systems

Several field tests have been conducted on LTF residential houses. A single-storey LTF house was tested by BRANZ [39]. The test house was a standard Fletcher Homes house, typical of those at the low-cost end of the market available around 1990, having plasterboard linings and fibre-cement weatherboard claddings. The bracing walls were 2.4 m high, sheathed by 10 mm thick plasterboards, and had no hold-downs. The house plan and wall cross-section details are shown in Figure 16(a). Free vibration tests and cyclic racking tests were conducted. The test setup is shown in Figure 16(b). The load was applied using two hydraulic jacks to the ceiling plane at four locations along the house length. Four timber load beams located in the ceiling cavity were used to spread the applied force along the adjacent house walls. The two hydraulic jacks were fixed to separate reaction frames which were bolted to an existing concrete pad. According to the free vibration test results, this house had a natural frequency of 20.8Hz (fundamental period of 0.05s) and an average critical damping of 8.2%. The test results showed that the averaged cyclic strength of the whole house was 50% greater than that predicted based on the sum of all walls' strengths derived from P21 test results.

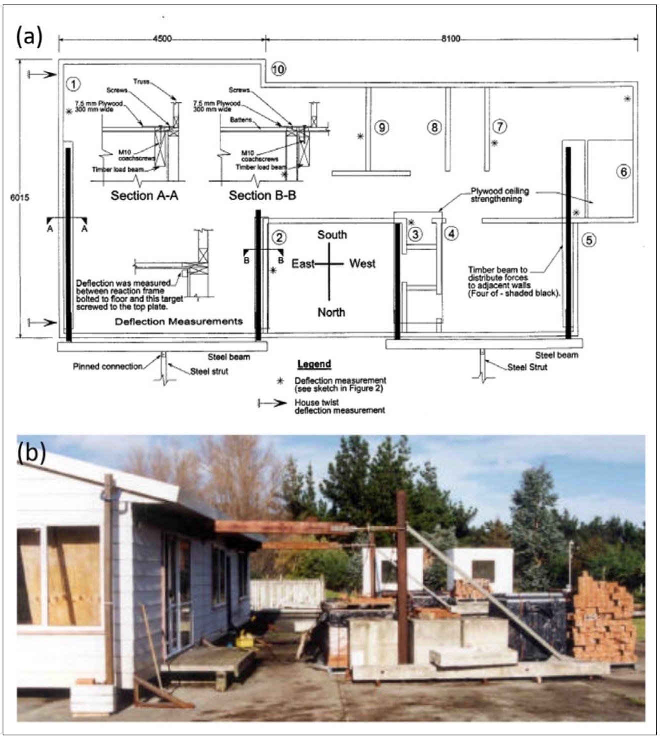

Figure 16: Test house [39]: (a) house plan; (b) general view of test in progress.

Morris et al. [40] summarised an in-situ test programme on post-quake houses after the 2010-11 Waitaha Canterbury earthquake sequence. This programme conducted quasi-static cyclic tests and snap-back tests on five houses. Two houses were built before the 1970s. One house, built in 1923, had walls lined with plaster-on-lath, and the other, built in 1947, had walls lined with fibrous plaster and light timber panelling. The remaining three houses, constructed after 1970, were braced with plasterboard sheathed walls. The measured structural properties of the houses, including lateral strength, stiffness, fundamental period, and damping ratio are listed in Table 1. It was found that the periods of two newer houses (built in the 1980s and 1990s) were all 0.14 s.

A full-scale one-storey simple building with long plasterboard bracing walls was tested at BRANZ [10] by applying cyclic loading to determine the bracing performance of long plasterboard-lined walls. Along the loading direction, the building had 2.4 m and 3.6 m long plasterboard walls, as shown in Figure 17. The test results showed the wall bracing strength of the test building degraded more slowly than that of the isolated long walls in the P21 test. The walls were more ductile than the isolated walls and the strength was around twice that of combined isolated walls with the same total length. This matches the findings by Thurston [9], that is, LTF houses appear to have higher capacity than the simple sum of individual bracing walls. Furthermore, it was found that the test building had a systems overstrength factor of approximately 2.0 and the author concluded that this was mainly attributable to the plasterboard tapes between the orthogonal walls. The value of this overstrength factor was the same as that found in [39].

Table 1: Information and structural features of test houses [40].

Address	Built year	Wall lining	Stiffness	Period	Damping
Retreat Road	1923	Plaster on lath	3.8 kN/mm	0.29 s	12% (snapback linear)
Bexley Road	1947	Fibrous plaster and light timber panelling	9.0 kN/mm	0.23 s	
Cardrona Street	1970+	Plasterboard	7.5-8 kN/mm	0.20 s	>6%
Wairoa Street	1983	Plasterboard	18 kN/mm	0.14 s	
Norcross Street	1993	Plasterboard	27 kN/mm	0.14 s	6% (by a hammer blow)

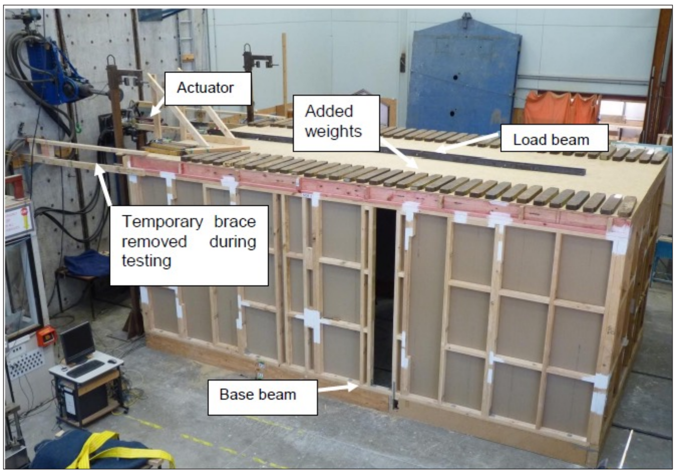

Figure 17: Test building and test setup [10].

BRANZ tested another single-room one-storey building to determine stiffness degradation of LTF houses after earthquake shaking and the effectiveness of different repair methods [41]. As shown in Figure 18(a), the building was nominally 2.4 m high and incorporated windows and doors and two short internal walls. It had plasterboard bracing walls and a plasterboard-lined ceiling. Most of the plasterboard bracing walls were sheathed by standard plasterboards and did not have hold-downs except one wall labelled “BP10” sheathed by bracing plasterboards and with hold-downs. The test setup is shown in Figure 18(b). The structure was first loaded with three cycles of displacement amplitude of 1.65mm, 3.92mm and 7.29mm. Then it was repaired using one method and was retested. Following another repair method, the structure was tested again. The repair methods used in the different phases are listed in Table 2. The comparison between backbone curves from different test phases is illustrated in Figure 19. The results showed that the cosmetic repair was moderately effective at reinstating the initial building stiffness, and adding additional screws showed little improvement compared to the cosmetic repair only. The most effective repair method was fully overlaying plasterboard sheets and adding hold-downs to the ends of the bracing walls. It should be noted that the repair methods suggested in this study were based on the racking tests in which the walls only reached early plastic phases instead of complete failures.

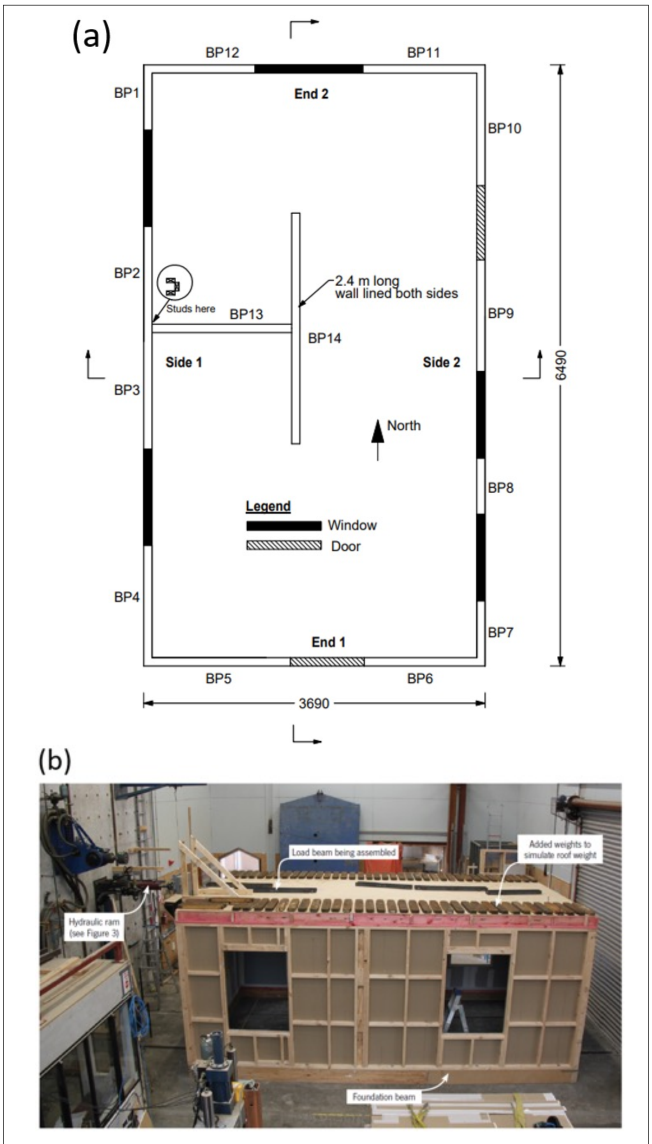

Figure 18: Test building [41]: (a) plan view; (b) test setup.

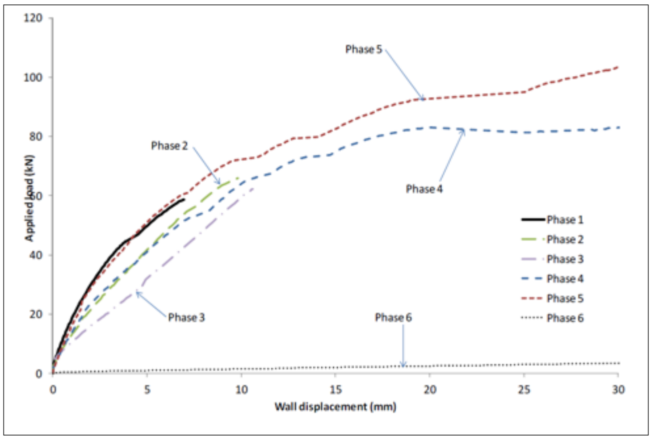

Figure 19: Comparison of backbone curves from each test phase [41].

Table 2: Construction used in the various test phases [41].

Test Phases	Building Condition
1	As-built
2	Cosmetic repair
3	All plasterboard-to-timber framing glued joints broken (after test 2, hammered a wooden block placed over the plasterboard inward from the outside of the building at all glue joint locations)
4	Cosmetic repair plus strengthening by adding drywall screws between all adjacent existing screws
5	A complete overlay of plasterboard added. Wall hold-down anchors added at ends of bracing elements.
6	All lining on Side 1 and 2 walls removed and replaced. Internal walls removed.

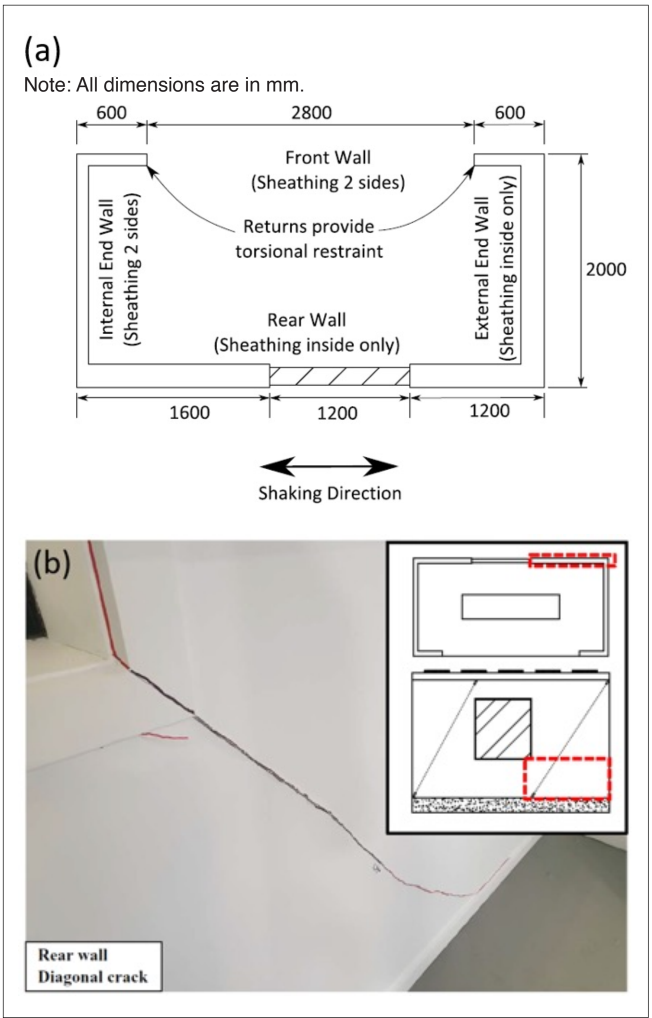

Figure 20: Test structure [42]: (a) floor plan; (b) diagonal shear cracking around window opening.

A full-scale LTF building with plasterboard sheathed walls was designed and tested on the University of Waitaha Canterbury shake table by Francis et al. [42]. The test structure represented a corner room of a common one-storey house conforming to NZS 3604. The floor plan is shown in Figure 20(a). The rear wall and right wall represented external walls, sheathed by 10 mm plasterboards as internal linings. The front wall and the left wall represented internal walls, sheathed by 10 mm plasterboards on both sides. Hold-downs were used at each end of front and rear walls. Nominal screw spacing for non-bracing walls was used for all walls. The authors explained that if the common screw spacing for bracing walls was followed, the fundamental period would be too short, or an unrealistic quantity of weight would need to be added to the roof to alter the structural period. The total seismic weight of this structure was 11.35 kN after adding an additional 920 kg of mass to the roof. The test structure was excited in the longitudinal direction only by three Aotearoa NZ earthquake records. The fundamental period of the test structure was found to be 0.1 s. During low-intensity tests, only minor damage was observed with small cracks initiating from the large opening's corner on the front wall. During the full intensity run of the Darfield earthquake record, the peak inter-storey drift ratio reached 0.23% and more structural damage was observed, including cracks around the corners of the front wall and diagonal shear cracks around the window opening (as shown in Figure 20(b)). Meanwhile, the hold-downs and sill plates were found to be undamaged.

A further shake table test was conducted by Francis [43] for a base-isolated LTF building. The super structure was the same as the test structure in [42]. A low-cost base isolation system was proposed which used custom built bi-directional friction slider devices with pucks made of Polytetrafluoroethylene (PTFE) sliding against a grade 8 mirror finish stainless steel surface embedded in a concrete slab. Four base isolation devices were arranged at the four corners of the test structure on the shake table. The shake table test results showed that the isolation system can provide excellent protection to the superstructure and contents resulting in no observable damage throughout 31 tests under full-intensity ground motions.

Numerical Modelling Approach of Wood-Framed Shear Walls

Since the bracing walls are the most important components of LTF structures to resist seismic loads, accurate wall modelling is an essential part of their seismic simulation. The racking model of timber shear walls can be broadly classified into two main categories: analytical models, and numerical models. Analytical models are mathematical models that have a closed-form solution. This kind of model of timber shear walls is adopted by many design standards to build up the relationship between the bracing capacity and deformation. According to the Engineering Basis of NZS 3604 [44], the lower bound of elastic modulus from NZS 3603 [45] is used for calculating the deflections and limiting plate loads of bracing walls. This originated from the plastic lower bound model, a classic analytical wall model developed by Neal [46]. Numerical methods with hysteresis wall models are more suitable for full-structure nonlinear dynamic analysis to simulate seismic responses. There are two types of numerical models for LTF structures: detailed finite element models and macro element models.

Detailed Finite Element Models

The detailed models consider almost all structural components by modelling most parts of the timber shear walls, normally including beam elements for frame members, shell/plane elements for sheathing panels, and spring elements for connections. The spring models for connections are the key parts of the detailed model. Specific hysteretic models of these springs are required for nonlinear dynamic analysis, accounting for hysteretic damping and strength/ stiffness degradation. There are various types of hysteretic models for timber connection: mechanics-based models, empirical models, and mathematical models.

Mechanics-based models rely on the basic material properties of the fastener and the embedment characteristics to model the connection's hysteresis [47, 48]. Empirical models fit the connection's hysteresis using a combination of linear segments or curves for loading and unloading paths obtained from testing. Several empirical models for timber connections have been proposed, including the bilinear model [49], the trilinear model [50], and the damage-considered model by Wen [51]. Mathematical models do not directly rely on mechanical properties but on physical understanding of the hysteretic system. The Bouc-Wen model [52] is one widely used mathematical model for the hysteretic behaviour in civil and mechanical engineering, and Foliente [53] modified this model to characterise the general features of the hysteretic behaviour of wood connections and structural systems, which is known as the Bouc-Wen-Baber-Noori (BWBN) model.

The following four wood shear wall models represent different detailed modelling techniques. A finite element computer programme named WANELS was developed by Gutkowski and Castillo [54–56] for the analysis of single- and double-sheathed wood shear walls under static loads. In this programme, the sheathing panels are modelled by two-dimensional orthotropic-plane stress elements. The nailed connections between the frame and sheathing are modelled by nonlinear nondimensional spring elements, and the joints between the frame members are modelled by linear spring elements. This programme can also examine the nail forces and their distribution, and failed nails can be removed automatically to trace progressive failure. For lateral nail resistance, this programme uses a solution combining the advantages of the variable stiffness method and the load correction method. The related parameters are determined by a stepwise approximation to either nonlinear lateral nail resistance data or a chosen empirical relationship. This model can predict the load-displacement relationship of target walls with a high degree of accuracy well into the nonlinear range. Figure 21 shows the comparison between the simulation results of the programme WANELS and ten experimental results on a wall sheathed by plasterboards on one side only.

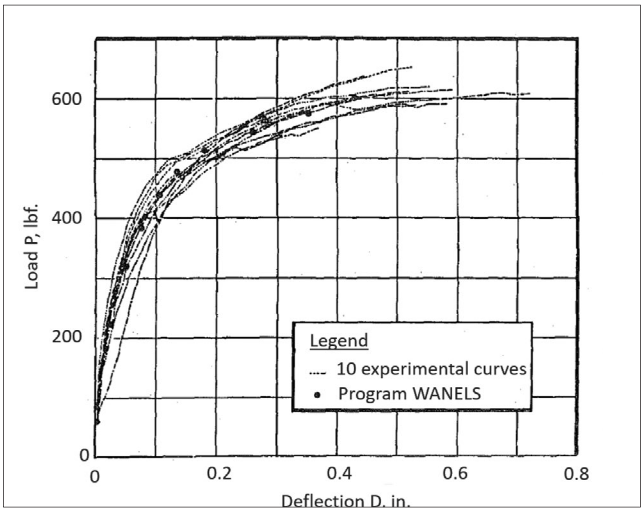

Figure 21: Comparison between the simulation results of programme WANELS and ten experimental results on a wall sheathed by plasterboards on one side only [56].

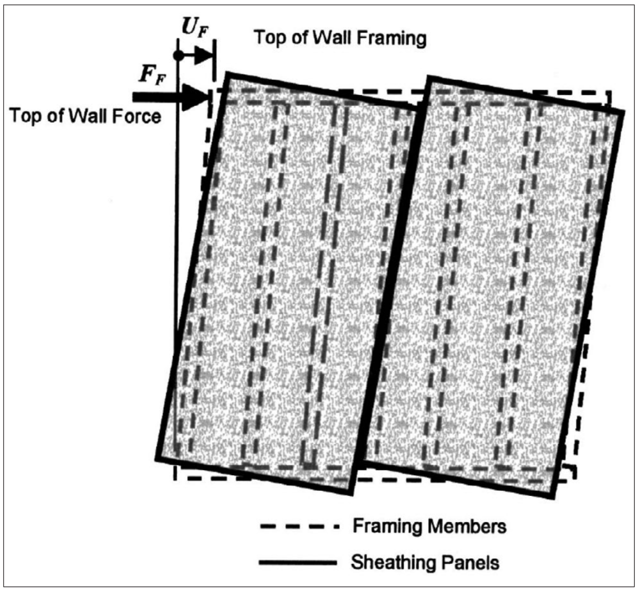

Figure 22: Assumption of racking mode for wood-framed shear walls [27].

A simpler numerical model was developed by Filiatrault and Folz [27, 57] to predict the response of timber shear walls under cyclic loads. The model was composed of three types of structural components: rigid framing members, linear elastic sheathing panels, and nonlinear sheathing-to-framing connections.

The racking mode of wood-framed shear walls was assumed to be as shown in Figure 22. The framing members were assumed to be rigid with pinned connections, so the wall frame alone has no lateral stiffness. The out-of-plane deformation of sheathing panels was ignored as it is a two-dimensional wall model. Each rectangular sheathing panel developed a uniform in-plane shear deformation, superimposed on horizontal and vertical rigid-body translations and rotations. The relative displacements between sheathing and framing resulted in inelastic deformations at the sheathing-to-framing connections. Previous studies support these assumptions [58, 59]. An empirical hysteretic model for sheathing-to-framing connections, originally proposed by Foschi [60], was modified to minimise the path-dependent rules. The force-displacement response of connections under monotonic and cyclic loading is shown in Figure 23. The wall model was verified against tests of wood-framed shear walls and has been incorporated into the computer programme CASHEW (Cyclic Analysis of Shear Walls).

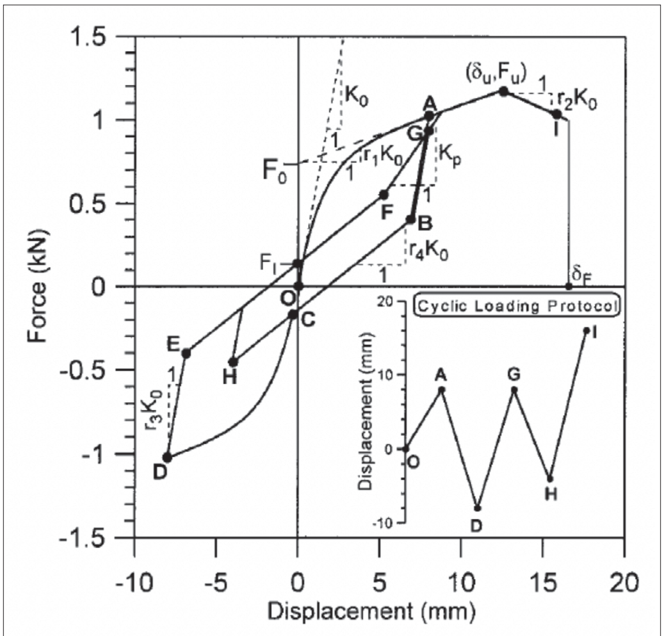

Figure 23: Force-displacement model of sheathing-to-framing connections under monotonic and cyclic loading [57].

Pang and Hassanzadeh [61] claimed that the simplified assumptions in the CASHEW programme limit its applications to modelling engineered and fully anchored shear walls only. A new detailed model using the nodal condensation technique was developed [61] and coded into a computer programme named M-CASHEW. As shown in Figure 24, this model employs three types of connection elements to model the partial composite action between the frame and the panel, including panel-to-frame, frame-to-frame, and panel-to-panel connections. Each connection is represented by a 2-node, three degrees of freedom (3-DOF) connection element with three orthogonal uncoupled springs. Each of the orthogonal springs can be assigned the properties of one of the seven elastic and hysteretic spring models available in M-CASHEW. By combining the M-CASHEW wall and diaphragm element models, a global 3D platform named Timber3D was developed by Pang et al. [62] for nonlinear time history analyses of light-frame wood buildings. In Timber3D, the frame elements are modelled individually as 12-DOF elastic elements with pinned ends. Horizontal loads are resisted by shear springs that span between floors and represent the behaviour of the wall elements.

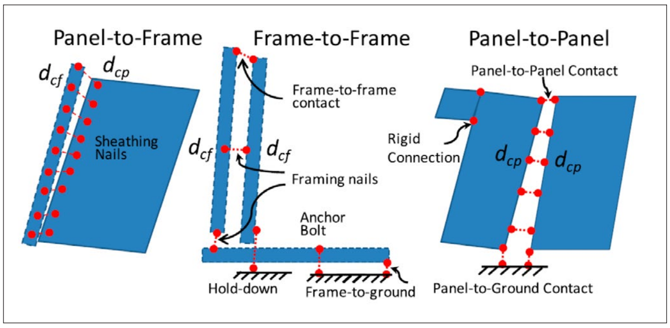

Figure 24: Connection elements of the detailed model [61].

Christovasilis and Filiatrault [63] pointed out that most of the previous wall models did not consider the rocking and uplifting deformation among frame members and connections to the diaphragms, which could reduce the wall strength and stiffness. They developed a shear wall sub-structure model to predict the lateral stiffness, strength, and energy dissipation capacity. Their model is configured to require a smaller number of DOFs to make the wall model more efficient for the global analysis of complete buildings. In this model, the sheathing panels are described with 4 DOFs and sheathing-to-framing connections are described with two orthogonal independent phenomenological springs. Similarly, the frame members are represented with 2-noded elastic beam elements. Figure 25 illustrates the numerical model of the framing domain. As can be seen in this figure, contact springs are introduced to model the framing-to-framing and framing-to-floor connections, and the horizontal forces between the top plates and diaphragms are transferred through the master nodes (normally at the centre of the top plate). It enables modelling of the uplifting response without consideration of geometric nonlinearity.

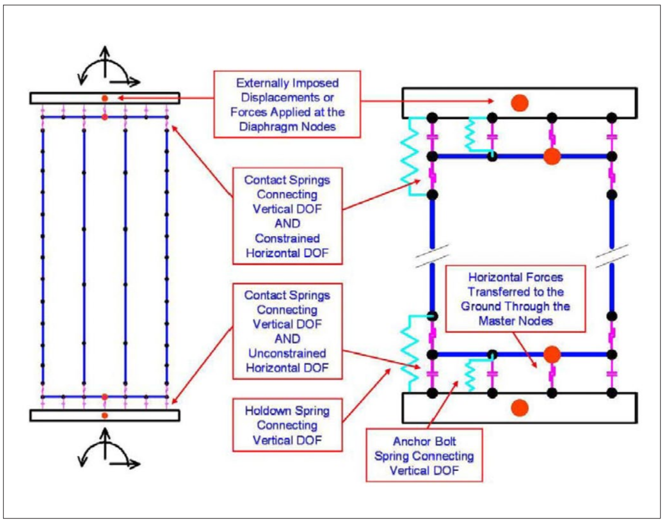

Figure 25: Detailed numerical model of the framing domain [63].

Macro Element Models

Although the finite elements-based numerical models can capture the detailed behaviour of framing members, nails and sheathing panels, they are often not computationally efficient to simulate the entire building. For the whole building simulations, the focus is on the overall wall performance or storey drift rather than the specific responses of the individual components. For this purpose, macro element models have been developed to simplify the wall models in whole-building analyses.

A typical macro element model consists of three rigid truss elements (acting as a frame) and one or two nonlinear springs, where the springs represent the nonlinear behaviour of sheathing-to-framing connections, which mostly govern the nonlinear wall behaviour. Figure 26 shows two examples of the macro element model [64]. The wood-frame shear wall is simplified to single horizontal shear-springs (a) or diagonal-spring elements (b). Chen et al. [65] developed a modified macro element model that also accounts for the wall rotations. As shown in Figure 27, in this model two vertical springs are added to the bottom of the rigid truss element and pinned to the ground.

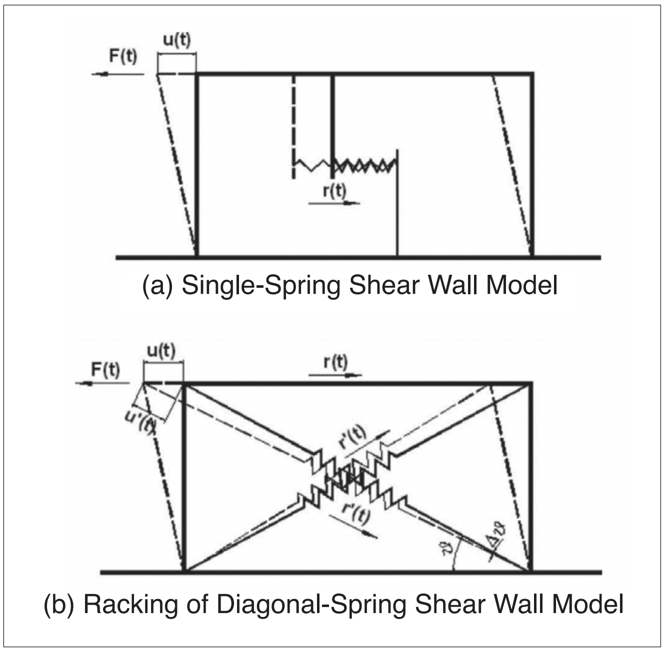

Figure 26: Examples of the macro element model [64].

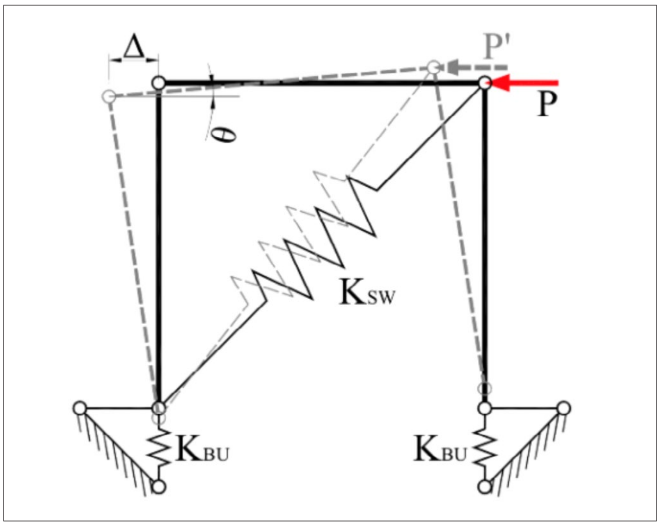

Figure 27: A modified macro element model [65].

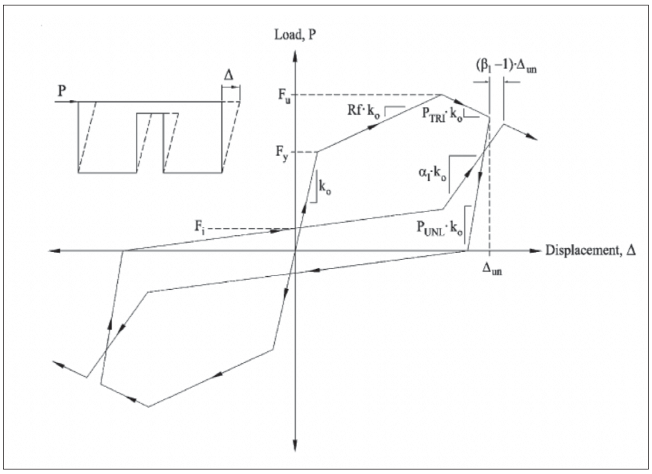

Figure 28: Stewart's hysteresis model [50].

The definition of the hysteretic rule of these springs is the key to the reliability of macro element models in predicting nonlinear dynamic response. The three types of hysteretic models introduced above (mechanics-based models, empirical models, and mathematical models) are also applied to the macro springs. The hysteretic parameters of the springs can be obtained from the detailed shear wall models or shear wall test data. As an example, Filiatrault and Folz [27] proposed a three-dimensional nonlinear pancake model for LTF buildings. In this system, each wall is modelled by a single zero-height nonlinear in-plane shear spring using Stewart's empirical hysteresis model [50], as shown in Figure 28.

A macro spring model for shear walls has also been derived by modifying the individual nail connection model [64, 66]. This approach was based on the fact that the global hysteretic behaviour of LTF shear walls is similar to that of nail-to-wood connections, including strength/stiffness degradation and pinching effect. The "pseudo-nail" model is a typical macro wall model developed by Li et al. [66], revised from a nail connection model named HYST [47]. HYST is a common panel-frame nail connection model used in wood shear walls. Figure 29(a) illustrates the schematics of HYST. A modified HYST algorithm, developed by Li et al. [48], improved the computational efficiency and addressed the stiffness degradation effect. Figure 29(b) shows the loading and unloading of wood medium in the modified HYST algorithm. The parameters in this model include the nail length L, nail diameter D, and six parameters to describe the compressive properties of the surrounding embedment medium. These parameters can be calibrated by shear wall test data or detailed wall models. The "pseudo-nail" wall model was incorporated into a computer-based structural analysis tool called "PB3D" developed by Li et al. [66]. "PB3D" is an efficient three-dimensional analysis platform for nonlinear time history analysis of residential post and beam timber buildings under seismic loads. In this platform, the diaphragms are modelled by beam elements and diagonal truss elements considering the in-plane stiffness, and beams and posts are modelled by elastic beam elements. The uplifting is simply prevented by wall post elements which are fully end-restrained onto the foundation or stories. Figure 30 shows the schematics of a PB3D model.

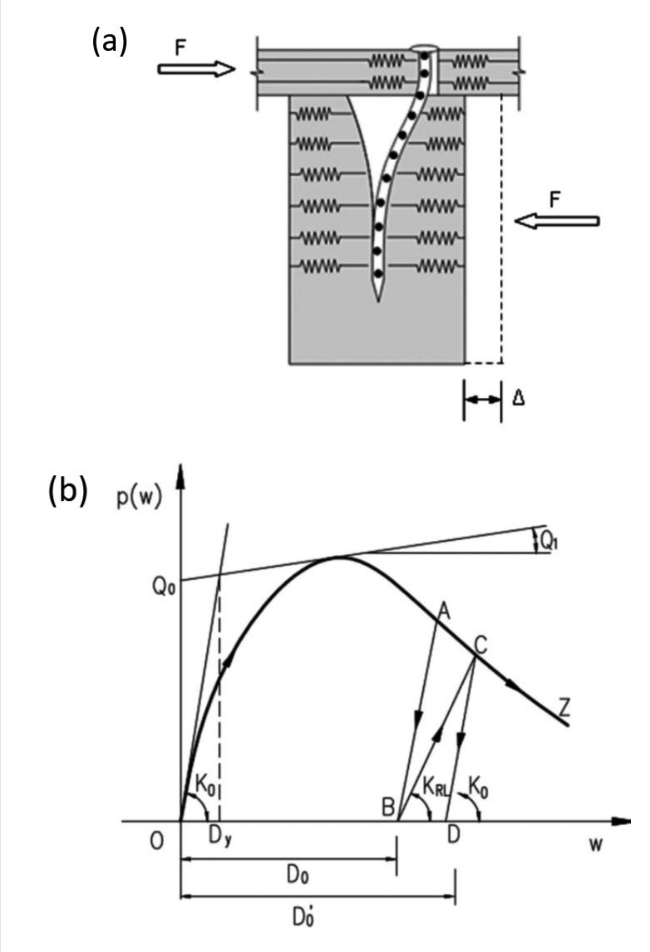

Figure 29: HYST algorithm: (a) schematics of HYST panel-frame nailed connection; (b) loading and unloading of wood medium in modified HYST algorithm [48].

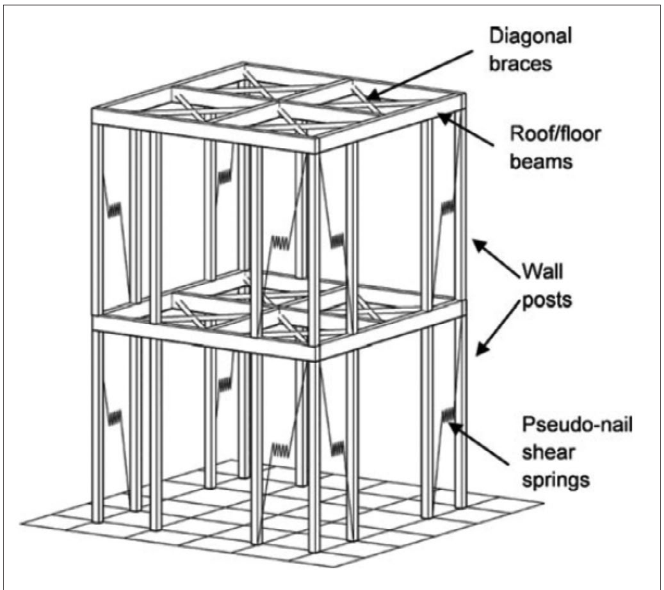

Figure 30: Schematics of a "PB3D" model [66].

Simulation Studies on Seismic Performance of LTF Residential Structures

A closed-form racking model was developed by Liu and Carradine [3] for Aotearoa NZ plasterboard walls based on the racking test results of 12 plasterboard walls. It was assumed that the total lateral deformation of the wall is the sum of the flexural deformation and the equivalent shear deformation that accounts for all other sources of deformations including sheathing panel shear, connection slip, and hold-down uplift. The expressions are as follows:

$$\Delta_{total} = \Delta_{flexural} + \Delta_{uplift} \tag{3}$$

$$\Delta_{flexural} = \frac{2VH^3}{3EA_C L^2} \tag{4}$$

$$\Delta_{uplift} = \frac{VH}{G_e L t_e} \tag{5}$$

where V is the racking load at the top of the wall, H is the height of the wall, E is the modulus of elasticity of timber chords, A_c is the area of chords, L is the length of the wall, t_e is the total thickness of plasterboard sheathing, and G_e is the equivalent shear modulus. The key parameter was the equivalent shear modulus at different deflection levels, and it was determined based on P21 wall test results. This racking model can reasonably capture the skeleton curve of the plasterboard walls. However, the model was calibrated based on limited wall configurations of the test specimens and did not consider the effect of strength degradation.

Based on the in-situ tests on post-quake houses after the 2010-11 Waitaha Canterbury earthquake sequence, a single-degree-of-freedom nonlinear model was developed by Morris et al. [40] for the 1923 tested house as a case study. The load-deflection hysteresis of this numerical model was calculated by the HYST model and calibrated based on the push and pull test results. (Figure 31(a) illustrates the hysteresis curves of test results and the calibrated HYST model). After that, time history analyses were conducted using 19 ground motions from the 2011 Christchurch earthquake. The original records were scaled to two seismic design levels (500- and 2500-year return periods). Figure 31(b) illustrates the cumulative distribution of peak displacement responses. It showed the average peak displacement at the 500-year return period design level was only 9 mm (0.4% drift ratio), which meant this post-quake structure would still perform well at this seismic level.

A series of building models were developed in 3D ETABS by Liu and Shelton [67] to evaluate seismic effects of permissible irregular distribution of bracing resistance within the scope of NZS 3604. Six case-study single-storey LTF houses braced by plasterboard bracing walls were designed according to NZS 3604 and modelled. The first three houses had the same identical rectangular floor plan but different irregularity levels (0, 50%, and 100% respectively). The structural irregularity was caused by irregular arrangements of bracing walls. In the first building (0% irregularity), the bracing arrangements were perfectly regular. “100% irregularity” meant that the bracing arrangements were very irregular and reached the specified limits in NZS 3604. Similarly, “50% irregularity” meant the bracing arrangements were between “100% irregularity” and perfectly regular, where the minimum bracing capacity of each bracing line is 75% of the total bracing demand divided by the number of bracing lines. The floor plans of rectangular case study houses are illustrated in Figure 32. The other three houses were L-shaped in plan. They also shared the same outline but different irregularities, 0%, 50% and 100% respectively. In the 3D ETABS models of these houses, plasterboard bracing walls and plasterboard ceiling diaphragms were modelled as shell elements. Then equivalent static push-over analyses were conducted in the Y direction. The results showed that the extremely irregular bracing arrangements as allowed by NZS 3604 could result in a significant increase in the maximum lateral deflections compared to the houses with regular bracing wall arrangements, approximately 5 times for rectangular cases and 3 times for L-shaped cases. Taking the rectangular case study houses as an example, the maximum drift of the regular arrangement case was 0.31% while that of the 100% irregularity case was 1.58%. Besides, it was found that the effect of irregularity on the fundamental periods (T_1) was not significant. T_1 of rectangular case study houses was about 0.2 to 0.3 s, and T_1 of L-shaped houses was about 0.45 to 0.55 s.

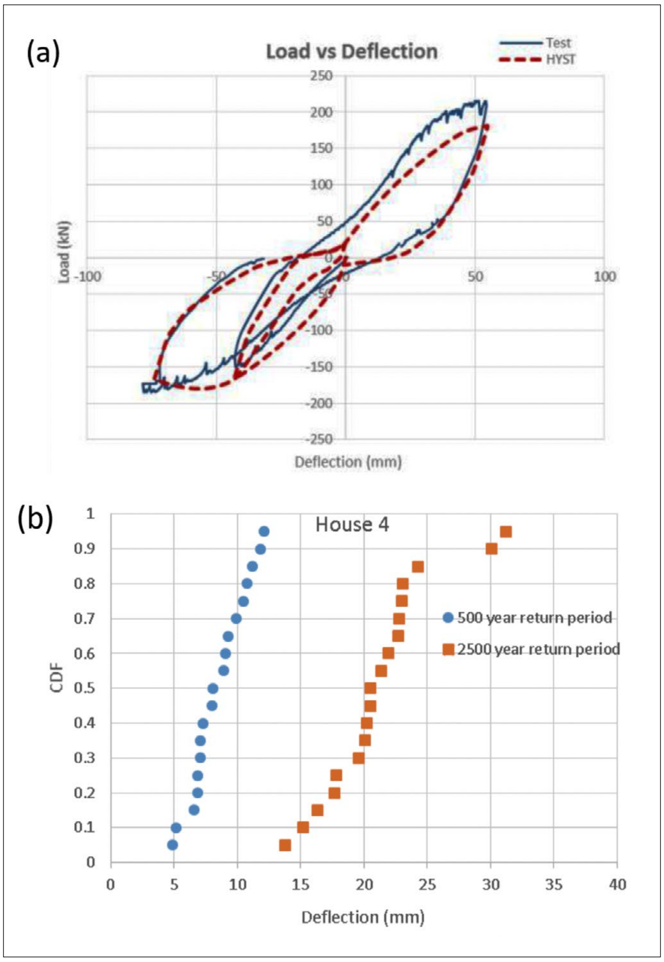

Figure 31: Single-degree of freedom nonlinear model based on HYST model for a case study building [40]: (a) hysteresis curves; and (b) cumulative distribution of peak displacement responses.

Ma et al. [68] conducted a parametric study for quantifying the effect of different levels of bracing wall irregularity as well as the rigidity of ceiling diaphragms on the seismic performance of LTF houses. Three groups of single-storey baseline houses with different bracing wall layouts were designed per NZS 3604. Within each group, three levels of bracing wall eccentricity were designed including a symmetric layout, 50%, and 100% of the specified irregularity limit (the same as the definitions in [67]). The floor plans of the baseline houses are shown in Figure 33. All bracing walls were sheathed with plasterboards on one side and had no hold-downs, and the diaphragm system was the GIB Rondo branded ceiling diaphragm. Rayleigh damping model was used, and the damping ratio was assumed to be 5% according to NZS 1170.5. Numerical modelling was conducted in the "PB3D" simulation platform, and the "pseudo-nail" model was used for the bracing walls as introduced in the last section. A suite of historical earthquake ground motions from the 2010–2011 Waitaha Canterbury

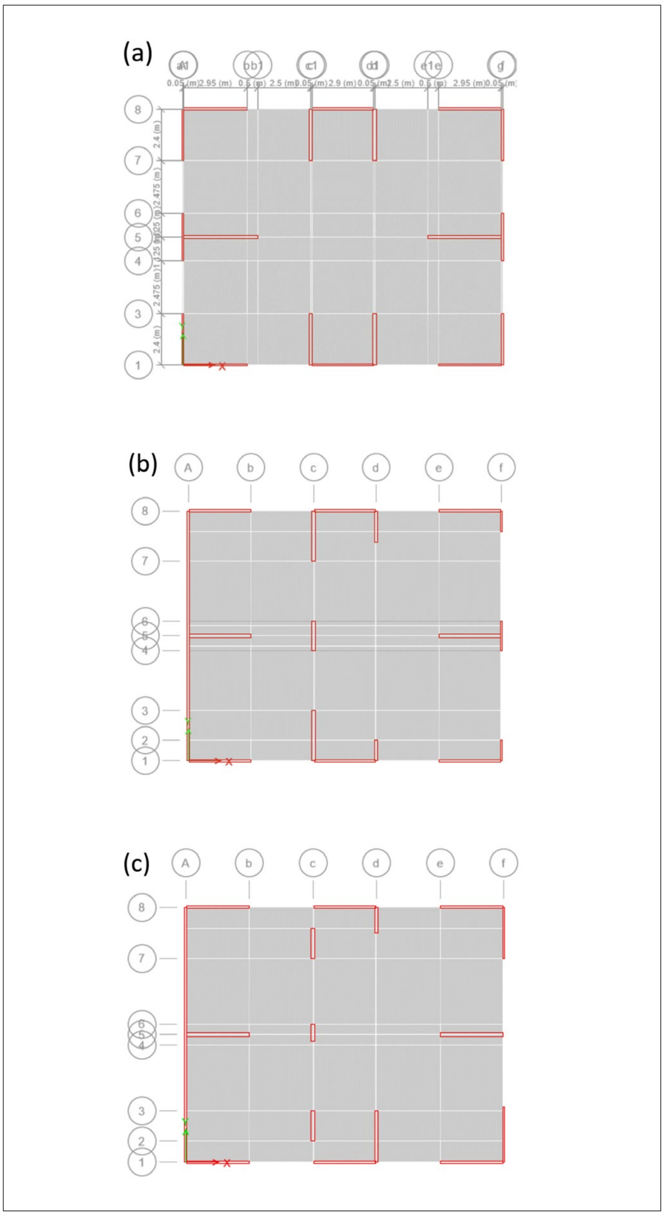

Figure 32: Floor plans of single-storey rectangular case study houses [67]: (a) regular bracing arrangement; (b) 50% irregularity in bracing arrangement; and (c) 100% irregularity.

earthquake sequence was used for time history analysis. These records were scaled to match their mean 5% damped spectral value over a period range of 0.1–0.56 s with the design spectra: 0.9 g spectral acceleration for the ULS level and 0.225 g spectral acceleration for the SLS level. The results showed that, in the baseline houses with rigid diaphragms and bracing walls of limit irregularity allowed in NZS 3604, the maximum drift response was about three times that in houses with symmetric bracing wall layouts. This means that the allowed irregularity of the bracing wall layout in NZS 3604 may cause significant torsional effects and excessive damage.

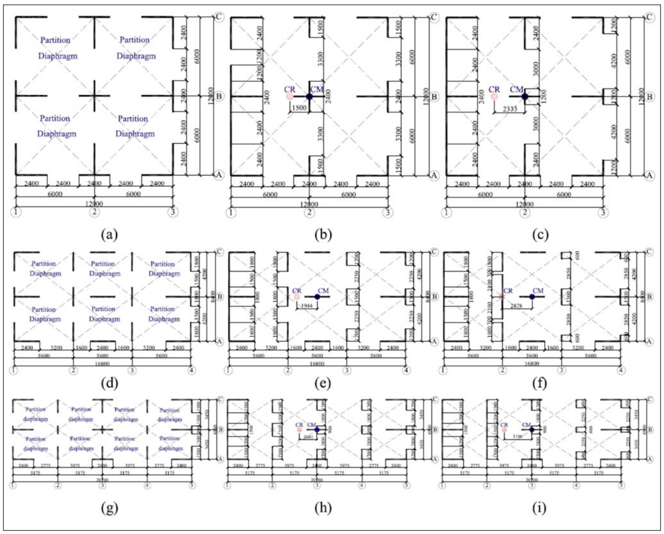

Figure 33: Three groups of single-storey baseline houses with different bracing wall layouts [68].

A numerical model was developed in Timber3D by Francis et al. [42, 43] for the LTF building introduced in the experimental studies section. Figure 34 illustrates the framing and wall elements of the Timber3D model. Based on this model, the modal analysis, pushover analysis, and time history analysis were performed. 2% and 5% Rayleigh damping were used in the time history analysis to compare the effect of different damping values. Figure 35 illustrates the displacement response of test results and models using 2% and 5% damping under three Aotearoa NZ earthquake records. It was concluded that the model using 5% damping provided a better prediction of the displacement response of the test building. This agrees with the damping approach used by Ma et al. [68].

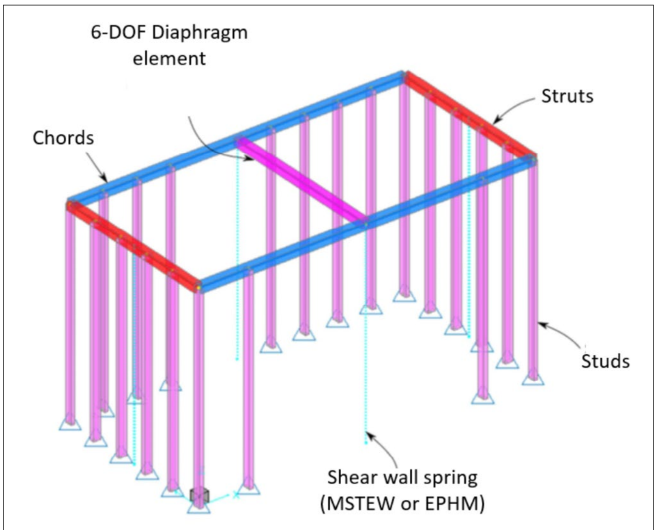

Figure 34: Timer3D model for test building [42].

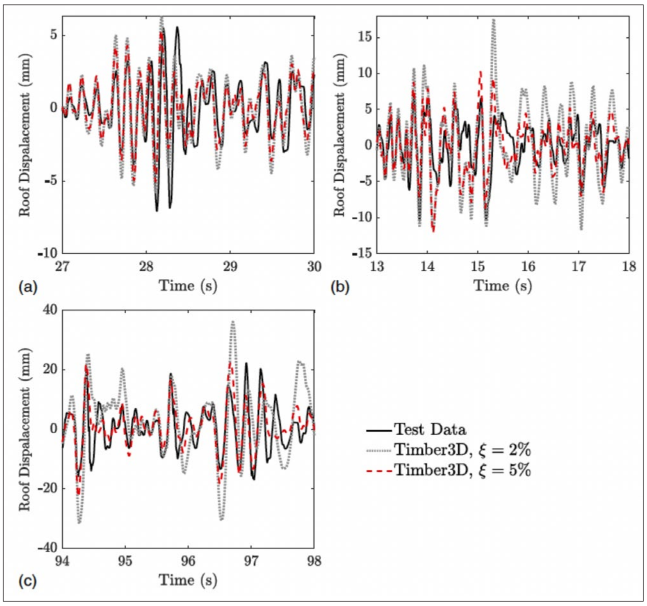

Figure 35: Displacement response of test results and models using 2% and 5% damping [42] under earthquake records of (a) Darfield 2010, (b) Lyttleton 2011, (c) Kaikōura 2016.

Seismic Damage and Loss Models of Aotearoa NZ Residential Houses

Seismic Damage Incurred by LTF Residential Houses

After the 7.1 Mw Darfield earthquake on 4 September 2010, Beattie et al. [8] conducted a post-earthquake damage survey on residential houses near Christchurch. Cracks on plasterboards in lower storey walls were observed in some houses, including diagonal cracks that emanated from the top corners of large door openings (Figure 36(a)) and vertical cracks on the joints at the corners of openings (Figure 36(b)). The survey also found that the “L” and “U” shaped houses suffered greater (no serious) damage at the intersection of the wings.

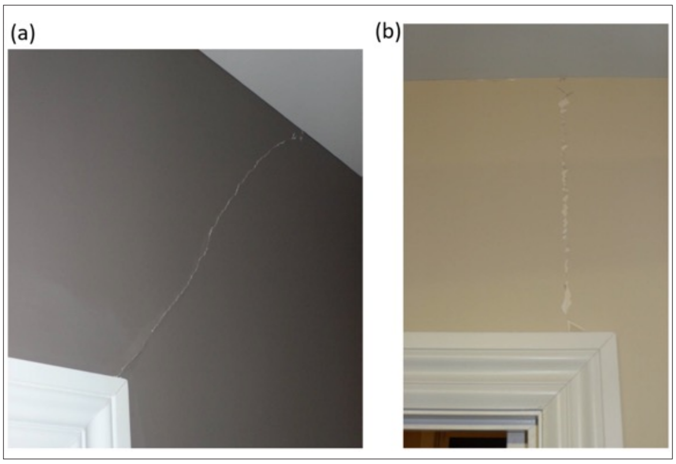

Figure 36: Plasterboard bracing walls damage after the 2010 Darfield earthquake: (a) diagonal cracking of plasterboard; (b) vertical cracking of plasterboard [8].

The 2011 Christchurch earthquake was a 6.3 Mw aftershock of the 2010 Darfield earthquake. Although it had a lower magnitude, the epicentre was closer (approximately 6 km southeast) to Christchurch City [69]. Many buildings in the CBD were severely damaged and some were demolished. Buchanan et al. [5] reviewed the performance of houses after this earthquake and found that LTF houses generally performed well to meet the life safety performance target. The envelopes and diaphragms of most timber houses successfully maintained structural integrity. Minor damage such as cracks in plasterboards was typically observed in the houses and some houses suffered more severe damage in bracing walls. Figure 37 illustrates a case where the plasterboard was completely detached from the wall frame.

Figure 37: A severe failure of the plasterboard internal linings after the 2011 Christchurch earthquake [5].

Seismic Loss Models

Researchers in Aotearoa NZ have also made efforts in the last century to develop building seismic loss models for Aotearoa NZ earthquakes. Damage ratios (i.e. ratios of damage repair cost to the building replacement cost) have been evaluated for different building types in several high-intensity earthquakes including the 1931 Hawke's Bay earthquake [70], the 1968 Inangahua earthquake [71], and the1987 Edgecumbe earthquake [72, 73]. These loss models can be categorised as empirical models, and the main basis of these studies was insurance claim data. Dowrick and his team [70–74] catalogued and categorised the damage to almost all building types in the higher Modified Mercalli intensity (MMI) zones and were therefore able to relate the distribution of damage ratio to the intensity level for many classes of building and their contents. The damage ratio, D_r, is used to express the degree of damage to any class of property at risk, and it is defined as:

$$D_r = \frac{Cost\ of\ Damage\ to\ Property}{Value\ of\ Property} \tag{6}$$

In this equation, the Value of Property is defined variously in the literature and could be the replacement value, market value, indemnity value, or insured value. The damage ratios are studied as functions of the intensity of ground motion and are related to the MMI isoseismals. The damage ratios for low-rise Aotearoa NZ buildings have been estimated by Dowrick et al. [71, 74] and they are modelled as:

$$\underline{D_r} = A \times 10^{\left(\frac{B}{MMI-C}\right)} \tag{7}$$

where $\underline{D_r}$ is the mean damage ratio, MMI is the shaking intensity, and A, B, and C are constants. The relationships between the intensity and damage ratios of four common building types are shown in Figure 38. These functions are based primarily on the Aotearoa NZ data for intensity zones MM5 to MM7 and a combination of Aotearoa NZ and United States data [75] for zones MM8 to MM10. When considering the uncertainty of damage ratios, the shape of the statistical distribution of non-zero damage ratios for various classes of property at each intensity level is found to be well approximated by a lognormal distribution [71].

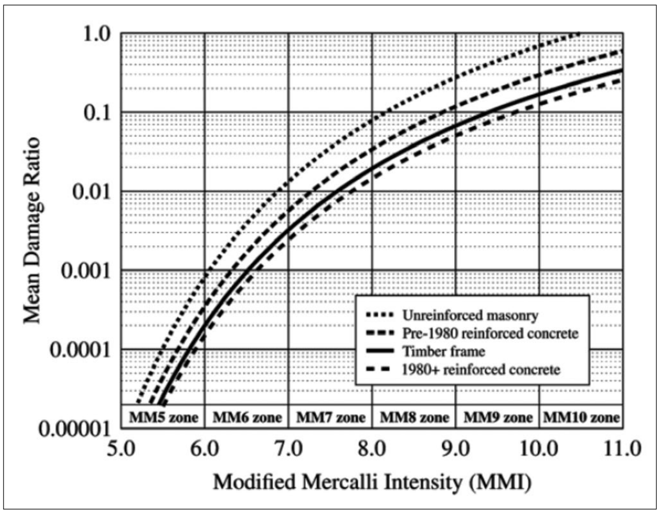

Figure 38: Mean damage ratios of low-rise Aotearoa NZ buildings in Dowrick's damage model [76].

Since Dowrick's loss models were developed based on damage data in the last century, these models did not include modern construction types and may not align with current economic conditions. Horspool et al. [7] updated the damage and loss model for residential houses based on the data during the 2010-2011 Waitaha Canterbury earthquake sequence. The building information comes from the Aotearoa NZ Earthquake Commission (EQC) which is a government entity that provides natural disaster insurance to residential properties, covering damage to buildings, contents, and some coverage of land. The damage and loss data were from the EQC Claims database (providing almost total coverage of claims from natural disaster events), and the undamaged buildings' information was from the EQC portfolio database (a national building level database for every residential building in Aotearoa NZ). Then, the collected data at each intensity level were fitted to a four-parameter inflated beta distribution [77] to the damage ratios for each typology class. Figure 39 illustrates the mean damage ratio curves for LTF residential houses built pre-1940s (TWL5), 1940-1980 (TWL7), and post-1980s (TWL9). This work has been expanded from the MMI related functions to the peak ground acceleration (PGA) related functions, as shown in Figure 40.

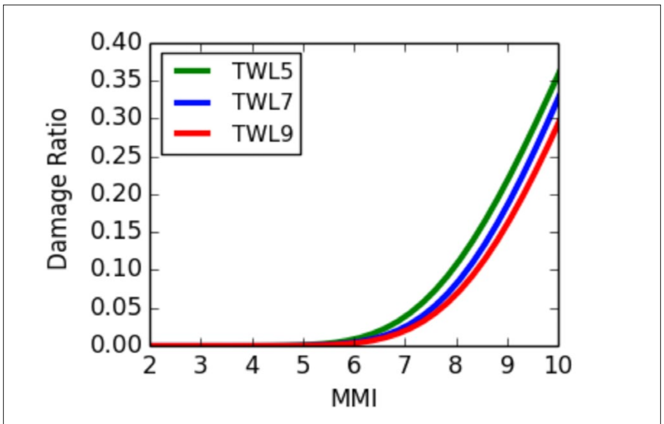

Figure 39: Vulnerability models [7] for mean damage ratios for LTF residential houses built pre 1940s (TWL5), 1940-1980 (TWL7), and post-1980s (TWL9).

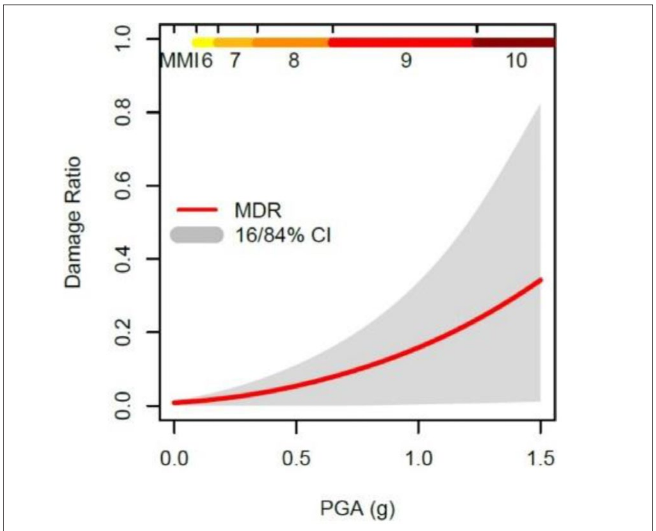

Figure 40: Vulnerability models from [7] for mean damage ratios versus PGA for LTF residential houses [78].

The Pacific Earthquake Engineering Research (PEER) Centre developed a clearly defined loss estimation framework [79,80]. The PEER framework consistently accounts for uncertainties in the relationships between earthquake hazard, structural response, seismic damage, and economic loss. This process, as shown in Figure 41, is separated into four probabilistic expressions combined using conditional probabilities to account for the uncertainties in the relationships between different parameters. Following the PEER framework, a seismic performance assessment methodology was proposed in FEMA P-58 [81], and a fragility database was established, consisting of damage state definitions, related repair costs, and repair time for most structural and non-structural components.

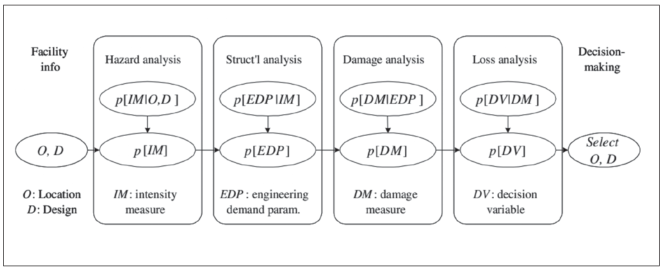

Figure 41: Seismic loss estimation framework proposed by PEER [80].

However, adapting the overseas data from the FEMA P-58 database to the Aotearoa NZ context may be challenging because Aotearoa NZ has different building practices for residential houses, different costs of materials and labour, and different repair methods. A database of Aotearoa NZ consequence functions was developed by Fox et al. [82]. In accordance with FEMA P-58, this database included the damage states, repair cost, and repair time of most components in common RC and steel frame buildings. The repair costs were collected by cooperating with a local construction company, and the 10th, 50th, and 90th percentile values of the distribution were estimated to account for the uncertainty. This dataset is freely available on the NEHRI Design Safe website [83]. Figure 42 illustrates the comparison of the repair cost between FEMA P-58 (referred to as "benchmark") and the Aotearoa NZ-specific database. It shows that there is a clear difference between them with a ratio of benchmark to Aotearoa NZ-specific repair varying from 0.25 to 2.5. To analyse the impact of the Aotearoa NZ-specific consequence data on the expected annual losses, a case study of a 12-storey steel frame building designed by Yeow et al. [84] was re-examined by using the Seismic Loss Assessment Tool (SLAT) [85].

The hazard model adopted here was an Aotearoa NZ-specific rupture forecast model by Stirling et al. [86] and ground motion models by Bradley [87] for spectral acceleration at 2.0s. Figure 43 illustrates the expected annual losses conditional on intensity including and excluding collapse cases. The results show that, at lower intensities, the losses were larger when using Aotearoa NZ-specific consequence functions, but at higher intensities the FEMA P-58 consequence functions result in larger losses.

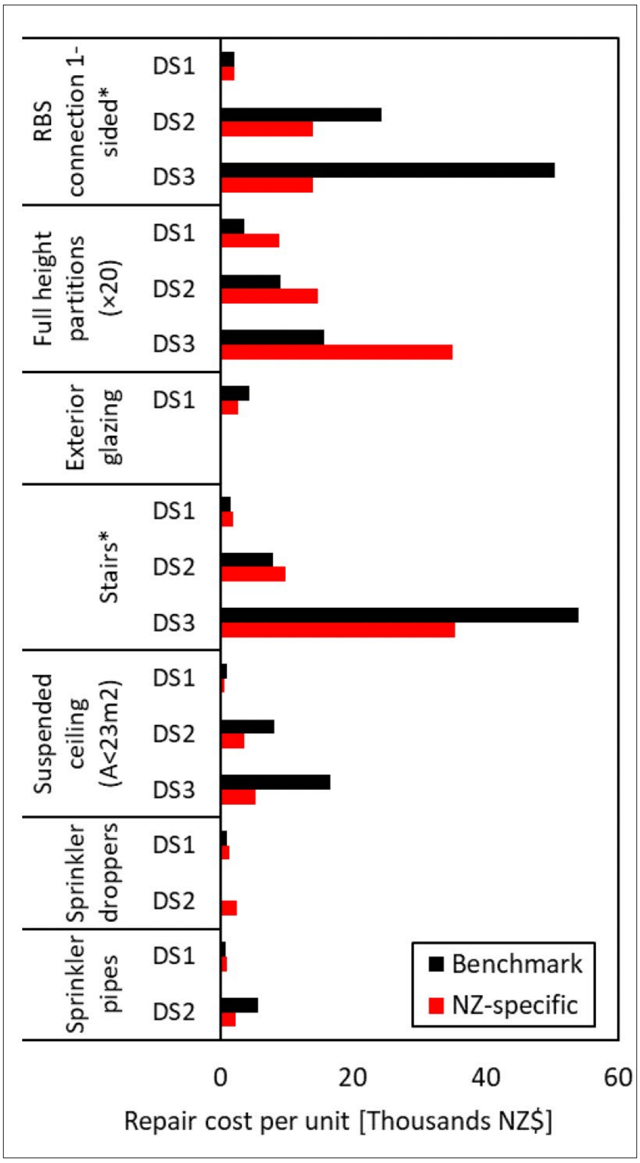

Figure 42: Comparison of the repair cost between FEMA P-58 (referred to as "benchmark") and the Aotearoa NZ-specific database developed in [82]. (*) Damage state numbers refer to the benchmark consequence functions.

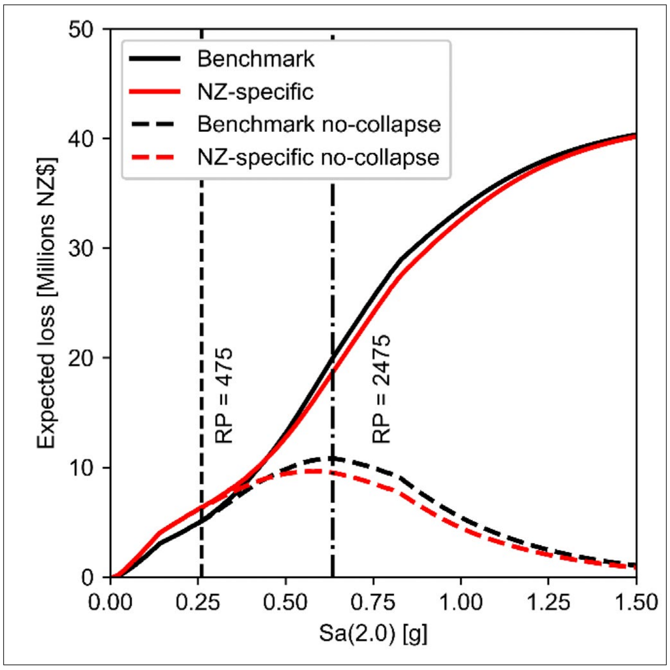

Figure 43: Expected loss conditional on intensity of the case study building (with collapse cases both included and excluded) [82].

It is important to note that the database developed in [82] did not include the common elements in LTF structures, so further contribution needs to be made for adding damage and loss functions of LTF shear walls, ceilings, roofs, etc. An experimental study about damage state quantification of LTF walls was conducted by Liu and Carradine [88]. A quasi-static cyclic test was conducted on a full-scale LTF wall and floor system. The bracing walls were arranged symmetrically in both directions and the loading was applied in the short-side direction, as shown in Figure 44. The walls were sheathed by 10mm thick standard plasterboard on the inside and 9mm thick F8 grade plywood sheets on the exterior and had hold-downs installed. The following damage was observed. Tearing of tapes between walls and ceiling initiated along the joint lines when storey drift was 0.36%. When the drift reached 0.72%, local plasterboard cracks occurred at the bottom corner of the walls along the loading direction. Noticeable load degradation occurred at 1% drift, when plasterboard damaged locally at the bottom corner (Figure 45(a)) and the vertical sheet joints of plasterboard failed (Figure 45(b)). When the drift reached 1.45%, out-of-plane buckling of plasterboards occurred on the walls along the loading direction (Figure 45(c)). The testing was terminated following the ±60mm actuator displacement (2.5% drift) cycles because of significant wall damage and reduction of the applied loads. Based on the damage observation on the test structure, the damage state definitions for LTF plasterboard bracing walls were summarised, as reproduced in Table 3. These definitions provide a meaningful linkage among the damage states, storey drifts, and potential repair actions.

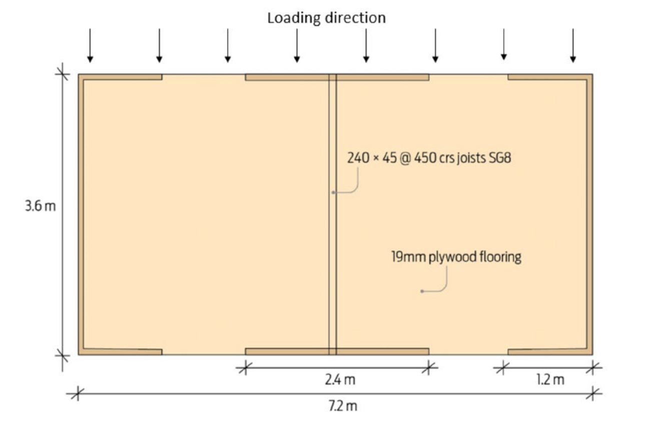

Figure 44: Floor plan of the test structure [88].

Figure 45: Damage observations of the test structure [88]: (a) local plasterboard damage; (b) failure of vertical sheet joints of plasterboard; (c) out-of-plane bucking of plasterboard.

Table 3: Damage state definitions proposed in [88].

Damage state	Description	Potential repair action	Storey drift
1	Tape wrinkling, screw distress, tape tearing in isolated area	Retape, repair stressed screws and paint	≤ 0.6%
2a	Plasterboard damage - local crushing or cracking within sheets	Replace damaged plasterboard, retape the joints and paint	0.7%
2b	Plasterboard sheet joints opening	Replace the damaged plasterboard, reinstall the screws, retape and paint	1%
2c	Plasterboards detached from the framing, Out-of-plane buckling of plasterboards	Replace the board, refix the boards to frames, retape and paint	1.5%
3	Plasterboard significantly damaged and uneconomical to repair	Repair is uneconomical. Demolition is required	1.8%

Summary and Future Research

This paper thoroughly reviewed recent research on seismic performance assessment of Aotearoa NZ LTF residential houses and development of seismic loss models for Aotearoa NZ houses. It introduced the evolution of Aotearoa NZ residential construction with the focus on seismic load-resisting systems, characteristics of plasterboard bracing walls, and experimental as well as simulation studies on the seismic performance of plasterboard bracing walls and LTF houses. The seismic damage incurred by LTF residential houses in the 2010-11 Waitaha Canterbury Earthquake sequence is summarised and the existing Aotearoa NZ seismic loss models are introduced. Based on the review presented herein, conclusions and future research directions can be summarised as follows:

The plasterboard bracing walls used in modern Aotearoa NZLTF residential houses are unique bracing systems. Overseas and Aotearoa NZ research concluded that the seismic performance of plasterboard bracing walls is clearly different from that of shear walls sheathed by wood-based panels, with lower ductility, lower strength, lower energy dissipation, and smaller ultimate displacement in the Aotearoa NZ versions.

According to the experimental results and post-earthquake observations, typical failures of plasterboard bracing walls include disengagement of screws between plasterboard and timber framing around wall corners, plasterboard cracking around screws in the wall corners, diagonal cracking of plasterboards orienting from the corners of windows and doors openings, bolts of hold-downs pulling out from the walls with hold-downs, and, in some severe cases, out-of-plane buckling and sometimes detachment of plasterboards from wall framing. It was also concluded that plasterboard bracing walls are more susceptible to damage such as cracks. However, the relationship between the damage levels and structural responses needs to be further quantified.

There are several well-established numerical simulation methodologies for the hysteretic behaviour of timber shear walls, including detailed finite element methods and macro elements methods. By adapting the existing methods to Aotearoa NZ cases, some numerical models have been developed for simulating plasterboard bracing walls and LTF structures in Aotearoa NZ. However, greater attention needs to be given to the up-lift resolution of bracing walls with or without significant axial loads as well as the studies of houses with various irregularities, such as houses with irregular plans and multi-storey houses built on slopes, which are often associated with significant vertical irregularities. Incremental dynamic analysis (IDA) could also be applied to estimate their seismic risks at different earthquake intensity levels.

Experimental tests and numerical simulations showed that the drift limit of 2.5% at ULS specified in NZS 1170.5 for general structures is not suitable for Aotearoa NZ LTF structures braced by plasterboard bracing walls. It is recommended that drift be limited to 1%, as the plasterboard bracing wall is usually severely damaged and significant strength loss could occur after 1% drift.

Most of the existing seismic loss models for Aotearoa NZ are empirical models, developed based on damage data from the post-earthquake surveys. Some attempts were made to build analytical models with Aotearoa NZ-specific loss functions following the PEER seismic assessment framework. However, only a few studies have considered classifying and quantifying seismic damage to plasterboard bracing walls. Further research should focus on developing fragility functions and seismic loss estimations. For this purpose, it is very important to collect Aotearoa NZ-specific information on damage and repair costs.

Lastly, potential avenues for future research and exploration are outlined below:

1. A more conservative design drift limit could be suggested for plasterboard bracing wall systems to reduce seismic damage in major earthquakes. Reasonable drift limits can be determined based on experimental observations and numerical simulation results. Economic benefits, social impacts and industry acceptance may also be considered when establishing new drift limits in design standards.
2. A seismic economic loss hazard model could attract wider attention and raise awareness of the economic risk associated with the current bracing system. Developing a comprehensive framework for seismic loss estimation specifically for LTF residential houses in Aotearoa NZ would also be highly beneficial for this purpose.
3. Methods to mitigate seismic vulnerability of plasterboard bracing walls need to be explored to improve their resilience. Such methods could involve the use of different types of sheathing panels, different fastener types, and the addition of hold-downs. Given these measures often lead to higher construction cost, it is essential to clearly articulate the full life-cycle economic benefits so as to inform better decision-making. Calculating the expected annual loss of retrofitted structures, based on seismic loss estimation, can help assess the viability of various mitigation techniques.
4. A deeper study on the effects of bracing irregularities in LTF residential houses is also recommended. In past major earthquakes, it was often observed that damage could be exacerbated by horizontal irregularities (with irregular floor plan and bracing wall layouts), vertical irregularities (using different bracing systems on the first and second storeys), as well as in houses built on slopes. More specific design guides could help these irregular structures improve their seismic performance and reduce earthquake damage.

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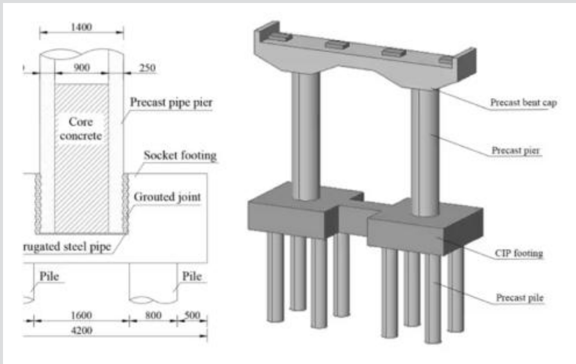

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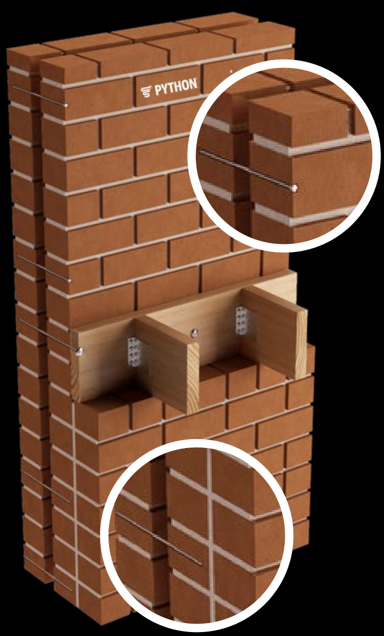

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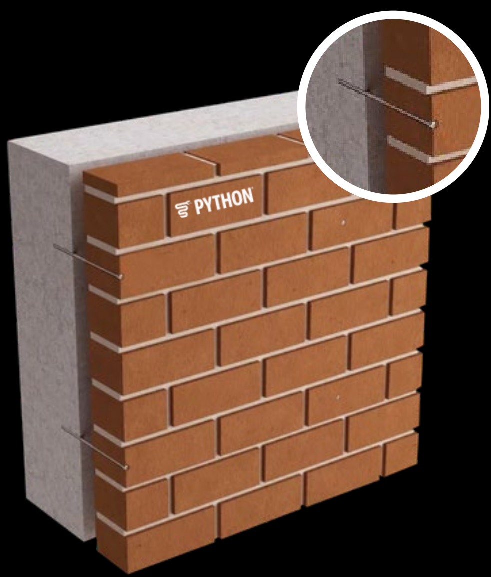

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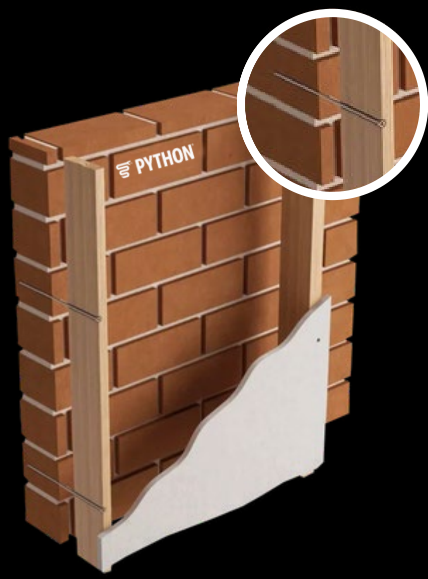

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SPECIFICATION OF WELDING CONSUMABLES FOR STRUCTURAL APPLICATIONS UNDER AS/NZS 5131

Karpenko, M.[1a], Clifton, G.C.[2b], Taheri, H.[1c], Fenemor, P.[1d], McClintock, A.[1e], & Cowie, K.[3f]

INTRODUCTION

For most structural applications designed to NZS 3404, welding consumables should be selected to match the strength of the base material. This is achieved by choosing pre-qualified welding consumables from AS/NZS 1554.1. However, it includes several options for welding consumables that the fabricator can select. This publication highlights the underlying weld design considerations and clarifies the compliance and selection requirements for welding consumables used in welded connections for steel types 1 to 7.

Weld design considerations

Ensuring that welds match the strength of the base metal is important for creating durable joints in construction and manufacturing. It was found that strength mismatch between the weld metal and the base metal significantly affects low-cycle fatigue strength; however, this effect becomes negligible in the high-cycle fatigue region [1]. Specific requirements for matching welds can vary depending on factors like material type and application. In some cases, such as high-pressure pipelines where welds are subjected to tension normal to the effective area (for example, girth welds in pipes) strength overmatching can be beneficial to avoid strain localisation in the weldment during service. Non-critical components and weld joints subjected to other types of loadings may have undermatched welds [2, 3]. Overall, careful design and testing are essential to guarantee that welded joints meet performance and safety standards.

The Steel Structures Standard, NZS 3404 [4], references the AS/NZS 1554:2004 suite of standards, which define compliance requirements for welding consumables. New editions of the AS/NZS 1554 suite of welding Standards [5] were published in 2014, and these refer to newly published editions of the AS/NZS Standards for welding consumables.

Clause 9.7.2.7 of NZS 3404 specifies that the design capacity of a full penetration butt weld should be considered equal to the nominal capacity of the weaker part of the joint. This is based on the use of matching welding consumables to ensure mechanical property alignment. Additionally, this clause specifies that for butt welds connecting Category 1 or 2 members, the ultimate tensile strength of the weld metal must be greater than or equal to that of the base metal.

For fillet welds, Clause 9.7.3.10 of NZS 3404 specifies that the design capacity should be based on the minimum (guaranteed) tensile strength of the weld metal as defined in the relevant AS/NZS ISO welding consumables standards for example, AS/NZS ISO 17632 [6], Table 1B. Clause 9.7.2.7(b) further extends this logic to partial penetration butt welds, which should be treated as equivalent to fillet welds of similar throat thickness. The nominal tensile strength of weld metal is classified as either 430 MPa or 490 MPa under the new classification system for welding consumables in accordance with AS/NZS ISO 17632, Table 1B (There is a discrepancy in the nominal tensile strength of weld metal between Table 1B of AS/NZS ISO 17632 and Table 9.7.3.10(1) of NZS 3404:1997, which should be addressed in the updated version of the NZS 3404:2026 standard). For matching the properties of weld metal with structural steel grades 300 and 350, a nominal tensile strength of 490 MPa is recommended as per required in NZS 3404 Table 9.7.3.10(1).

The 1997 edition of NZS 3404 no longer requires using the lesser of nominal tensile strength of weld metal (f_{uw}) or parent material (f_u) when calculating fillet weld capacity. It now uses f_{uw} alone. Additionally, the requirement for $f_{uw} \geq f_u$ was removed from the NZS 3404:1997. This requirement was originally intended to ensure that the weld would fail before the parent metal, thereby preventing

PAPER CLASS & TYPE: GENERAL REFEREED

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premature failure of the base material. However, AS/NZS 1554.1 now addresses this concern by requiring proper matching of welding consumables to the parent metal by providing Table 4.6.1 (A) to avoid this issue and unnecessary overdesigned welds. Still, for members experiencing significant inelastic demand, the condition $f_{uw} \geq f_u$ remains necessary.

Classification system for welding Consumables

Welding electrodes and rods are classified according to the mechanical properties of the weld metal they produce. Aotearoa New Zealand and Australia have adopted the harmonised ISO welding consumables classification system that brings together two seemingly incompatible systems in common usage:

(a) System A is used in Europe where consumables are classified predominantly by yield strength and the temperature at which 47 J minimum impact energy is guaranteed.

(b) System B is used extensively around the Pacific Rim and North America, where consumables are classified by tensile strength and the temperature at which 27 J minimum impact energy is guaranteed. “U” sign indicates that the weld metal under system B also meets the minimum impact energy of system A (47J).

For example, a flux-cored wire for applications covered under NZS 3404 with the following designation (AS/NZS ISO 17632-B: T49 3 T15 M H5) is not prequalified because the “U” sign (indicating that the weld metal under system B also meets the minimum impact energy of system A (47J)) is missing in the designation (before H5).

Aotearoa NZ and Australia have generally followed System B practice, using a tensile strength-based classification system with local variations, including a 47 J minimum impact energy requirement at the test temperature as the basis for their consumable classification requirements.

Prequalified welding consumables should meet the classification requirements of the ISO standards referenced in Section 4.6.1 of AS/NZS 1554.1, and either comply with the notch toughness designations in Columns 2 - 6 of Table 4.6.1 (A) or have approval from a ship classification society as specified in Columns 7, 8, or 9.

Welds for seismic applications

Welded connections are designed to possess an appropriate level of ductility as well as to satisfy the earthquake loading provisions. Welds in the seismic-resisting systems (category 1 and 2 systems) shall be capable of sustaining inelastic demand sufficient to develop plastic hinges in welded primary seismic-resisting members. The relationship between steel (and weld metal) properties, plate thickness, seismic strain rates and minimum service temperature has been established for a variety of steel grades using a fracture mechanics procedure [7].

Steel and weld metal respond to a decrease in temperature and/or strain hardening due to seismic loadings with a distinct decrease in toughness. A measure for assessing the toughness of steel is the Charpy V-Notch (CVN) impact test. Welds subject to seismic loads or effects should have sufficient ductility and toughness to resist brittle fracture.

While weld metal CVN is an important factor in mitigating the risk of brittle fracture, other factors to be considered are good detailing to avoid notches, weld quality, stress allocation and Z-properties of steel.

To avoid brittle fracture, the general recommendation is that the CVN test temperature of the consumables (as specified in the relevant standard for the consumables) should not be warmer than the design service temperature (refer to note 3 in Table 4.6.2 of AS/NZS 1554.1).

For steel types and service temperatures covered under Clause 2.6.4 of NZS 3404, this is achieved by selecting prequalified consumables from AS/NZS 1554.4, Table 4.6.1(A).

For seismic applications, the weld metal should achieve a minimum average impact energy of 47 J at -20°C. Clause 7.5.17.1 of AS/NZS 5131 [8] requires welds subject to earthquake loads or effects to meet the following requirements: (a) For steel types 2S, 5S, 3 and 6, the welding consumables shall have a Ships’ Classification Societies Grade 3 approval (47 J at -20°C) and (b) The heat input in a run of deposited weld metal shall not exceed 2.5 kJ/mm.

Weld metal strength matching

Different strengths between the weld metal and the parent metal are described in terms of their matching, with over-matching indicating that the weld metal is stronger and under-matching indicating the opposite. Matching is an engineering simplification, assuming that the weld metal’s tensile strength is not lower than that of the weaker part of the joint. Weld strength mismatch significantly impacts the performance of steel joints, particularly in seismic (low cycle and fatigue) loading scenarios. Overmatching, where the weld metal has a higher strength than the base metal, generally protects the weld region by redistributing stresses to the surrounding heat-affected zone (HAZ). However, this can also elevate stress concentration in the HAZ, increasing the risk of brittle fracture under cyclic or seismic loads [9, 10].

If the welds are under-matched, plastic deformation (strain) will occur in the weld metal. Where the weld metal has lower strength, it enhances ductility and plastic strain capacity in the weld but concentrates deformation at stress hot spots such as weld root. This can lower the joint's fatigue life, particularly in low-cycle fatigue conditions. Studies demonstrate that strain localisation in undermatched joints increases the likelihood of crack initiation and propagation in areas with stress concentrators such as incomplete penetration or weld toes [1, 11]. These findings emphasise the need for careful consideration of material properties and welding techniques to optimise joint performance in demanding structural applications.

FEMA 351 [12] highlights the advantages and limitations of using undermatched weld metals. While undermatched welds can reduce residual stress in joints, they are less suitable for applications involving yield-level stresses where limiting plastic deformation is critical. In such cases, balanced or slightly overmatched weld metals are recommended. Successful tests often employ weld metals with yield and tensile strengths between 58 and 70 ksi, offering matching to moderate overmatching for Grade 50 steel. For Grade 65 steel (A913), weld metals with at least 80 ksi tensile strength are advised.

For structural applications, some degree of filler metal undermatching can be acceptable for fillet welds in members subject to compression and high cycle fatigue. An engineer's judgment is required while specifying undermatched filler metal, as this increases the chances of a ductile overload fracture in the weld if this becomes the "weakest link" in the structural system. However, the degree of acceptable overmatching is not well defined in the applicable standards framework.

According to the commentary in NZS 3404, the use of weld metal with higher tensile strength than those specified in Table 9.7.3.10(1) is permitted, "provided that failure at the interface between the weld metal and the parent material is prevented". This implies that weld metal failure shall be prevented by the correct sizing of the weld.

If the welds over-match, gross section yielding is likely to occur in the parent metal, resulting in the weld deforming less than the parent metal. While some overmatching is beneficial for low-cyclic fatigue applications, a substantial overmatching will lead to a metallurgical notch effect and higher residual stresses. AISC 360-22 [13] and AWS D1.1 [14] permit the use of filler metal with one strength level greater than matching to avoid this happening.

International Institute of Welding (IIW) document IIW-X-1504-03 [15] includes recommendations that the yield strength of the weld metal should overmatch the actual strength of the parent material of the beam and column. Undermatching weld metal strength should only be used if additional strengthening has been provided.

The increase in weld metal strength leads to a higher level of residual stress and a greater susceptibility of the welded metal to cold cracking, particularly in the transverse direction of the weld. The susceptibility of the weld metal to cold cracking increases with the rise in diffusible hydrogen levels. Hydrogen-controlled consumables produce less than 15 mL/100g of diffusible hydrogen (HD) in the weld metal. The commonly used (prequalified) flux-cored arc welding (FCAW) gas-shielded metal-cored wires typically produce HD levels of 5–15 mL/100g of deposited weld metal [16].

To mitigate the potential effects of increased weld metal strength on cold cracking, it is recommended to limit the hydrogen content in overmatching flux-cored tubular electrodes and Manual Metal Arc Welding (MMAW) electrodes to HD 5mL/100g of deposited weld metal. This hydrogen level is designated as "H5" in FCAW wire specifications. The use of seamless flux-cored wires is also recommended, as they are less susceptible to contamination or hydrogen absorption than seamed wires.

With regard to structural applications covered under NZS 3404, the use of over-matched welding consumables should be limited to one classification step higher than the prequalified welding consumables listed in Table 4.11(A) of AS/NZS 1554.1. The higher-strength welding consumables should comply with AS/NZS 1554.1, Table 4.6.1(A), as applicable to steel types 7C and 8C, or AS/NZS 1554.4, Table 4.6.1(A).

Engineers should be aware that AS/NZS 1554.1:2014, Table 4.6.1(A), includes an option for under-matching consumables, such as T-B43. The recommended approach is to use the highest weld metal strength prequalified for the selected steel type, as specified in AS/NZS 1554.1:2014, Table 4.6.1(A). The design specification should include the minimum weld metal ultimate tensile strength as f_{uw} =490 MPa.

Overmatching welding consumables are considered not-prequalified to Table 4.6.1(A) of AS/NZS 1554.1. They require qualification by testing to Clause 4.6.2 of AS/NZS 1554.1.

Examples of acceptable overmatching welding consumables classification are included in Table 1 based on the FCAW process:

Table 1: Applicable overmatching welding consumables based on the FCAW process

Steel types	Matching AS/NZS 1554.1, Table 4.6.1(A)	Overmatching AS/NZS 1554.1, Table 4.6.1(A)	Ship classification societies' approval
2S, 5S, 3 and 6	AS/NZS ISO 17632 A-T42 3 B-T49 3U	AS/NZS ISO 17632 A-T46 3 xxH5 B-T55 3U xxH5	3 (or higher)

Weld metal overmatching case study

For example, a fabricator is considering purchasing wire A-T46, which does not meet the B-T49 grading requirements. However, it complies with Clause 7.5.17.1 of AS/NZS 5131, specifically the Ships 'Classification Societies Grade 3 approval. The question is whether this wire can be qualified through testing as per Clause 4.6.2 of AS/NZS 1554.1.

The ISO 17362 AT46 classification is one step higher than the prequalified welding consumables listed in AS/NZS 1554.1, Table 4.6.1(A). The specification should include the level of diffusible hydrogen as H5. The wire may be used, provided it undergoes testing in accordance with Clause 4.6.2 of AS/NZS 1554.1, which involves testing based on a butt test piece (12mm butt test plate). The rest of the welding procedures for butt welds and fillet welds can be qualified by macro test.

Buckling considerations

The resistance to local buckling of members could be affected by the level of residual compressive stress generated during manufacturing or fabrication. The plate slenderness yield limits specified in Table 5.2 of NZS 3404 are highest for stress-relieved members and lowest for those that are heavily welded. In this context, the Standard differentiates between lightly and heavily welded members using a residual stress threshold of 40 MPa. Since higher heat input during welding leads to greater residual stresse, members with smaller welds or those using multi-pass welding techniques are generally considered lightly welded.

Furthermore, for members subject to axial compression, the nominal capacity (N_c) incorporates the effects of flexural buckling. It is determined by multiplying the nominal section capacity (N_s) by a geometric slenderness reduction factor (α_c). This reduction factor (α_c) is influenced not only by the geometrical slenderness ratio (L_e/r), but also by the yield stress and the type of section. The section type itself is characterised by the member section constant (α_b). There are five specified values for α_b, each corresponding to different section types and the associated residual stress profiles and distributions.

Accordingly, when the strength of the weld metal is significantly higher than that of the base material, it can measurably influence the magnitude and distribution of residual stresses and, in turn, affect the member's buckling performance.

Qualification of welding procedures

Welding procedures should be requalified using the same shielding gas mixture classification as recommended by the manufacturer and listed by the Shipping Classification Society for the original Grade (e.g. 3) approval for that weld filler metal.

The welding conditions used such as amperage, voltage, travel speed, etc. shall be within the range recommended by the manufacturer and the composition of the shielding gas shall be reported.

A change in consumable classification an increase in filler metal strength and a change from a hydrogen-controlled consumable to a non-hydrogen-controlled consumable or any increase in hydrogen classification of the consumable require a new WPS (Table 4.11(A) of AS/NZS 1554.1.

Weld specification (design engineer)

For structural applications involving Steel Types 1 to 7, the following requirements shall apply:

1. The weld metal shall have a minimum ultimate tensile strength of f_{uw}= 490 MPa.
2. The sizing of the welds should be based on f_{uw}=490 MPa.
3. Welding consumables shall be matched with the respective steel types as specified in AS/NZS 1554.1, Table 4.6.1(A).
4. Overmatching weld metal is permissible, provided that

the increase in weld metal yield and tensile strength does not exceed one classification step higher than the prequalified welding consumables with f_{uw}=490 MPa listed in AS/NZS 1554.1, Table 4.6.1(A).

5. The higher-strength welding consumables should comply with AS/NZS 1554.1, Table 4.6.1(A), as applicable to steel types 7C and 8C.
6. For welds in Category Members 1, 2, and 3, as defined by NZS 3404, the welding consumables shall have Ships' Classification Societies Grade 3 approval or higher (Clause 7.5.17 of AS/NZS 5131).

Welding consumables ordering checklist (fabricator/supplier)

1. Are welding consumables designated to one of the standards listed under AS/NZS 1554.1: 2014 Table 4.6.1(A)?
2. Are the welding consumables matched with the steel types in accordance with AS/NZS 1554.1: 2014 Table 4.6.1(A)?
3. Does the weld metal have a minimum ultimate tensile strength of 490 MPa?
4. Do the welding consumables have a Ships' Classification Societies Grade 3 approval and meet the requirements of Clause 7.5.17 of AS/NZS 5131 (seismic welds)?

Welding consumables application checklist

1. Is the storage of welding consumables in compliance with the manufacturer's recommendations?
2. Are the welding consumables being used within the parameter range specified by the manufacturer's recommendations?
3. For seismic applications, does the arc energy input comply with Clause 7.5.17.1 of AS/NZS 5131?
4. Is the shielding gas being used in accordance with the required Ships' Classification Societies (e.g. Grade 3) approval?
5. Have the test certs and batch numbers of welding consumables been recorded and traceable to the welds?

Summary

The design capacity of the welds designed to NZS 3404 is based on the minimum (guaranteed) tensile strength of the weld metal as defined in the relevant AS/NZS ISO welding consumables Standards.

Welding consumables shall be matched with the respective steel types as specified in AS/NZS 1554.1, Table 4.6.1(A).

For structural applications involving Steel Types 1 to 7, the weld metal shall have a minimum ultimate tensile strength of f_{uw}=490 MPa. Overmatching weld metal is permissible, provided that the increase in weld metal yield and tensile strength does not exceed one classification step higher than the prequalified welding consumables with f_{uw}=490 MPa listed in AS/NZS 1554.1, Table 4.6.1(A). The sizing of the welds should still be based on f_{uw}=490 MPa.

Welds subject to seismic loads or effects should have sufficient ductility and toughness to resist brittle fracture. Clause 7.5.17.1 of AS/NZS 5131 requires welding consumables used for seismic applications to meet Ships' Classification Societies Grade 3 approval (47 J at -20°C).

It is recommended to limit the hydrogen content for overmatching flux-cored tubular electrodes and MMAW electrodes to H5.

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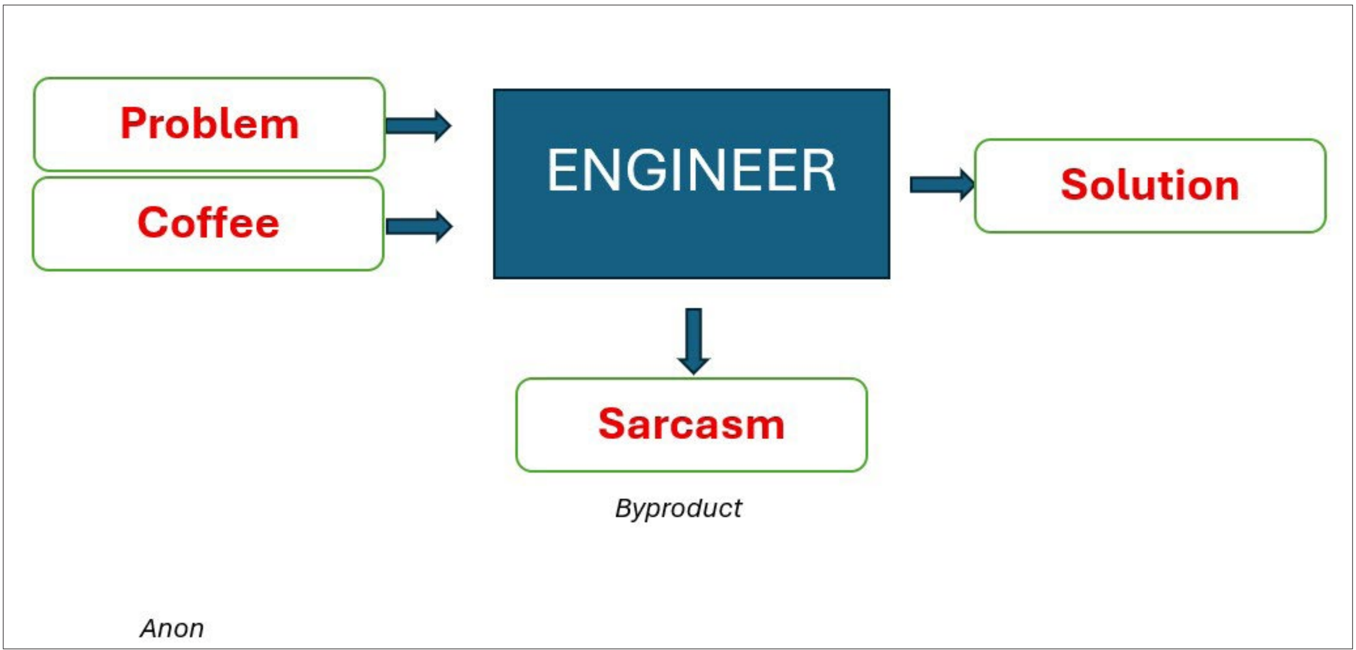


ENSURE QUALITY AND COMPLIANCE: SPECIFY READY MIXED CONCRETE FROM AN AUDITED PLANT

concretenz
BUILDING RESILIENCE
AUDITED PLANT
Quality Assured NZS 3104

Structural engineers play a pivotal role in ensuring the durability, safety, and performance of concrete structures. One of the most effective ways to achieve this is by specifying that ready mixed concrete for their projects comes from a plant holding a current certificate of audit issued through the Concrete NZ Plant Audit Scheme.

This certification provides assurance that the concrete meets industry standards and project requirements, offering a range of benefits to engineers, clients, and the wider construction sector.

INDEPENDENT ASSURANCE OF QUALITY

The Plant Audit Scheme provides an independent and rigorous audit of the quality systems in place at ready mixed concrete plants. Without this scheme, engineers and project specifiers would need to conduct their own costly and time-consuming audits for each construction project. By choosing audited plants, engineers gain confidence in the consistency and compliance of the concrete supplied, reducing the risk of quality-related issues during construction.

COMPLIANCE WITH INDUSTRY STANDARDS

The scheme ensures compliance with *NZS 3104:2021 Specification for Concrete Production*, a critical standard governing the manufacture of concrete in New Zealand. It also aligns with other key standards, including:

- *NZS 3109:1997 Concrete Construction* – governing structural concrete performance.
- *NZS 3122:2009 Specification for Portland and Blended Cements* – ensuring cement quality.
- *NZS 3112.1-4:1986 Methods of Test for Concrete* – setting criteria for material and product testing.

Structural engineers specifying concrete from an audited plant can be assured that the materials and production processes meet these rigorous requirements, supporting compliance with the *New Zealand Building Code.*

RIGOROUS AND ONGOING AUDIT PROCESS

The Plant Audit Scheme is managed by a committee of registered professionals, including representatives from Concrete NZ and Engineering New Zealand (ENZ). The committee operates under a quality assurance programme certified to ISO 9001, with audits conducted by Bureau Veritas (New Zealand) Limited. This independent oversight ensures that the scheme maintains high and consistent standards across all participating plants.

Audited plants must:

- Employ appropriately qualified staff for concrete testing.
- 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.

CROSS AUS & UK

USE OF LOAD-BEARING POLYSTYRENE IN STRUCTURES

(CROSS AUS November 2025)

Concern has been raised about the apparent lack of availability of detailed technical information regarding the use of 'structural polystyrene' in Aotearoa New Zealand.

Designers, required to use polystyrene to improve thermal properties in situations where structural loads exist, currently lack guidance to produce safe designs. Education on the properties of this material is urgently needed.

Reporter's Submission

The reporter is concerned about the growing use of 'structural polystyrene' in load-bearing situations.

The recent trend in Aotearoa NZ towards increasing thermal efficiency appears to have caused rapid growth in the use of polystyrene products supporting structural loads, including beneath footings. It is the reporter's view that in Aotearoa NZ there are currently no codes, no established design procedures, no approved technical literature, nor any other guidance on which design engineers can rely to demonstrate compliance with New Zealand Building Code Clause B1 Structure or Clause B2 Durability, or to justify the material's use as an alternative solution.

For example, some proprietary floor slab systems state that their slabs are subject to Specific Engineering Design (SED) in accordance with 'the technical literature'. However, no technical literature is supplied, and it is not clear what literature is referred to. A recent example is a matter in which a local council unilaterally imposed a condition on an architect for a project with a waffle slab (designed by the reporter) to require a 50mm thick layer of polystyrene under the entire slab footprint. The reporter only discovered this when the builder enquired about which grade of polystyrene should be used.

The reporter is concerned that engineers are required to specify and accept the use of polystyrene to support structural loads without detailed design information, while such products are now widely employed for that very use. Without that information, in the view of the reporter, engineers are to some extent designing by guesswork.

Expert Panel Comments

The use of polystyrene in structural engineering as a non load-bearing material is not uncommon, with polystyrene blocks often employed as permanent void formers in concrete waffle slabs on ground. In marine environments, they can also be used to assist buoyancy in floating structures. However, due to their tendency to decay and break apart when subject to weathering, they tend to be encased in such environments.

The reporter's concern, however, is about the adoption of polystyrene as a structural load-bearing material, and the apparent lack of information available for the design of such use, particularly in Aotearoa NZ.

In Australia, where an important application of the product is under freezer slabs in supermarkets, polystyrene products in the market appear to have been (or can be specified to be) tested to AS 2498.3 Methods of testing rigid cellular plastics, Method 3: Determination of compressive stress. Testing of this nature by a National Association of Testing Authorities (NATA) accredited laboratory may provide adequate assurance of compressive capacity, but designers may also want to pursue issues of durability, and where relevant, creep characteristics relative to adjacent or other composite materials, as well as susceptibility to fire. It is the reporter's opinion that such information is not readily available in Aotearoa NZ, nor is any guidance on design methodology.

In the absence of sufficient design information, designers should exercise extreme caution in deciding how or whether to adopt the incorporation of polystyrene as a load-bearing material. As an absolute minimum, independent performance certificates and a warranty from the supplier should be provided prior to proceeding.

The use of polystyrene in certain composite structural elements is not a new development. In many countries, polystyrene sandwiched between steel or aluminium sheeting is available for cladding purposes. In some cases, durability has proven to be problematic with products having failed due to delamination from heat or decay caused by moisture ingress. Safety in fire conditions may also be an issue.

Structural Insulated Panel systems (SIPs) using polystyrene or other foams such as polyurethane between timber sheets have also been used for wall systems. Many manufacturers exist worldwide who have undertaken extensive testing for load-bearing, fire rating, and other similar requirements. These panels are widely regarded as an acceptable structural system for these applications.

BOLTED CONNECTION FAILURE IN A STEEL ROOF TRUSS

(CROSS AUS 7 November 2025)

This report concerns the partial failure of a roof truss in a large, freestanding roof structure. A subsequent investigation found that this appears to have been caused by the failure of the bolted connection at the end of a chord member in one of the trusses.

The design capacity of the bolts calculated in accordance with AS 4100:2020 - Steel structures was much less than the calculated design force in the member under the wind conditions at the time.

Reporter's Submission

The reporter suggests that findings from the investigation into this incident could help engineers avoid a similar failure. They emphasise that the correct design, specification, and installation of bolted connections is critical in steel structures.

Bolts may represent only approximately 1-2% of the total steel package, and only about 0.1-0.2% of the total structure cost, but they hold the entire structure together and are absolutely crucial to a successful structural outcome.

The reporter also notes that the following Technical Notes from the Australian Steel Institute (ASI) can assist designers, procurers, and installers:

- TN-001, High strength structural bolt assemblies to AS/NZS 1252:2016
- TN-007, Compliance issues and steel structures
- TN-016, Installation of bolted connections to AS/NZS 5131

Expert Panel Comments

The reporter has drawn attention to a common problem. Many papers have been written on this topic, and much research has been undertaken.

This situation could have been much worse, as the wind conditions at the time are likely to have been significantly less onerous than those in an ultimate limit state.

Bolts, nuts, and washers are critical design components in steel-to-steel and steel-to-concrete connections. Almost no bolts are now manufactured in Australia or Aotearoa NZ. Whilst high-quality structural bolts are available from overseas, it is essential to ensure that all bolts comply with Australian and Aotearoa NZ standards and are supplied by a reputable supplier.

Accordingly, engineers must clearly document and specify all bolts used in the connection and erection of structural steel and concrete structures, referencing the appropriate standards that the bolts and their associated nuts and washers must comply with.

In Australia and Aotearoa NZ, the designer of the structure is responsible for the design of end connections. 'Standard' details may be adequate and relied upon for 'standard' situations, but their adequacy should be confirmed for combined force envelopes on specific connections. Critical connections that could result in disproportionate failure (for example, supporting transfer beams) should be given particular attention. A robust review of the design by an independent engineer should always be carried out.

Secondary effects such as eccentricities should not be ignored. Avoiding offsets where possible and considering centre-line alignment in all planes can help reduce additional demand. In addition, flexible end plates can create prying effects and can substantially increase bolt tensions. Adequate rigidity when specifying end plate thickness can help to avoid this type of scenario. Similarly, avoiding large bolt groups with multiple columns of bolts can ensure load paths are clearly understood. Wherever possible, connections providing inherent ductility should be employed.

As the complexity of structures increases, enhanced skill and knowledge are necessary together with more coordination and liaison between internal and external parties. In these cases, designers should work together with the detailer/contractor to develop the models so that the resulting structure can be fabricated and erected safely and cost-effectively. The fabrication shop drawings should be reviewed by the original designer to ensure the proposed fabricated connection complies with the original design requirement.

Durability is also essential, and typical bolt finishes include:

- plain
- zinc-plated
- galvanised (most structural PC 8.8 bolts are hot dip galvanised)
- stainless steel (used near the sea and in other aggressive environments)

Some finishes are not suitable for all environmental conditions, and the level of exposure to the weather and potentially hazardous conditions may require bolts with higher levels of durability.

HIGHER RISK BUILDING (HRB) RESIDENTS' CONCERNS ABOUT SAFETY NOT HEARD

(CROSS UK 20 January 2026)

This report involves a cladding and building remediation project on a Higher Risk Building (HRB). The process was fraught with challenges, and crucially, says the reporter, residents' voices were not heard or considered on some safety matters.

Reporter's Submission

Due to this lack of consultation, residents may continue to harbour concerns about the safety of their building, and some significant safety risks are said to remain unresolved. There were a number of issues during the process and this report focusses on one of them.

As part of the works, new doors were installed onto balconies which resulted in a raised threshold being added to existing doorsteps. The change in section involved adding door frame bottom members and a projecting external sill to create an upstand in the middle of what was previously a flat topped step. This has resulted in a trip hazard which is difficult and potentially hazardous for residents to navigate. The change to the step is shown diagrammatically in Figures 1 and 2.

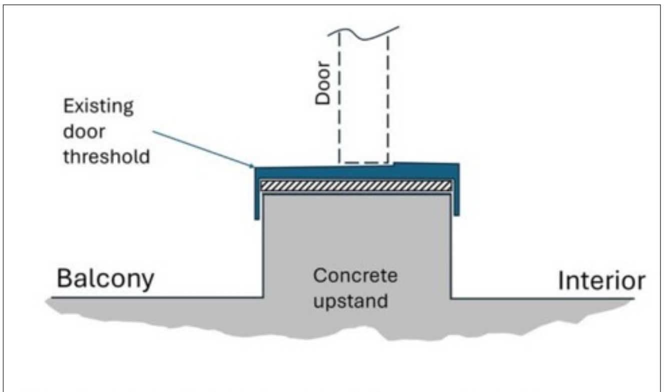


Figure 1: Indicative balcony door threshold prior to the works

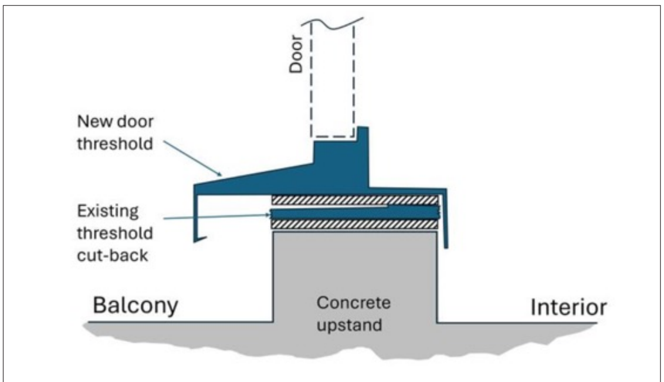


Figure 2: Indicative balcony door threshold after the works

After much campaigning by residents with support from external groups, the owner finally accepted that the thresholds needed to be redone. The revised design will have a lower threshold but still be higher than the original. Of concern is that the residents have been told that this remediation will only be carried out if requested by the occupants of individual apartments. The reporter questions why, if the thresholds are unsafe, they are not all being remediated? In their view a potentially unsafe feature should be universally corrected.

Residents questioned the building regulation implications of raising the threshold height and believed that even if the new height was compliant this did not remove the trip hazard. It took several difficult and lengthy meetings to reach a position when it was agreed that the original remediation work should be redone to improve the newly installed thresholds.

Experiences with the new thresholds have included one resident suffering a fracture when tripping on the step, many residents having caught shins on the top of the step with consequent bruising, and many elderly residents no longer being able to use the balcony as it is too high and too wide for them to step over.

Addressing resident concerns is crucial for effective risk management in refurbishment projects, and listening can ultimately reduce project costs, avoid rework, and minimise compensation claims. The cost and wasted resources of not listening to residents should, concludes the reporter, be a warning to all. This is particularly important after the findings of the Grenfell Tower Inquiry[1].

Expert Panel Comments

The report highlights issues of resident engagement and building safety and focusses on concerns about hazards associated with a step onto the balconies. Residents' input is critical since they live with these buildings daily, and ignoring their concerns risks disquiet. Residents should not only be safe but also feel safe. Failing to listen erodes trust and the sense of safety amongst them.

The unresolved technical issue appears to be the introduction of a trip hazard between flats and balconies. While this design is not ideal, an upstand is a common feature in existing UK housing. BS 8579:2020 - Guide to the Design of Balconies and Terraces gives detailed information about upstands and access requirements.

In this context, it should be noted that slips, trips, and falls in buildings cause more deaths than fires or structural failures.

1. The fire in the Grenfell (West London) tower block in 2017 killed 72 people. The inquiry found all their deaths were avoidable. It concluded that the fire was the result of a chain of failures by governments, "dishonest" companies and the fire service.

Comments suggest that the local authority's unresponsiveness is already widely recognised and that the report appears to reflect more on internal concerns and management issues than strictly technical failures.

The premises is a Higher Risk Building (HRB) so there are mechanisms under the Building Safety Act 2022 for residents to report concerns. However, it appears that these may not be producing the anticipated outcome and in due course the matter may come to the attention of the Building Safety Regulator for a resolution.

Following the Grenfell tragedy, it was rightly determined that residents' voices must be heard, and safety issues addressed quickly and visibly. When safety concerns are raised, they must be acted upon, and the obligation to engage with any source of safety reports should be emphasised.

This is an unusual report for CROSS in that it is about people and culture rather than technical issues but it introduces an important point relating to structural and fire safety which is likely to be significant. We welcome other reports on the cultural aspects of safety as part of our commitment to improve safety for the public, those engaged in building works, and fire and rescue services.

Overall, this report serves as a valuable reminder that building safety is as much about listening and transparency as it is about technical compliance. It encourages ongoing vigilance and a resident-centred approach, both of which are essential for maintaining public trust and ensuring that safety concerns are addressed before they escalate into major issues.

TEMPORARY STABILITY RISK DUE TO CONSTRUCTION CHANGES ON BRIDGE DECK

(CROSS UK 13 January 2026)

During the final stages of design for an approximately 20 m span filler beam bridge deck, shortly before the Category II check was completed, the contractor altered the intended construction sequence. Instead of lifting steel beams in connected pairs, as initially planned, the beams were to be lifted individually.

The contractor's change introduced a significant temporary stability concern: the risk of lateral-torsional buckling (LTB) of the beams under the weight of the wet concrete applied during construction.

Reporter's Submission

This report from a Category II checker addresses the design and construction process for a filler beam bridge deck. It explains the events and decisions that arose when a late change in the construction sequence introduced critical temporary stability risks.

Figure 1 shows the pair of beams intended to be connected together by pre-casting the concrete infill to provide temporary stability for each pair. The contractor's change introduced a significant temporary stability concern: the risk of lateral-torsional buckling (LTB) of the beams under the weight of the wet concrete applied during construction.

The Category II checker identified the separation of the beams into individual units for lifting as a critical issue. The main beams, if left unbraced and without rotational restraint at the ends of the flanges, could experience buckling under a utilisation factor of over 200%. This demonstrated that, in such conditions, the beams were at risk of failure. The designer confirmed this assessment and notified the contractor that temporary LTB restraints would be necessary to ensure the stability of the beams during construction.

To facilitate the works, the designer offered to undertake the temporary bracing design and coordinate its inclusion with the permanent works. However, the contractor declined the proposal, insisting that their own temporary works design team

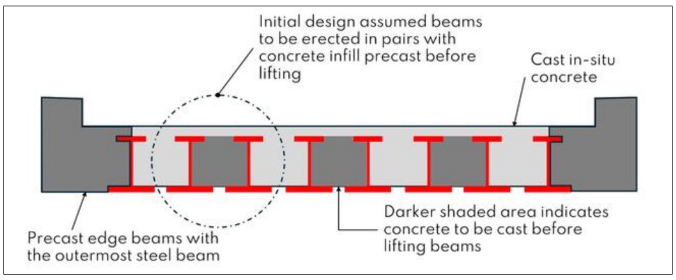


Figure 1: Cross section through the bridge deck as designed showing pairs of beams with concrete precast before lifting. Note that this is not the final design which incIded bracing.

would develop a separate bracing scheme. To assist coordination and clarify responsibilities, the designer added appropriate notes and highlighted hazards on the permanent works drawings to cover the interface between the permanent and temporary works.

Following these adjustments, the design received client approval, and the permanent works drawings were formally issued. The sequence of events, however, did not end there.

Approximately three months after the initial issue was identified and resolved, and just two weeks before the deck concrete was scheduled to be cast, the contractor and their temporary works designer (TWD) challenged the necessity of the temporary bracing once again. This renewed issue was raised with the designer, as the contractor was arguing that the bracing was not required. In the opinion of the reporter, this indicated a fundamental misunderstanding on the part of the contractor and their TWD regarding the purpose and function of the temporary bracing.

Discussions led to further commercial negotiations and programme considerations about the design and verification responsibilities for the temporary bracing. The conversation centred on whether the contractor's in-house temporary works team or the permanent works designer should be accountable for delivering the required design, as well as how this would affect the project's progress and resourcing.

Eventually, and within a few days of the concrete pour, the contractor asked the designer to devise a scheme for the bracing. By the time the bracing design was being undertaken, most of the deck reinforcement had been fixed, restricting the available space for the installation of bracing and complicating its installation. The final bracing design tied the inner girders back to the precast edge beams, which have much greater lateral stiffness and were sufficient to act as restraint.

The design and check programme for the bracing was compressed, and both teams had to work late at night and with less 'thinking time' than would usually be the case, which could have led to errors in the design.

The case highlights the complexities that can arise when late changes are made to a construction sequence, especially when such changes introduce significant temporary stability risks. It also underscores the importance of clear communication and understanding between designers, checkers, and contractors regarding the responsibilities and technical necessities for temporary works. The need for robust and timely coordination between permanent and temporary works design teams is essential to ensure safety, avoid misunderstandings, and maintain programme and commercial certainty. In general, says the reporter, structural safety would be improved if permanent works designers took more responsibility for temporary stability, even if the bracing itself was designed by a third party.

The event was classified by the reporter as a near miss.

Expert Panel Comments

The series of events described by the reporter demonstrate a series of technical and procedural challenges. Of particular concern were individual beams, which were likely susceptible to lateral torsional buckling. The additional weight from the concrete infill exacerbated the problem, increasing both the size and cost of lifting the steel-concrete pairs and motivating the decision to lift beams individually. During lifting, single beams in a temporary condition are especially vulnerable to lateral torsional buckling due to the potential for load reversal.

Procedurally, the role of the Temporary Works Coordinator (TWC) appears to have been underdelivered. The TWC is responsible for ensuring the clarity of design responsibility at each stage of the structure's life. In this case, the lack of clarity on temporary condition design responsibility was further complicated by the shift away from using precast concrete for stabilising central pairs of beams.

Communication and responsibility

Effective communication is a crucial skill for engineers, especially regarding safety-critical information. Although the contractors were informed early on about the need for temporary bracing, it remains unclear if this message was properly understood. Engineers have a duty to verify comprehension; asking recipients to repeat back and explain instructions can be a valuable practice. Too often, professionals assume an intuitive understanding that may not exist outside their own expertise.

The report's section on the responsibility for designing temporary works also raises concerns. Without access to contracts and scope of services, a definitive opinion is not possible, but clarity of responsibility is essential. The stability of structures in temporary configurations should be explicitly addressed, particularly in such a case when braced pairs of beams are assumed before pouring concrete.

Despite being classified as a near miss, this incident demonstrates the critical value of independent checking. Without it, the issue could have gone unnoticed, potentially leading to failure and harm.

Technical analysis and practical considerations

To vividly illustrate the risk of collapse, physics-enabled software may be used, demonstrating in real time what happens as concrete is poured and elements fail. Such visual tools can be powerful in persuading doubters of the risks.

The lack of seriousness with which bridge construction is sometimes treated is disappointing, highlighting the importance of the Category II checker's role in identifying such issues.

Lessons and broader implications

There are numerous records of failures during construction when final design conditions and restraints are not present. This report underscores the necessity for strong interaction between design and construction teams as miscommunication or misunderstanding between these groups can have severe consequences. This lesson extends beyond bridge construction to all structural works, emphasising the need for comprehensive education and training for engineers.

Key technical points

- Single beams being lifted are vulnerable to lateral torsional buckling, especially if there is load reversal during lifting
- Beams can also buckle when supported only at their ends because the top is in compression and has no restraint at either end
- Simply providing cross-bracing between pairs of beams is not generally sufficient; plan bracing is needed to prevent both beams from buckling together. Lack of appropriate bracing has led to major failures in the past

Updates to standards and guidance

Following a review of the consideration of temporary conditions such as described in the report, Network Rail has produced a definition of 'temporary conditions' and is updating relevant standards and guidance. A new engineering advice note was published in September 2025, entitled B&S Engineering advice note EAN03/2025, a consideration of temporary conditions. This highlights the effects of neglecting temporary conditions and gives guidance to ensure proper coordination between temporary and permanent works in design and execution. It is available to all who register for Network Rail standards via the "Reviewing our Standards" webpage.

Communication of lateral torsional stability

The need for lateral torsional stability in all temporary conditions must be clearly communicated as some permanent works designers will need reminding about such issues. Permanent works and temporary works designers depend upon each other and must be open in exchanging ideas for their mutual benefit.

UPCOMING CONFERENCES

- **Bridge and Geotechnical Conference 2026**
  New Zealand International Convention Centre in Tāmaki Makaurau Auckland,
  August 25–26, 2026.
- **Woodworks 2026 Conference**
  Heretaunga Hastings, 28-29 October 2026
- **SESOC Conference**
  Cordis Auckland, 30 June – 2 July 2027.
  Papers accepted early 2027 – yet to be advised (note that web page yet to be finalised)

SESOC Sustainable Design Task Force

Report for SESOC Management Committee Meeting – February 2026

Prepared by Charlotte Toma

General Updates:

- Regular monthly meetings have been held in December and February. The make up of the Task Force covers all primary structural materials, different practitioner organisations, small and large companies, contractors, central government and academics. Now at 20 people strong, the Task Force will look to implement a "one in – one out" policy going forward.
- In December, the Task Force hosted Rachelle Habchi, the incoming Chair for the SEAOC Sustainability Committee. She talked through California's Embodied Carbon Regulation, and the proposed changes to California Air Resources Board (CARB). The Task Force also discussed opportunities for collaboration on topics including developing carbon baselines, specifications, and seismicity and carbon.
- The Task Force is contributing to a session within the IStructE Global Safety, Sustainability and Resilience conference: Australasian Session on the 10th of March, where there will be presentations on the work of the Task Force, Seismicity and Carbon and also the EPB Reform.
- The ECO2ALP Pilot developed by Nick Carman (Mott MacDonald) in collaboration with MBIE has had over 80 projects added to the database, and was noted in the second emissions reduction plan (ERP2). MBIE funding has been secured to progress from pilot to procurement, and a Request for Quotation (RFQ) issued to incumbent platform suppliers; there has been good engagement to date. The draft timetable suggests the new platform may be live in October 2026.
- Charlotte has given her intention to step down as Chair, and the process is underway to appoint a new chair.

Work underway:

- Brendan Donnell has been working with Masterspec to develop technical specification clauses to cover the common structural engineering materials of concrete, steel and timber with respect to embodied carbon. The objective is to make it easy for designers to specify materials with reduced embodied carbon, driving faster uptake of those materials.
- To sit alongside this, and support wider demand for low carbon materials, the Task Force have drafted the Embodied Carbon Specification Guide for structural construction materials. The guide has now been through extensive industry consultation and is in final editorial review stages before publication. This document provides technical guidance on specifying commonly used structural materials to reduce embodied carbon emissions in construction projects.
- The Task Force has drafted a revision to the SESOC Design Features Report to include a position on Carbon and high-level carbon reporting. Currently this is being reviewed by the Task Force which will then work with the Management and Communications team to integrate into the wider Design Features Report update.
- The Task Force has been discussing a webinar on Modern Slavery and Materials, and had a session on Planetary Boundaries.
- The Task Force are in early planning stages for a SESOC Sustainability Summit in 2026.

SESOC Emerging Structural Engineers (ESE) Group

Chair: Henry Rowden

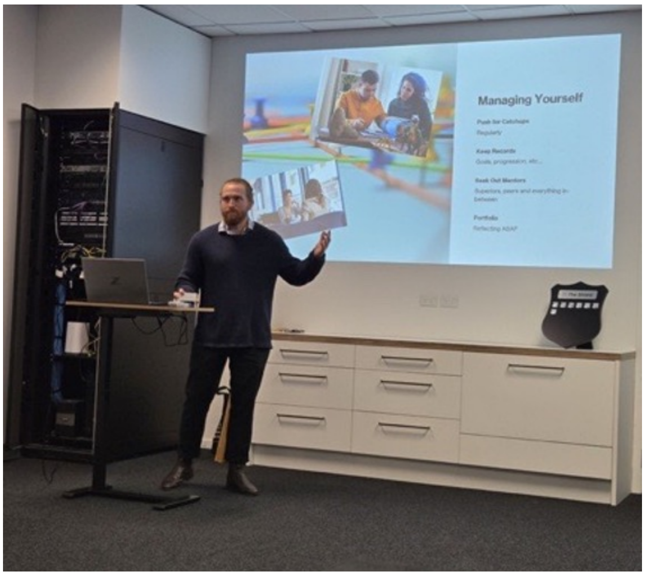
Contact: ese-chair@sesoc.org.nz

RECENT NEWS / UPDATE

- We currently have 721 members subscribed to the SESOC ESE mailing list, and 774 followers on LinkedIn (please follow us if you don't already).
- Alex Paterson & Rico D'Anvers from Holmes have recently joined the Tauranga/Hamilton ESE Chapter Committee.
- Faakhira Hassan from Holmes has recently joined the Te Whanganui-a-Tara Wellington ESE Chapter Committee.
- Matt Lindsay from WSP has recently joined the Ōtautahi Christchurch ESE Chapter Committee.
- We are currently looking for ESE Committee representatives in Christchurch, Wellington, Whakatū Nelson, Ōtākou Otago/Tāhuna Queenstown. If you are interested, please contact Henry Rowden at ese-chair@sesoc.org.nz.

RECENT PRESENTATIONS / ACTIVITIES

Lightning Talks Event, Tāmaki Makaurau Auckland - 18 September 2025



The Tāmaki Makaurau Auckland Chapter of the SESOC ESE Committee recently held a Lightning Talks event at the Mitchell Vranjes office in Grafton. The talented speakers gave some special insights and advice, draw-ing from their career experience and projects thus far.

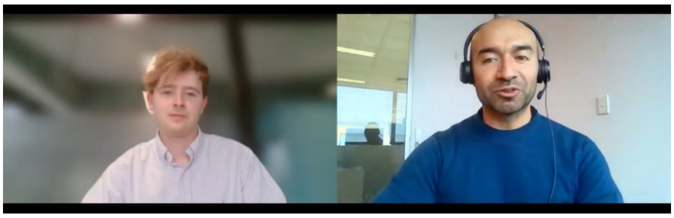
We were thrilled to hear from:

- Jake Togher (Northrop Consulting Engineers) on managing time and projects.
- Blair Corbett (Beca) on what makes structural engineering so rewarding.
- Ziqi Yang (Blue Barn Consulting) on nonlinear time-history modelling in a substation seismic assessment.

Thank you to each of the above speakers for sharing your experience, insights and energy. Your talks sparked great discussion amongst the crowd in the room, with some tasty pizza to go along with it!

A big thank you to the attendees for your engagement/ questions, and to Mitchell Vranjes for kindly hosting the event. We look forward to more of these events soon - keep an eye out for the next one!

SESOC ESE Short Interview - February 2026



Julio Ortiz recently sat down to talk to Henry Rowden, Chair of the SESOC ESE Group, to talk about what re-ally matters to emerging engineers. This short interview explored:

- What emerging engineers are looking for in their workplaces.
- How senior engineers and leaders can better support development.
- Why connection, purpose and growth opportunities matter more than ever.

If you missed the short interview, make sure to check it out on SESOC's LinkedIn page!!

UPCOMING PRESENTATIONS

- SESOC ESE x SEAONC YMF Collaboration Event 'Structural Engineering Across the Pacific Ocean 2026' – late-March 2026.

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 642 people on our mailing list with a 55% average open rate.
- ASG events hosted in 2025: 11 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

8th September 2025 - Design Recommendations for Concrete Wall-to-Steel Beam Bolted Web Plate Connections

On 8th September 2025, Claire Pascua (Graduate Engineer at Holmes) presented to ASG at the Holmes Auckland office on 'Design Recommendations for Concrete Wall-to-Steel Beam Bolted Web Plate Connections', covering key findings from her PhD, including a series of design recommendations based on these findings and capacity design principles. There was a great turnout of around 60-70 people.

17th November 2025 - Changes to the Earthquake-Prone Building System

On 17th November 2025, Ken Elwood (UoA Professor/ Chief Engineer (Building Resilience) for MBIE) presented to ASG at UoA on 'Changes to the Earthquake-Prone Building System', covering the background of the EPB system review, key elements of the new system, and the steps ahead. There was a good turnout of around 40-45 people, including a great Q&A session at the end.

UPCOMING PRESENTATIONS

- **23rd February** - Maryam Hasanali (HERA) & Ali Rad (Reconstruct) are presenting to ASG on 'Unresolved Design Issues with Steel Eccentrically Braced Frame System'.

Waikato-Te Moana-a-Toi BoP Structural Group

Chair: Clark Taylor
Contact: clarkt@bcdgroup.nz

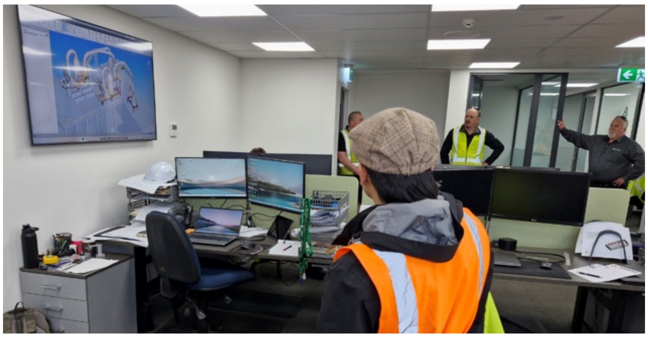
LOCAL UPDATE

- We are looking for more events in Kirikiriroa Hamilton, Taupo, Rotorua or Tauranga. If you have a presentation you would like to share with the group or sites we can visit, please reach out to the chair.
- If you have any questions, please contact the chair.

RECENT PRESENTATIONS

October 7th - JP Marshall site visit

A site visit was held at steel fabricator JP Marshall in Kirikiriroa Hamilton to view their operations. Around 15 people attended.



October 29th - NZ Steel EAF presentation

NZ Steel held a presentation in Kirikiriroa Hamilton on getting spec-ready now for EPD-backed, lower-carbon reinforcing & plate, and what changes as NZ Steel's electronic arc furnace (EAF) comes online in Mar/Apr 2026.

Te Matau-a-Maui Hawkes Bay Structural Group

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

LOCAL UPDATE (22/2/2026)

- Limited events held since November 2025 update.
- Attendance remains a struggle with emphasis being placed in mustering commitment from local firms to send their staff along, particularly juniors who are often not SESOC members
- We welcome opportunities for webinars and hosting recorded presentations. Frequency of seminars and speakers is less in the regions so if there is anything that may be of interest, please let me know.

RECENT EVENTS

November 18th – Seminar Series: Low Damage Seismic Design

Almost 20 attendees being a mix of structural and geotechnical engineers with a forestry engineer and project manager thrown in for good measure.

UPCOMING EVENTS

March 16th – Seminar Series: Steel Portal Frame Design

Currently there are limited registrations at just 11 persons but expecting circa 15 persons with recent commitment from firms to send their staff.

Whakatū Nelson Structural Group

Chair: Jaden Whiunui
Contact: jaden@tutika.nz

LOCAL UPDATE

October 2025 - Tier 2 RBA Training

In October 2025, a number of local engineering and building professionals attended the Tier 2 Rapid Building Assessment (RBA) training, delivered by MBIE's Building Emergency Management team in Whakatū Nelson. The training focused on preparing participants to operate effectively within post-disaster environments and to take on leadership roles as Tier 2 Building Assessor Team Leaders.

The course provided a comprehensive overview of how buildings are managed during emergencies, including the legislative framework that underpins the RBA system and the roles of MBIE, territorial authorities, and Rapid Building Assessors. Particular emphasis was placed on understanding assessor responsibilities, team structures, and decision-making under time-critical and high-pressure conditions.

The training also reinforced the importance of health, safety, and wellbeing for assessors working in challenging field environments, alongside the need for consistent and defensible assessment outcomes. Overall, the Tier 2 RBA training strengthened local readiness and capability, ensuring Whakatū Nelson has trained professionals able to support effective and timely building assessments following future emergency events.

UPCOMING PRESENTATIONS

March – date TBC EPB with Bruce Mutton

Bruce is ready to guide us through the proposed changes to the EPB (Earthquake Prone Building) legislation, helping us understand what the updates could mean for local stakeholders and highlighting key points raised by Engineering New Zealand.

Future – date TBC

3 sites visits are being worked on and we are looking at getting permission from owner, user, designers.

Waitaha Canterbury Structural Group

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

LOCAL UPDATE - 2025 RECAP OF CSG EVENTS:

Meetings have been hosted at the UoC Campus, Christchurch City Council building, Arts Centres and Beca's Office.

Attendance at the meetings has ranged from 35- 60 attendees, with an average meeting attendance of 40-45. Site visits typically have a limit of 20-30 attendees. Due to the high demand, attendees need to book a ticket. These events are booked out within hours of listing the event, and some booked out within minutes. Attendees include a wide range of experience from university students to retired engineers.

- **(5) Technical presentations**
    - 66 Oxford Terrace – (Seismic Retrofit)
    - CCC on Adrian Collis
    - IStructE President and CEO
    - The Arts Centre Strengthening
    - Project Showcase – Timber Structures
- **(4) Site tours**
    - 200 High St – 6-story steel-framed BRB building
    - 211 High St – 4-storey mass timber post & beam building with Tectonus Braces
    - John Jones Steel Steel Fabrication Plant
    - Project Ark – Strengthening of the Noah's Hotel - 14-storey in situ concrete building
- **(2) SESOC seminars**
    - Cold Form Steel Design
    - Low Damage Design

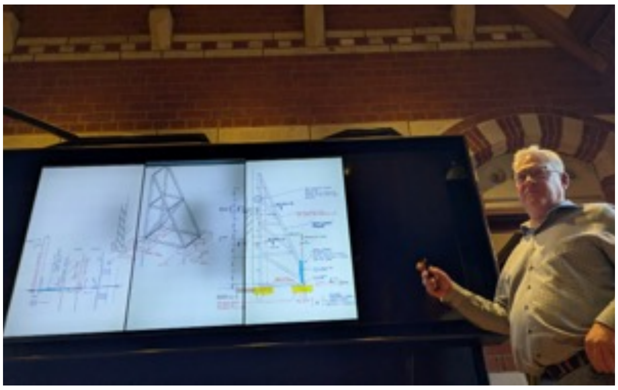
CURRENT UPDATE:

- We currently have over 500 members subscribed to the CSG email mailing list. This list is used to notify members of upcoming CSG events. To get added to the CSG mailing list, or suggest ideas for presentations or site visits, please contact CSG Chair - matt@h3structural.co.nz
- In November, Stu Oliver, Craig Lewis, and Matt Stewart met with Christchurch City Council (CCC) regarding the status and progress on the Adrian Collis work and quality assurance for engineering work. CCC is scheduled to present to CSG again in May.
- 50 Year anniversary of CSG this year. We have started to plan a celebration event for June – more information to come.
- CSG is looking for a volunteer to assist with event planning and meeting setup.
- The CSG maintains regular contact with UoC, NZGS, and CCC to explore opportunities for meetings and presentations. We also reach out to other technical interest groups such as TDS, SESOC Sustainability Task Force, NZSEE, Engineering NZ, and the local SESOC ESE. We welcome opportunities to collaborate with these groups on shared presentations.

RECENT EVENTS

16 October 2025 – The Arts Centre Strengthening CSG presentation on the IStructE award-winning project, Arts Centre Strengthening by the Holmes Group.

John Hare (Holmes), John Trowsdale (Holmes) & Brian Wood (Holmes Wood Poole & Johnstone)



The presenters, Brian Wood, John Hare, and John Trowsdale, took us on a journey from Brian Wood taking UoC classes at the Arts Centre in the 1950's and later leading strengthening works that provided the structures with the load paths and strength to survive the Canterbury Earthquake Sequence (CES) to John Trowsdale leading the rebuilding and strengthening from CES to now.

Having the presentation at the iconic Ernest Rutherford Den at The Arts Centre, in the suite of buildings at the centre of the presentation, made it really special and reminded us how important these buildings are.

The social and presentation were well attended with about 60 attendees.

28 October 2025 – SESOC Seminar – Low Damage Design

Stu Oliver (Holmes) & Jan Stanway (WSP)

27 November 2025, Project Showcase - Timber Structures - at the Beca Office

Kohinga St. Albans Community Centre

– Emma O'Neil (PTL)

Queenstown Country Club

– Mike Cusiel – ENGCO

AgResearch Tuhiraki Lincoln Workspace

– Peter Mai - Beca

The social and presentation were attended by 40 people. All three presentations were about new design timber projects with material types ranging from light timber framing to CLT to post and beam construction. Some common topics and key takeaways in these timber project presentations included tips on fire design, shop drawing review, connections, and moisture management plans. The Q&A session included insightful questions that led to further discussion.

6 March 2026 – Site Tour Ōmōkihi, South Library and Service Centre

Chris Hurst (Quoin), Paul Anselmi (Ignite), Rob Armstrong (Cook Brothers)

Site tour of the $32M rebuild of Ōmōkihi, the new South Library and Customer Services building in Beckenham. It was the site of the earthquake-damaged South Library. The new design called for demolishing the superstructure and retaining the ground-floor slab and foundations. The new foundation and superstructure are built on top of 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 is a combination of steel moment frames and steel braced frames.

Hearing the architectural drivers, followed by the structural challenges and solutions, and input from the builder led to an interesting site tour and a great learning opportunity for the attendees.

UPCOMING PRESENTATIONS

- CCC Update to CSG – May
- Canterbury Museum – May
- 50 Year anniversary of CSG – June
- 3-storey Light Frame Timber Building Testing at UoC – June/July

Ōtepoti, Otago Structural Group

Report for SESOC Management Committee Meeting – February 2026

Prepared by Taylor Koens / Simon Burrough

LOCAL UPDATE

- A period of uncertainty hangs over the market as owners and the council consider how to deal with the upcoming EPB changes in the interim. This is complicated by Dunedin shifting to a medium zone. Looking forward to draft guidance and firmer timelines.
- Central Otago continues to be busy with large number of construction projects underway. Staff shortages have been limiting growth in the short term.

RECENT PRESENTATIONS

- Presentations paused over summer, restarting in the coming weeks.

UPCOMING PRESENTATIONS

- Queenstown
  - 12th March – SESOC Seminar for Portal Frame Design.
  - April – Meeting with CODC Building Services team.
  - June – Meeting with QLDC Building Services team.
  - TBC – Target two site visits throughout the year.
- Dunedin
  - 19th March - C8 'Unreinforced Masonry Buildings' - an overview of last year's changes to the assessment guidelines and a group discussion.

BRAIN TEASERS ANSWERS

1. CANTILEVERED DECK:

In fact no – the deck would likely collapse. The moments for the original deck were such that the entire top of the deck was in tension – and hence the reinforcing steel was all placed on the top face of the concrete. In adding supports to the exterior line of the deck it now becomes a propped cantilever and likely experiences moments on the underside of the deck. As there is likely no bottom reinforcing steel the concrete on the bottom goes into tension and fails.

2. MOVING LOAD:

Calculate the Resultant Load and Position:

Total load (P):
$P = P_b + P_s = 60 + 40 = 100$ kN

Resultant Position (x_R):
Calculate the location of the resultant R from the larger load (P_b).

Taking moments about P_b:

$$100 \text{ kN}.x_R = 40 \text{ kN}.5\text{m}$$
$$x_R = \frac{200}{100} = 2 \text{ m}$$

So, the resultant is 2 m from the 60 kN load.

Position the loads for maximum moment

The maximum moment always occurs under the larger load when the centre of the beam lies exactly midway between that load and the resultant of all loads

Distance from P_b to R = 2 m

Midpoint between P_b and R is 2 m / 2 = 1 m

So, place the 60 kN load P_b to the left of the centre (5 m point)

Location of P_b: 5 m – 1 m = 4 m from left support A

Location of R: 5 m + 1 m = 6 m from left support A

Location of P_s: 4 m + 5 m = 9 m from left support A

Note that the 40 kN load is 9 m from the left (still on the 10 m span).

Calculate the reaction at support A (R_A)

We could use the formula for maximum moment, $M_{max} = (PL – P_s d)^2/4PL$

but from first principles calculate R_A:

$$R_A.10 \text{ m} = 60 \text{ kN}(10 – 4) \text{ m} + 40 \text{ kN}(10-9) \text{ m}$$
$$R_A.10 = 60 \times 6 + 40 \times 1$$
$$R_A = 40 \text{ kN}$$

Calculate the maximum bending moment:

The maximum moment occurs under the larger load (P_b = 60 kN) which is 4 m from support A.

$M_{max} = M_{4m} = RA.4$ m

$M_{max} = 40 \text{ kN} \times 4 \text{ m} = 160$ kNm

This can be verified with the above formula for M_{max}.

3. INTERMEDIATE TIMBER PARTITION:

Yes. Particularly at basement level although all levels are susceptible due to differential creep and displacements. Due to creep and live load displacements the floors will place load on the intermediate timber framed walls below. This can cause overstress of the timber framing (potential buckling loads in the timber) and it also changes the behaviour of the concrete floor – resulting in negative moments in the concrete floor system over the timber framed wall and for which the floor was likely not designed.

Designers should use a deflection head detail to ensure relative vertical movement can be accommodated. Consideration should also be made to ensure that the timber framing does not provide unintended lateral load paths.

SIGNING DOCUMENTS DURING CONSTRUCTION (SMALL WORKS)

It has come to light that some engineers are still failing to properly review documentation prior to issuing instructions or signing off inspections. Site engineers need to be properly briefed before attending site, particularly for a start-up contract. Sending junior engineers is only recommended if they are familiar with the type of work and properly understand the process in terms of what is required of the site engineer.

The first port of call for the site engineer is to review the Council consented documents. On several occasions I have been involved with complaints where a job has turned sour and the inspecting engineer signed off a concrete pour without consulting the geotechnical engineer (or having evidence that the geotechnical engineer had preceded them to site) and without a building consent in place.

So, minimum requirements when arriving on site are:

1. Check with the contractor that a building consent document is on site. Refer ACENZ / ENZ guidance on guidance on construction monitoring;
2. Also other documentation such as drawings and specification need to be checked on site;
3. Review the council documentation and see whether a geotechnical engineer certification is required and whether there are any special requirements for the structural engineer to sign off;
4. Ensure that the contractor has safety procedures in place;
5. Discuss check points and test requirements with the contractor;
6. On subsequent site visits ensure that propping looks satisfactory and if not sure query the contractor;
7. Ensure that the contractor is provided with a site note listing inspection(s) and attach photos – state whether passed, partial pass or fail and what is required;
8. Ensure that the Contractor has the site note with attached photos, preferably before their booking with a Council.

EDITOR

IStructE Update

February 2026

Chair: Brad Nichols
Contact: brad.nichols@robertbird.com

LOCAL UPDATE

- IStructE Global safety, sustainability and resilience conference: Australasia session (Free and Online)
  - o 10th March from 6-8:30pm NZ time.
  - o BN arranging 2x NZ presenter slots, including the following topics:
    - SESOC sustainable design task force summary, and "Carbon in a Seismic Context" paper from 2025 conference.
    - EPB system and seismic risk management
- IStructE 2026 president: Brian Uy.
  - o Inaugural address: https://www.istructe.org/resources/career-profiles/brian-uy-president-inaugural-address/
  - o Key themes for the year:
    - Registration and supervision
    - Structural efficiency and embodied carbon
    - Technical competency and research
  - o Brian is currently Scientia Professor of Structural Engineering at the University of New South Wales, and chairs AS/NZS standards on steel and composite structures and bridges. Brian is also the vice president of IABSE.
- Structural Awards entries close 13 April
- SESOC/IStructE Affiliate scheme
  - o Refer to SESOC newsletter for details.

NATIONAL EMBODIED CARBON REPOSITORY

This first release provides free access to embodied carbon data for thousands of construction products and materials available in Aotearoa New Zealand, supporting informed and responsible design decisions. Developed by BRANZ and CIL Masterspec, it is the only MBIE-endorsed project for a national carbon repository and replaces any previous static spreadsheets with a maintained, regularly updated dataset standardised for Aotearoa NZ.

It includes major updates from earlier BRANZ data, with thousands more entries and refined assessment methods.

Under a collaboration agreement, BRANZ is the data Steward, ensuring ongoing scientific rigour, while CIL Masterspecs team of Architects are Custodians ensuring accurate industry translation and regular publication. Feedback is welcome through the "Contact Us" option and more information on the project is available here.

CSG CELEBRATIONS

On 28 June 2026 (date as yet to be verified) Canterbury Structural Group will be celebrating 50 years. With over 500 members this will be a significant milestone and event for CSG.
For more information contact Matt Stewart matt@h3structural.co.nz.

SESOC Membership Report

Report for SESOC Management Committee Meeting – February 2026

Prepared by Jenni Tipler

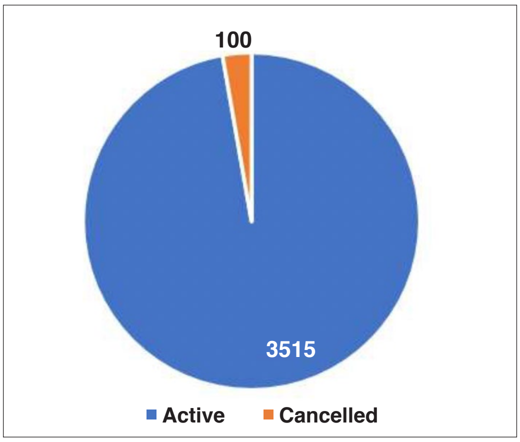
SESOC MEMBERSHIP SUMMARY AS AT 09.02.2026

Member Type	Feb-26	Nov-25	Aug-25	May-25	Feb-25
Complimentary Member	9	10	12	14	14
Journal Only	8	9	11	11	11
Journal Only Overseas	0	0	0	0	0
Life Member	20	20	20	19	19
Member	3034	3029	3067	3135	2918
Member Overseas	163	163	151	163	157
Member with IStructE(NZ)	157	160	161	160	160
Student Member	124	116	110	96	199
Total	**3515**	**3507**	**3532**	**3598**	**3478**
Growth over the Previous Quarter	8	-25	-66	145	25
Growth over the Previous Year	1%	-1%	3%	3%	-1%

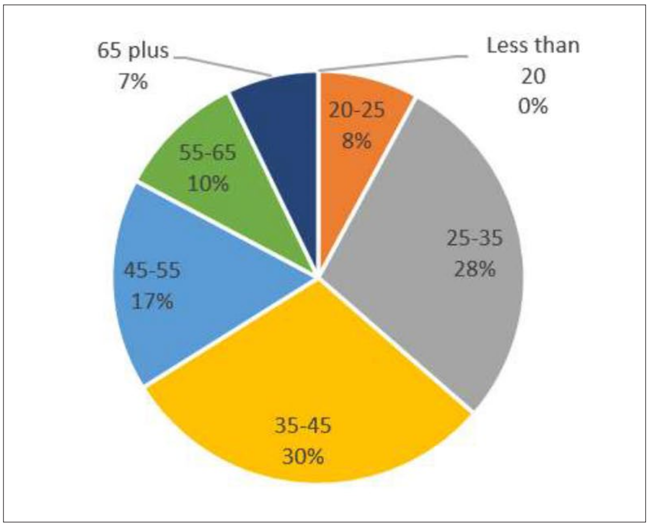
MEMBERSHIP GENDER SPLIT

Male	83.3%
Female	14.2%
Gender Diverse	0.0%
Not disclosed	2.5%

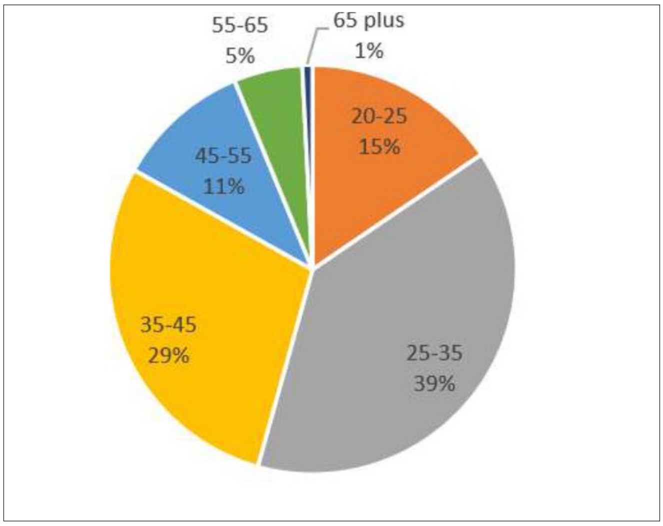
MEMBERSHIP STATUS



TOTAL MEMBERSHIP BY AGE



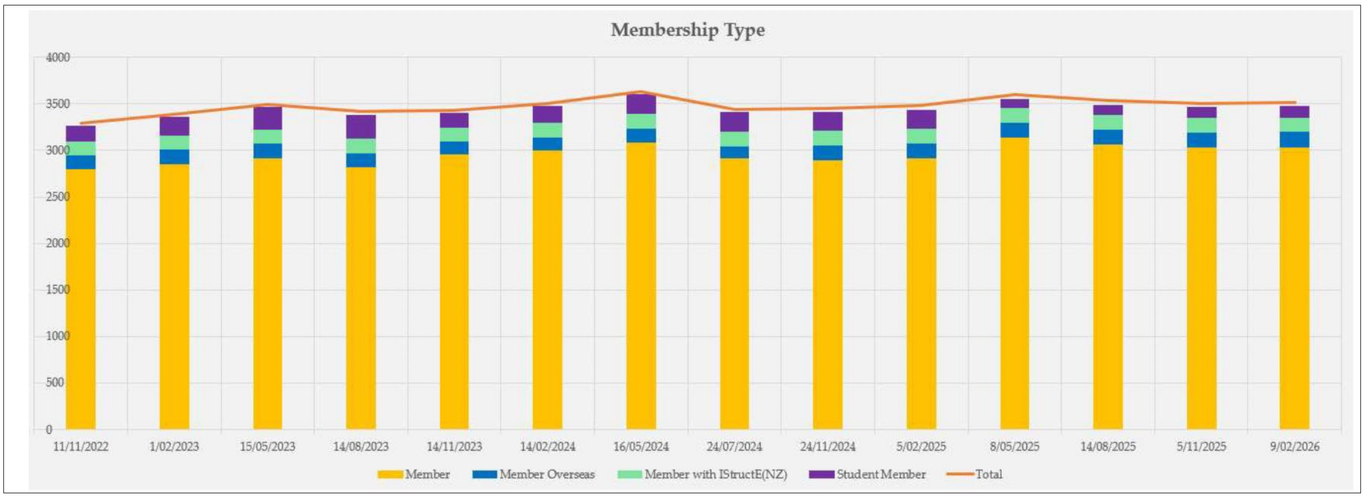
WOMEN - MEMBERSHIP BY AGE



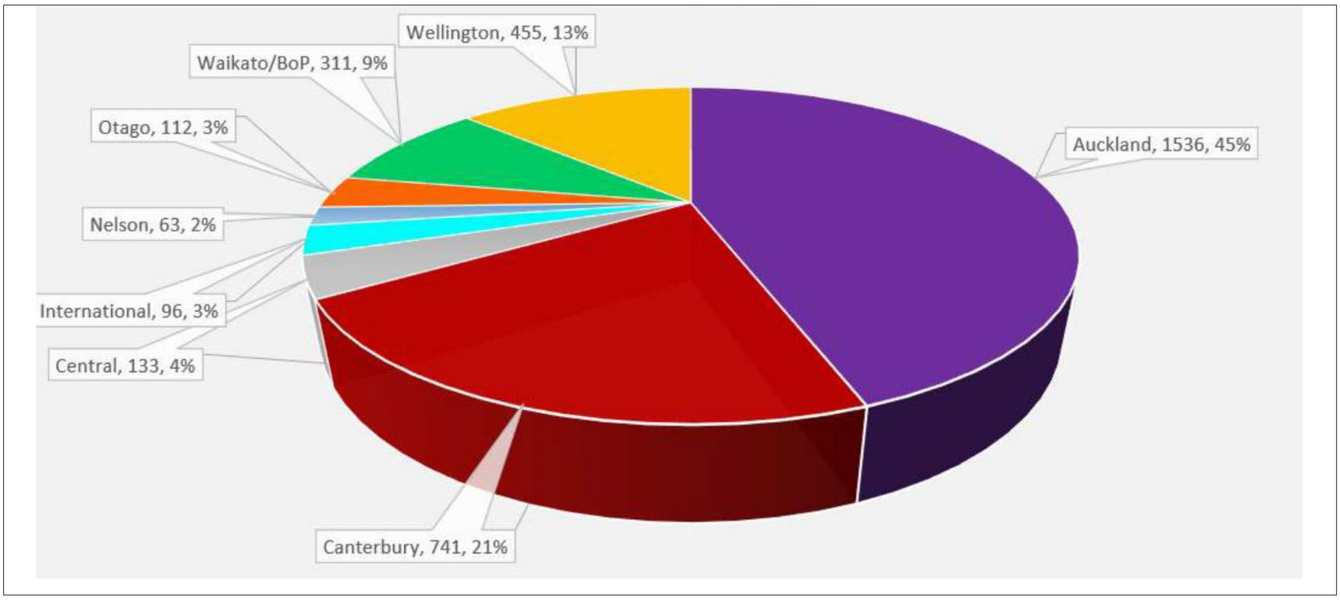
SESOC LIFE MEMBERS

Barry Brown	1996	Geoff Bird	2016
Ian Billings	2002	David Brunsdon	2016
Richard Fenwick	2009	Des Bull	2017
Barry Davidson	2009	Charles Clifton	2017
Trevor Robertson	2010	Michael Stannard	2017
Ashley Smith	2010	Paul Campbell	2018
Mark Batchelar	2012	Jason Ingham	2020
Robert Jury	2013	Stewart Hobbs	2023
Gordon Hughes	2016	Hamish McKenzie	2023
John Hare	2016	Michelle Grant	2025

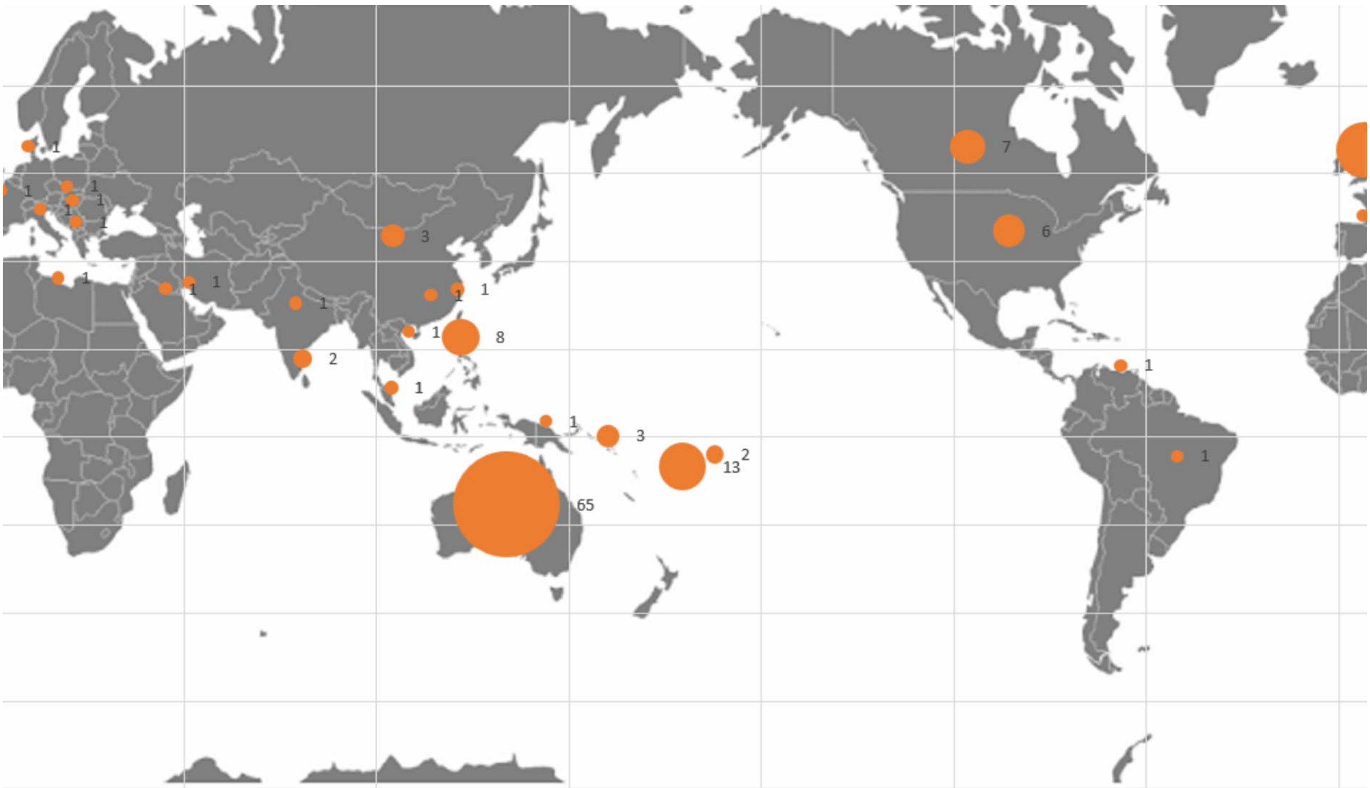
MEMBERSHIP GROWTH



MEMBERSHIP BY REGION OF AOTEAROA NEW ZEALAND



INTERNATIONAL MEMBERSHIP



SESOC TREASURER'S REPORT

Management Committee Meeting 25th February 2026

Based on Engineering New Zealand Account Reporting to 31 January 2026

Current Assets

	Accounts Receivable	$36,913
	Westpac Cheque 08 (ENZ)	$317,242
	Westpac Term Deposits (ENZ)	$150,000
	Westpac Trust Investment (SESOC)	$395,000
	Westpac Base 00 Account (SESOC)	$3,266
	Westpac Bonus Saver 25 Account (SESOC)	$40,415
	ENZ Receivables	$0
	Total Current Assets	**$942,836**

Current Liabilities

	Sundry accruals	$0
	Accounts payable (Includes $42,866 for LDSD Project)	$48,653
	GST	$2,010
	Total Current Liabilities	$50,663
	Nett Assets over Liabilities	**$892,173**

Comparison

	February 2025	$772,545

Income & Expenditure

	For the Financial Year to 30 September 2026, to date	
	Income	$362,359
	Expenditure	$60,637
	Nett Surplus	**$301,722**

John Snook
SESOC Acting Treasurer

Structural Engineering Society New Zealand (Inc.)
Membership Application

A Technical Group of Engineering New Zealand

Send to: Charlotte Toma
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